



FOUNDATION INVESTIGATION AND DESIGN REPORT

Replacement of Hornepayne Creek Culvert
Site No. 38N-0004/CO
Highway 631, Township of Wicksteed, Ontario
MTO Contract DB 2021-5168

Submitted to:

Facca Incorporated

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Table of Contents

PART A - FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	1
3.0 INVESTIGATION PROCEDURE	2
4.0 SUBSURFACE CONDITIONS	4
4.1 Regional Geology	4
4.2 Subsurface Conditions	4
4.2.1 Subsoil Conditions	4
4.2.2 Groundwater Conditions	5
5.0 CLOSURE	6

PART B - FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS	8
6.1 General	8
6.2 Consequence and Site Understanding Classification	9
6.3 Foundations	9
6.3.1.1 Design Tip Elevation	9
6.3.1.2 Geotechnical Axial Resistance	10
6.3.1.3 Set Criteria and Pile Driving Note	10
6.3.1.4 Downdrag Loads	11
6.3.1.5 Resistance to Lateral Loads	11
6.3.1.6 Group Effects for Lateral Loading	12
6.3.2 Frost Protection	12
6.3.3 Seismic Considerations	13
6.4 Stability and Settlement	13
6.5 Embankment Backfill and Lateral Earth Pressures	13
6.6 Construction Considerations	15
6.6.1 Excavations and Control of Groundwater and Surface Water	15

6.6.2	Temporary Protection Systems.....	15
6.6.3	Obstructions	16
6.6.4	Existing Watermain Monitoring	16
6.6.5	Analytical Testing for Construction Materials.....	17
7.0	CLOSURE	17

REFERENCES

DRAWINGS

Drawing 1 Borehole Locations and Soil Strata

PHOTOGRAPHS

Photographs 1 to 4

FIGURES

Figure 1 Model 1 – Deflection Towards Creek Bed (Single 360x132 HPile)
 Figure 2 Model 2 – Deflection Away from Creek Bed (Single 360x132 HPile)
 Figure 3 Settlement Monitoring Point Locations

APPENDIX A RECORD OF BOREHOLES

Lists of Abbreviations and Symbols
 Record of Boreholes HP-1 to HP-6

APPENDIX B LABORATORY TEST RESULTS

Table B1 Summary of Analytical Testing of Soil Sample
 Figure B1 Grain Size Distribution – Sand (Fill)
 Figure B2 Grain Size Distribution – Silt to Silt and Sand
 Figure B3 Grain Size Distribution – Sand to Gravelly Sand
 Figure B4 Grain Size Distribution – Sand to Gravelly Silty Sand to Sand and Gravel (Till)

APPENDIX C DESIGN BUILD SPECIAL PROVISIONS

DBSP with fill-ins DBSP0903 – Construction Specification for Deep Foundations
 Notice to Contractor Obstructions
 NSSP Vibration Monitoring for Existing Watermain

PART A

**FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF HORNEPAYNE CREEK CULVERT
SITE NO. 38N-0004/CO
HIGHWAY 631, TOWNSHIP OF WICKSTEED,
ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
CONTRACT DB 2021-5168**

1.0 INTRODUCTION

WSP Golder (formerly Golder Associates Ltd., now a member of WSP Canada Inc. hereafter referenced as WSP Golder) has been retained by Facca Incorporated (Facca) on behalf of the Ministry of Transportation, Ontario (MTO) to provide Foundation Engineering services to support detail design of the Design Build (DB) project for the replacement of the Hornepayne Creek Culvert (Site No. 38N-0004/CO), located on Highway 631 about 97 km north of Highway 17 in Wicksteed Township, Ontario. The key plan showing the general location of this section of Highway 631 and the location of the investigated area are shown on Drawing 1.

The scope of this report addresses the foundation component of the culvert replacement. WSP Golder (formerly Golder Associates Ltd.) was previously retained by LEA Consulting Ltd. (LEA) in 2017 on behalf of the MTO to carry out site-specific foundation investigation services for the previous detail design assignment of the replacement structure. Based on the subsurface information collected at the site (including the borehole drilling and laboratory testing program), this report provides sufficient geotechnical data and geotechnical design parameters and foundation design recommendations for the proposed culvert replacement strategy.

2.0 SITE DESCRIPTION

The existing Hornepayne Creek Culvert consists of a twin 29 m long ellipse corrugated steel culverts, each 3.5 m wide. The approximate invert of the culvert is Elevation 320.5 m. In general, the topography of the site and surrounding area is relatively flat with dense tree cover beyond the highway right-of-way. The existing approach embankments are about 3 m to 4 m high relative to the creek. The water level in Hornepayne Creek was measured at the culvert site at Elevation 320.9 m in November 2016 and June 2017.

Photographs at the culvert area are shown on Photographs 1 to 4, following the text of this report.

3.0 INVESTIGATION PROCEDURE

As previously mentioned, site specific investigation for detail design was carried between 7 June and June 2017, during which time six boreholes (HP-1 to HP-6) were advanced at the locations shown on Drawing 1. The borehole records are presented in Appendix A. The field investigation was carried out using the following drilling equipment:

- Boreholes HP-1 and HP-2 were advanced using a CME-55 truck-mounted drill rig supplied and operated by Landcore Drilling Inc. (Landcore) of Sudbury, Ontario.
- Boreholes HP-3 to HP-5 were advanced using a portable tripod drilling equipment supplied and operated by Downing Drilling Inc. (Downing) of Grenville-sur-la-Rouge, Quebec. A dynamic cone penetration test (DCPT) was advanced adjacent to each of these three boreholes.
- Borehole HP-6 was advanced using a CME-55 track-mounted drill rig supplied and operated by Downing.

The boreholes were advanced using 108 mm inner diameter hollow stem augers and/or NW casing and wash boring. Soil samples were obtained in the boreholes at 0.75 m and 1.5 m intervals of depth using 50 mm outer diameter split-spoon samplers driven by an automatic hammer on the drill rigs in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586) and a manual half-weight hammer (Acker) on the portable drill rigs dropped from the SPT height and the blow counts were corrected to the inferred values that would be obtained with a standard weight hammer. A dynamic cone penetration test (DCPT) was carried out from ground surface adjacent to Boreholes HP-3 to HP-5. The groundwater level in the open boreholes was observed during and immediately following the drilling operations as described on the borehole records in Appendix A. The boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The field work was supervised on a full-time basis by a member of Golder's staff, who located the boreholes in the field, cleared the site for buried services, directed the drilling and sampling operations and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's Sudbury Laboratory for further examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Index and classification tests consisting of water content, Atterberg limits and grain size distribution were carried out on selected soil samples. The results of the laboratory testing on samples from the boreholes are presented on the borehole records in Appendix A, and on the grain size distribution figures in Appendix B.

A soil sample was obtained on 8 June 2017, from Borehole HP-2, using appropriate sampling protocols and submitted to a specialist analytical laboratory under chain of custody procedures for testing for a suite of parameters including pH, resistivity, conductivity, sulphates and chlorides. The results of the analytical testing are presented in Table B1 in Appendix B.

The borehole locations and elevations were measured in the field by Golder personnel, relative to existing site features and surveyed to a reference benchmark. The borehole locations (referenced to the MTM NAD83 Zone 13 co-ordinate system), ground surface elevations (referenced to Geodetic datum) and borehole depths at refusal, except for Borehole HP-6, are presented on the borehole records in Appendix A, and are summarized below.

Borehole	Location (MTM NAD 83, Zone13)		Location (WGS84)		Ground Surface Elevation (m)	Borehole Depth (m)	DCPT Depth (m)
	Northing	Easting	Latitude	Longitude			
HP-1	5451566.6	247796.2	49.199525	-84.782251	323.8	12.5	-
HP-2	5451591.8	247798.2	49.199752	-84.782227	324.0	13.9	-
HP-3	5451599.5	247812.8	49.199822	-84.782020	321.2	5.3	8.8
HP-4	5451588.3	247817.7	49.199722	-84.781959	321.2	5.5	5.5
HP-5	5451582.2	247777.8	49.199664	-84.782506	321.7	5.2	7.1
HP-6	5451564.7	247785.8	49.199507	-84.782393	321.4	9.8	-

4.0 SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain (NOEGTS)¹ mapping, the Hornpayne Creek Culvert is located within an outwash plain, consisting primarily of sands and silts, bordered by bedrock knobs.

Based on geological mapping by the Ontario Ministry of Northern Development and Mines (MNDM)², the site is underlain by bedrock from the metasedimentary suite of rocks comprised of wacke, arkose, argillite, slate, marble, chert, iron formation, minor metavolcanic rock.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes together with the results of the laboratory tests carried out on selected soil samples, are presented on the borehole records in Appendix A, and the laboratory test sheets in Appendix B. The results of the in situ field tests (i.e., SPT 'N' values) as presented on the borehole records and in Section 4 are uncorrected, except for those obtained by use of the half weight hammer as noted in Section 3.0. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations. A summary of the subsurface conditions as encountered in Boreholes HP-1 and HP-6 is presented below.

Groundwater levels/conditions encountered in the boreholes during and shortly after drilling may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized. Groundwater and creek levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt.

4.2.1 Subsoil Conditions

A description of the soil deposits encountered in the boreholes is provided below.

Deposit/Layer Description	Boreholes	Deposit Surface Elevation (m)	Deposit Thickness (m)	N Values (blows)	Laboratory Testing
				Relative Density	
Asphalt	HP-1, HP-2	324.0, 323.8	0.05 – 0.06	n/a	n/a
(FILL) Silty SAND to Sand to Gravelly Sand	HP-1, HP-2, HP-5 and HP-6	323.9 - 321.3	0.3-2.6	N = 3 – 47 Very loose to Dense	w = 3% – 6% 2 – M (Fig. B1)
PEAT and Silty Topsoil	HP-3 to HP-6	321.7 - 321.2	0.1-1.4	N = 1 – 3 Very soft to soft	w = 58% and 139%
SILT to SILT and SAND	HP-1, HP-2, HP-5 and HP-6	321.7 - 320.2	2.5 – 6.1	N = WH (0) – 31 Very loose to Dense	w = 18% – 40% 8 – MH (Fig. B2) 2 – AL (NP)

¹ Ontario Ministry of Natural Resources and Forestry. Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 42FNE.

² Ontario Ministry of Northern Development of Mines. Bedrock Geology of Ontario – East Central Sheet, Ontario Geological Survey – Map 2543.

Deposit/Layer Description	Boreholes	Deposit Surface Elevation (m)	Deposit Thickness (m)	N Values (blows)	Laboratory Testing
				Relative Density	
SAND to Gravelly SAND	HP-2, HP-3, HP-4 and HP-6	320.9 - 315.3	0.7 - 4.5	N = 1 – 16	w = 15% – 22% 6 – MH (Fig. B3)
				Very loose to Compact	
Boulder	HP-2	317.7	0.9	N/A	N/A
(TILL) Gravelly Silty Sand to Sand and Gravel and Cobbles	HP-1, HP-2 and HP-6	316.8 - 313.8	> 2.2 – 6.7	N = 18 – 50/0.08	w = 10% – 20% 3 – MH (Fig. B4)
				Compact to Very dense	

Where:

N = SPT 'N' values; number of blows for 0.3 m of penetration.

w = Natural moisture content (%).

M = Sieve analysis.

MH = Combined sieve and hydrometer analysis.

AL = Atterberg Limits.

¹A 0.1 m thick layer of peat and silty topsoil was encountered over the granular fill in Boreholes HP-5 and HP-6, respectively.

4.2.2 Groundwater Conditions

The depths to/elevations of unstabilized groundwater levels measured in the open boreholes upon completion of drilling are presented below. Water levels should be expected to vary depending on the time of year and precipitation events.

Borehole No.	Depth to Unstabilized Groundwater Level (m)	Approximate Groundwater Elevation (m)
HP-1	2.3	321.5
HP-2	3.0	321.0
HP-3	0.0	321.2
HP-4	0.0	321.2
HP-5	0.5	321.2
HP-6	0.6	320.8

The creek water level was surveyed by others at Elevation 320.9 m in November 2016 and June 2017.

5.0 CLOSURE

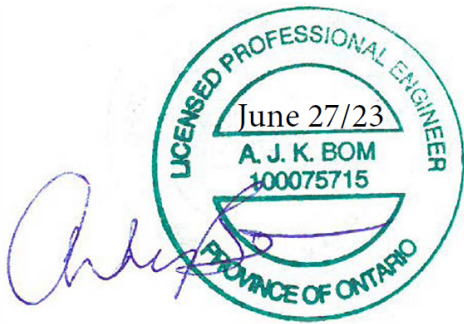
This Foundation Investigation Report was prepared by Ms. Aronne-Kay De Souza, P.Eng., and the technical aspects were reviewed by Mr. André Bom, P.Eng. Mr. Kevin Bentley, P.Eng., a Designated MTO Foundations Contact for WSP Golder, conducted an independent quality control review of this report.

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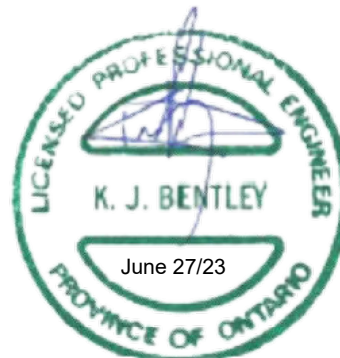
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PART B

**DETAIL FOUNDATION DESIGN REPORT
REPLACEMENT OF
HORNEPAYNE CREEK CULVERT –
SITE NO. 38N-0004/CO
HIGHWAY 631, TOWNSHIP OF
WICKSTEED, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
CONTRACT DB 2021-5168**

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation engineering recommendations for the proposed replacement of the Hornepayne Creek Culvert. The recommendations presented are based on interpretation of the factual data obtained from the boreholes advanced during the detail design subsurface investigation.

The discussion and recommendations presented are intended to provide the Design-Builder with sufficient information to complete the detail design of the proposed culvert foundations and associated approach embankments, and provide adequate geotechnical input / parameters for design of temporary structures. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required for construction.

6.1 General

The existing corrugated steel pipe (CSP) twin Hornepayne Creek culverts will be replaced with a proposed combined wall/precast concrete panel system that will span 10.6 m across the creek, with each wall located about 1 m beyond the outside edge of each of the existing twin culverts. The combined wall (combi-wall) system consists of driven structural steel H-piles (referred to as “king piles”) founded within the till layer to reach the required design axial geotechnical resistance for the foundations. A pair of permanent steel sheet pile will then be advanced between and interlocked / connected to the adjacent king piles to form the sides / wall of the culvert. Precast prestressed concrete panels will then be erected in segments to span across the creek and connect to the north and south combi-walls. The concrete panels will be waterproofed and covered with embankment fill / pavement structure to carry Highway 631 over Hornepayne Creek.

The creek bed inside the combi-wall culvert structure will be at about Elevation 320.6 m, about 0.1 m below the existing culvert invert. Each of the north and south combi-walls will be about 15.1 m long (measured along the centreline of the king piles). Permanent sheetpiles are also proposed at each quadrant of the culvert structure, extending from each of the four corners of the combi-wall system back about 6.5 m parallel to the highway centreline and beyond the shoulder, to allow for a reduced embankment footprint (minimize filling adjacent to the creek after removal of the twin culverts) and avoid impact to the existing watermain that currently runs below the east side of the existing culverts. Based on the as-built drawing dated December 2004, the watermain consists of a 200 mm diameter HDPE pipe installed by the Horizontal Directional Drilling (HDD) method. Reportedly, the watermain was installed in about 2004, at Elevation 318 m under the existing culvert end. We understand that daylighting of the watermain was completed in August 2018 by others and that the watermain was exposed at select locations. The approximate location of the watermain is shown on Drawing 1 and will need to be confirmed prior to commencing construction.

The new culvert structure will be constructed in two general stages with a temporary protection system proposed along the approximate centreline of Hwy 631 to allow for continuous flow of traffic during construction on the east and west half of the culvert. We understand that a slight grade raise (up to 200 mm relative to existing ground surface in some areas) is proposed to accommodate the new combi-wall system and highway widening is not required.

6.2 Consequence and Site Understanding Classification

A “Typical” consequence level is considered appropriate for the Horneypayne Creek culvert replacement as outlined in Section 6.5 of the Canadian Highway Bridge Design (CHBDC 2019) and its Commentary. Given the site-specific foundation field investigation and laboratory testing program, a “Typical” degree of site and prediction model understanding has been utilized. Accordingly, the appropriate corresponding Ultimate Limit States (ULS) and Serviceability Limit States (SLS) consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC (2019) have been used for design.

6.3 Foundations

Steel H-piles (king piles) will be driven into the “100-blow” till (with spacing of the king piles at about 1.9 m) to support the culvert structure. The king piles could consist of HP310x110 piles although due to the presence of cobbles and boulders within the overburden, which could cause the piles to “hang up”, twist or be deflected from their intended vertical alignment, consideration should be given to using a heavier H-pile section (such as HP310X132 or HP360x132) to reduce the potential for damage to the piles during driving to the required tip elevation.

The following sections provide details regarding the design tip elevation, geotechnical axial resistances, set criteria and pile driving notes, resistance to lateral loads and frost protection for driven steel H-piles.

6.3.1.1 Design Tip Elevation

The piles should be advanced to found on the “100-blow” sand and gravel to gravelly silty sand (till) at the estimated design tip elevations as follows:

Foundation Element (Relevant Boreholes)	Proposed Top of Piles (bottom of concrete panel) (m)	Estimated Design Pile Tip Elevation (m)	Estimated Design Pile Length (m)	*Estimated Design Pile Embedment Below Creek Bed (m)
North Wall (HP-2, HP-3 and HP-5)	322.7	313.0	9.7	7.6
South Wall (HP-1, HP-4 and HP-6)	322.7	313.0	9.7	7.6

*Creek bed at Elevation 320.6 m. It is assumed that a minimum pile embedment length of 5 m below creek bed is to be used for design (to be confirmed by structural engineer).

Varying pile lengths from those presented above will need to be considered due to the piles possibly “hanging up” on the cobbles and boulders or driving deeper into the till than anticipated. The tip of the king piles and connecting sheet piles should extend below frost depth (minimum of 2.6 m as discussed in Section 6.3.2) measured from the proposed creek bed. In addition, the minimum length of king piles embedded below the creek bed is assumed to be 5 m (MTO, 2021), although the minimum lengths of king piles and sheet piles will need to be checked and verified by the structural designer for lateral resistance as discussed in Section 6.3.1.5.

Based on discussions with the Contractor, if a king pile does not reach the minimum 5 m depth below the creek bed, the pile will be removed, pre-drilling will be required to 5 m depth below the creek bed and the pile will be driven from the bottom of the pre-drilled hole. Further, for the sheet pile combi-wall in between the king piles, if the sheet pile does not reach the 2.6 m design depth then the sheet pile will be removed and pre-drilled to 2.6 m and the sheet pile will be driven below the pre-drilling depth. Further, for sheet pile wing walls, the Contractor will install rock protection points to penetrate to the design depths.

6.3.1.2 Geotechnical Axial Resistance

For HP310x132 or HP360X132 steel H-piles driven into the “100-blow” till at the tip elevations above, a factored ultimate geotechnical resistance of 800 kN per pile is applicable for design. The factored serviceability geotechnical resistance for 25 mm of settlement is estimated to be 575 kN for design.

Based on the results of the soil investigation, the surface of the “100-blow” till and presence of cobbles / boulders is not consistent at the borehole locations and piles driven for the north and south walls will be subject to the following risks:

- **“Early” Pile Refusal and Obstructions:** The presence of cobbles / boulders being encountered can lead to piles “hanging-up”, “deflecting” or “twisting” out of alignment. Given that the king piles will likely have specified tolerances to achieve an “interlocking” system with the sheetpiles, careful control of the installation procedure is recommended and may require temporary guides to maintain tolerance requirements. The presence of a boulder about 6.3 m below road surface (about 2.9 m below the proposed creek bed surface) in Borehole HP- 2 and auger/casing refusal encountered at depths of 5.2 to 5.5 m below ground surface in HP-3 to HP-5 (about 4.5 m below proposed creek bed surface) may interfere with king pile installation and pre-drilling may be required to achieve design pile tip elevations and tolerances.
- **Driving Piles Deeper:** During the soil investigation, effective refusal (i.e., “100-blow” soils or refusal to advance the augers / casing / DCPT) was achieved at the termination depth of each borehole except for HP-6 (e.g., near southwest quadrant). Although the results of the soil investigation suggest that competent “100-blow” soils are generally present at the site, there is a risk that piles may need to be driven deeper to achieve the design geotechnical resistance.

To further mitigate these risks, it is recommended that at least 25% of the production king piles be tested using High Stain Dynamic Testing methods (e.g., pile dynamic analyzer (PDA) testing). The PDA testing should be performed near the west, middle, and eastern portion of both the north and south wall foundation elements. The PDA testing would be calibrated with the pile driving records and in combination with Hiley test results using Standard SS103-11 (to be included in the Contract Drawings) to determine the set criteria for the remaining king piles. The specific requirement for PDA testing has been added to the “fill-in” portion of DBSP0903 (Construction Specification for Deep Foundations) in Appendix C.

6.3.1.3 Set Criteria and Pile Driving Note

Pile installation should be carried out in accordance with DBSP0903 (Construction Specification for Deep Foundations) which supercedes OPSS 903 for this contract and is included in Appendix C.

Based on the presence of the cobbles and boulders and gravelly till layer, the piles should be fitted with driving shoes such as Titus bearing pile point, standard model or reinforced flange plates as per Ontario Provincial Standard Drawing OPSP 3000.100 (Steel H-Pile Driving Shoe), or equivalent, to minimize damage to the pile tip during driving.

The pile driving note that should be added to the foundation drawing, in addition to the “pile design data” note (for ULS and SLS values), is similar to Note 2 in Clause 3.3.3 of the Structural Manual (MTO, 2021) as follows:

- “Piles to be driven in accordance with DBSP0903 using an ultimate geotechnical resistance of 1,600 kN per pile but must be driven below Elevation 315.6.”

Select piles should be re-tapped as per DBSP0903.

6.3.1.4 Downdrag Loads

As the foundation soils are cohesionless and generally compact to very dense in relative density, minimal settlement is anticipated as a result of the proposed embankment loading. As a result, downdrag loads need not be considered for design of the pile foundations.

6.3.1.5 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered piles. Where vertical piles are used, such as the case for the current combi-wall design, the resistance to lateral loading will have to be derived from the soil in front of the king piles. The evaluation of the piles subjected to lateral loads should take into account such factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moment, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects.

As such, the horizontal non-linear resistance offered by the soil surrounding a single 360x132 king pile (below the bottom of the sheet piles) for the two combi-walls was modeled using RSPile (Version 3.008, 2021) by Rocscience to generate p-y curves. The p-y curves are based on empirical correlations with the soil parameters estimated from the results of the field and laboratory testing and used the API Method for sand (API RP 2A) in the RSPile program. The following soil parameters and idealized soil stratigraphy were used in the model. For structural design, it should be noted that no soil resistance should be modeled for 2.6 m along the pile below ground surface and behind the exposed sheet pile wall (if no frost protection / insulation is provided) to account for disturbance to the soil due to frost effects. The elevation of the highway was modelled to be at El. 323.9 m, the creek bed at El. 320.6 m, and the groundwater level was modelled to be at Elevation 321 m for the analyses.

Soil Unit	Elevation (m)	Effective Unit Weight (kN/m ³)	c' (kPa)	Φ' (degrees)	Initial Modulus of Subgrade Reaction, k (kN/m ³)
New Granular B Type II Engineered Fill	Below pile cap (El. 322.7 m) to approximately creek bed (El. 320.6)	22	0	35	40,000
Loose to compact silt, sand, gravelly sand	Below proposed creek bed (El. 320.6 m) to dense till (El. 313.5 m)	19	0	28	4,400
Dense Till	Below approximately Elevation 313.5 m	20	0	34	20,000

The results of the RSPile generated static p-y curves at selected intervals of depth below the creek bed (to model deflections towards the creek) and below the pile cap (to model deflections away from the creek) are shown on Figures 1 and 2, respectively. The representative spring constants (k_h) for any loading condition can be generated at the depths shown on the tables by dividing the force P (kN/m per metre length of pile) by the corresponding deflection value y (m). The non-linear spring constant (k_h , kN/m/m) represents the coefficient of lateral subgrade reaction value at 1 m intervals. The resistance of the sheet pile wall between the king piles was not included in the models.

The structural resistance of the pile should be evaluated to establish the governing case at ULS, although the design is often governed by limiting deformations at SLS that will need to be considered in the overall design of the structure and compatibility with the adjacent approach embankments.

6.3.1.6 Group Effects for Lateral Loading

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters between rows of driven steel H-piles. Group action can be evaluated by reducing the spring constant (i.e., coefficient of lateral subgrade reaction) in the direction of loading by a reduction factor, R as follows:

Pile Spacing in Direction of Loading (D = Pile Diameter)	Subgrade Reaction Reduction Factor, R
8D	1.0
6D	0.7
4D	0.4
3D	0.25

Where a pile group is oriented perpendicular to the direction of loading, group action may be considered by reducing the coefficient of horizontal subgrade reaction by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading (d = Pile Diameter)	Horizontal Subgrade Reaction Reduction Factor, R
4 d	1.0
1 d	0.5

Reduction for group effects is generally negligible when the centre-to-centre pile spacing exceeds three pile diameters measured in the direction perpendicular to loading.

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above summary.

6.3.2 Frost Protection

The depth of frost penetration is 2.6 m at this site as interpreted from OPSD 3090.100 (Frost Protection Depths for Northern Ontario). As discussed in Section 6.3.1.1, king piles and sheet piles within the combi-wall system are anticipated to extend below 2.6 m below the creek bed to account for frost action. The permanent sheet piles to be advanced adjacent to the highway shoulders (at each quadrant behind the combi-wall system) should extend at least 2.6 m below final ground surface elevation. Further, the new granular fill to be backfilled behind the sheet pile walls should extend at least 2.6 m for frost protection purposes, as discussed further in Section 6.5.

6.3.3 Seismic Considerations

Subsurface ground conditions for seismic site characterization were established based on the results of the limited depth of the borehole investigations. Based on the anticipated foundation levels, the site may be classified as Site Class D in accordance with Table 4.1 of the CHBDC (2019), in the absence of any geophysical testing. Geophysics testing, if carried out, could provide a more favourable Site Class designation.

Based on the seismic hazard values obtained for the nearby Nagagamisis Narrows site and in accordance with Table 4.10 of the CHBDC (2019), this site is considered to be located in Seismic Performance Zone 1. In accordance with Section 4.4.5.1 of the CHBDC, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.4 Stability and Settlement

The existing embankment at the culvert site is considered stable from a geotechnical perspective, if reconstructed and backfilled with granular material at inclinations of no steeper than 2 horizontal to 1 vertical (2H:1V) beyond the proposed wing walls.

Considering the existing embankment will not be widened, negligible settlement is expected to occur from the proposed 200 mm grade raise in some areas. If grades are to be raised more than 200 mm or if widening is being considered, we recommend the embankment stability and settlement be checked.

The internal stability of the permanent sheet pile walls extending back from each quadrant of the culvert structure should be checked by the structural designer when final grades and creek restoration details are established. Provided the sheet pile wing walls greater than 2.6 m below final grade, the global stability of the sheet pile wing wall is considered acceptable.

6.5 Embankment Backfill and Lateral Earth Pressures

Excavation of existing fills and backfilling behind the combi-wall is recommended to ensure free-draining material and consistent lateral pressures on the back of the wall. The lateral earth pressures acting on the combi-wall system will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the sheet pile wall and upper portion of the king piles, with sheet piles extending to at least 2.6 m below the creek bed level to reduce frost effects as discussed in Section 6.3.1.1. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Free-draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II, should be used as backfill behind the combi-walls to a depth of at least 2.6 m below final asphalt grade / ground surface. Longitudinal drains and weep holes are not considered necessary for this site based on the proposed new granular backfill and existing granular soils encountered at / near the wall locations, which allow for drainage of water to the creek level. Compaction of the new granular fill (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement) and OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement) without the need for the 10 to 1 frost taper based on the foundation soils and Granular B Type II backfilled behind the wall at this site.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2019) Section 6.12.3 and Figure 6.8. Care must be taken during the compaction operation not to overstress the wall, with limitations on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.PROV 501. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 2.6 m behind the back of the wall (in accordance with Figure C6.31 (a) of the Commentary to the CHBDC). For unrestrained walls, granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing in accordance with Figure C6.31(b) of the Commentary to the CHBDC (2019).
- For restrained walls, the pressures are based on the proposed embankment fill material behind the structure backfill zone, while for unrestrained walls, the pressures are based on the granular backfill; the following parameters (unfactored) may be used:

Stratigraphic Unit	Bulk Unit Weight, γ (kN/m ³)	Angle of Internal Friction, ϕ' (degrees)	Undrained Shear Strength, s_u (kPa)	Lateral Earth Pressure Coefficients		
				Active, K_a	At-rest, K_o	Passive, K_p^1
New Embankment Fill - Granular 'B' Type II from below pavement structure to 2.6 m below asphalt surface (about El. 321.4 m)	22	35	-	0.27	0.43	3.70
Existing Embankment Fill (loose to compact sand) from below asphalt to existing culvert invert (about El. 320.4 m).	19	30	-	0.33	0.50	3.00
Loose to compact silt, sand, and gravelly sand from existing culvert invert (El. 320.4 m) to El. 313.5 m.	19 - 20	28 - 30	-	0.36 – 0.33	0.53 – 0.50	2.77 – 3.0

Note 1: The total passive resistance below the proposed creek bed (i.e., adjacent to the sheet piles) may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement, in accordance with Figure C6.27 of the CHBDC (2019), to account for the fact that a large strain would be required for mobilization of the full passive resistance.

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the foundation design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, should be calculated in accordance with Section C6.12.1 and Table C6.12 of the *Commentary to the CHBDC*, 2019.
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.6 Construction Considerations

6.6.1 Excavations and Control of Groundwater and Surface Water

Permanent excavations at this site should be limited to the removal of the existing culverts (and surrounding bedding/embedment/cover soils) and a portion of the existing embankment for creek restoration and placement of new creek bed material. Temporary support of the combi-wall may be required if excavation in front of the wall or unbalanced lateral forces occur prior to connecting the walls with the precast concrete panels. Excavations and embankment restoration should in general accordance with OPSS.PROV 206 (Grading).

Temporary open-cut excavations up to 2.6 m deep are proposed behind the combi-walls to place new granular backfill. Temporary excavations may also be required to place the new creek bed material. All temporary excavations must be carried out in accordance with the guidelines outlined in the latest version of the Ontario Occupational Health and Safety Act (OHSA) and Regulations for Construction Activities.

The existing fill materials above the groundwater are classified as Type 3 soils according to the OHSA. The wet, organic soils and the very loose to loose zones of the native silt to silt and sand, and sand to gravelly silty sand are classified as Type 4 soil according to the OHSA. Temporary excavations (i.e., those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V in Type 3 soils above the water level and no steeper than 3H:1V in the Type 4 soils. Open excavations below the water should be limited and backfilled immediately where applicable.

Surface water should be directed away from the excavation at all times to prevent ponding of water or erosion that could result in disturbance and weakening of the foundation subgrade. During removal of the existing culverts and embankment fill, temporary cofferdams / flow diversion is likely required to allow for construction in the dry, restoration / regrading of the channel and placement of the stream bed material. Dewatering of all excavations should be carried out in accordance with OPSS.PROV 517 (Dewatering) and temporary flow passage systems should be in general accordance with SP517F01 (Temporary Flow Passage System).

Groundwater flow into any temporary excavations below the water table can be expected due to the relatively permeable nature of the granular embankment fill and non-cohesive native soils. The organic soil and sands/silts that will be exposed within the excavation will be susceptible to disturbance from construction traffic, ponded and flowing water, and groundwater inflow and the exposed surface should be covered with appropriate creek bed material as soon as possible (continuous operation). The sheet pile penetration depth and creek bed erosion protection must be designed to resist scour for the design life of the structure.

6.6.2 Temporary Protection Systems

Temporary protection systems will be required near the middle of Highway 631 to accommodate traffic staging for construction of each side (east and west) of the combi-wall system. Temporary protection systems should be designed and constructed in accordance with DBSP 0539 (Temporary Protection Systems). The lateral movement of the temporary shoring systems should meet Performance Level 2 as specified in DBSP 0539, provided that any existing adjacent structures or utilities can tolerate this magnitude of deformation.

It is considered that either a driven, interlocking sheet pile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support, based on the subsurface soil and groundwater conditions. The sheet piles or soldier piles would have to be driven or socketted to sufficient depth to provide the necessary passive resistance for the retained soil height, including any surcharge loads behind the protection system within at least a 1H:1V zone relative to the base of the excavation. Lateral support to the sheet piles or soldier piles could be provided in the form of rakers or temporary anchors.

The selection and design of the temporary protection system will be the responsibility of the Contractor. Similar to the risks associated with the permanent combi-wall system, pile installation may be inhibited by the presence of obstructions within the existing fills, and cobbles / boulders encountered within the native silts and sands and till soils. Preaugering and/or coring through obstructions may be required to penetrate the obstructions / cobbles / boulders.

6.6.3 Obstructions

The native soils at this site are glacially derived and as such are very dense and contain coarse gravel, cobbles and boulders as noted on the borehole records, which could affect the installation of the king piles and sheet piles for the combi-wall, temporary or permanent excavations, and installation of temporary protection systems. A Notice to Contractor should be included in the Contract Documents to alert the contractor of the presence of cobbles and boulders within the overburden soils; an example is included in Appendix C.

6.6.4 Existing Watermain Monitoring

The current combi-wall footprint and permanent sheet pile wall extensions in each quadrant allow for reduced impact to the existing watermain on the east side. As shown on Drawing 1, the approximate location of the existing watermain is approximately 2 m and 4 m east of closest permanent sheet pile or king pile installation at the southeast and northwest quadrant respectively. Reportedly, the existing watermain was exposed at several locations during daylighting in the previous design phase in August 2018 and the interpreted location is shown on Drawing 1 but is considered to be approximate. We understand the Contractor attempted to expose the existing watermain in the Fall of 2022 by daylighting and was unable to expose it due to saturated sloughing/flowing sands collapsing in the open hole. For diligence purposes, the watermain must be accurately located (in plan and profile) prior to driving any piles and preferably prior to completing detail design. It is understood that not only is the watermain susceptible to impacts from construction operations, but is also located shallower than indicated (i.e., possibly within the frost penetration depth) and will need to be assessed further to mitigate risks to damaging or shifting the watermain during existing culvert removal, creek bed restoration and construction operations.

At this stage, the watermain should be exposed by daylighting on both sides of the creek and a minimum of four settlement monitoring points should be installed such that the base of each point is positioned near the invert or on top of the watermain. Two sets of points should be installed on either side of the creek (2 points in northeast and 2 points in southeast quadrant) near the proposed sheet pile installations and an additional two points could be considered between the culverts, if required; refer to Figure 3 for approximate possible settlement point locations. A settlement monitoring program should be established to include a baseline (a set of readings taken each day over a three-day period) before construction starts and monitored daily during installation of temporary protection systems, king piles and sheet piles, and additional monitoring carried out over a reasonable period of time after construction (one month) to check settlements / movements are within tolerable levels.

Regarding vibrations to the existing watermain during construction, a maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for buried utilities such as an HDPE watermain. However, due to the importance of this high pressure watermain supply line, vibration monitoring is recommended to be carried out at the ground surface above the watermain during installation of temporary protection systems, king piles and sheet piles, adopting a maximum PPV of 50mm/s.

A separate geotechnical / foundations instrumentation monitoring report combining the settlement and vibration monitoring plan for the watermain is to be provided when the locations of the watermain is confirmed.

As per the Terms of Reference (TOR) for this assignment, the Contractor shall provide the MTO at least 3 months in advance a plan for addressing monitoring of the existing watermain.

6.6.5 Analytical Testing for Construction Materials

The results of an analytical test carried out on a soil sample from Borehole HP-2 are presented in Table B1 in Appendix B. The suite of parameters tested is intended to allow the design engineer to assess the need for corrosion protection of steel elements in general accordance with the MTO Gravity Pipe Design Guidelines (MTO, 2014) and MTO Structural Manual.

It should be noted that the creek water level in the area is subject to seasonal fluctuations and variations due to precipitation events and the water chemistry could also be variable. These recommendations are provided as guidance only; the structural designer should take the results of the laboratory testing, the potential for corrosion and the ultimate selection of materials and design sacrificial steel thickness into consideration for the design life of the culvert structure.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. André Bom, P.Eng. and Mr. Kevin Bentley, P.Eng., a Designated MTO Foundations Contact for WSP Golder, conducted an independent quality control review of this report.

Signature Page

Golder Associates Ltd.



Andre Bom, P.Eng.
Geotechnical Engineer



Kevin Bentley, P.Eng.
Designated MTO Foundations Contact

AD/AB/KJB/ca/ar

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- Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition. BiTech Publications.
- Canadian Standards Association (CSA), 2014. Canadian Highway Bridge Design Code and Commentary on CAN/CSA S6-14.
- Ministry of Northern Development of Mines. Bedrock Geology of Ontario – East Central Sheet, Ontario Geological Survey – Map 2543.
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- Rocscience RSPile Version 3.008, 2021
- Terzaghi, K. and Peck, R.B. 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York.
- Unified Facilities Criteria, U.S. Navy. 1986. NAVFAC Design Manuals 7.01 and 7.02: Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.

ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils
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Ontario Provincial Standard Drawings (OPSD)

OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3121.150	Walls Retaining, Backfill Minimum Granular Requirement

Ontario Provincial Standard Specifications

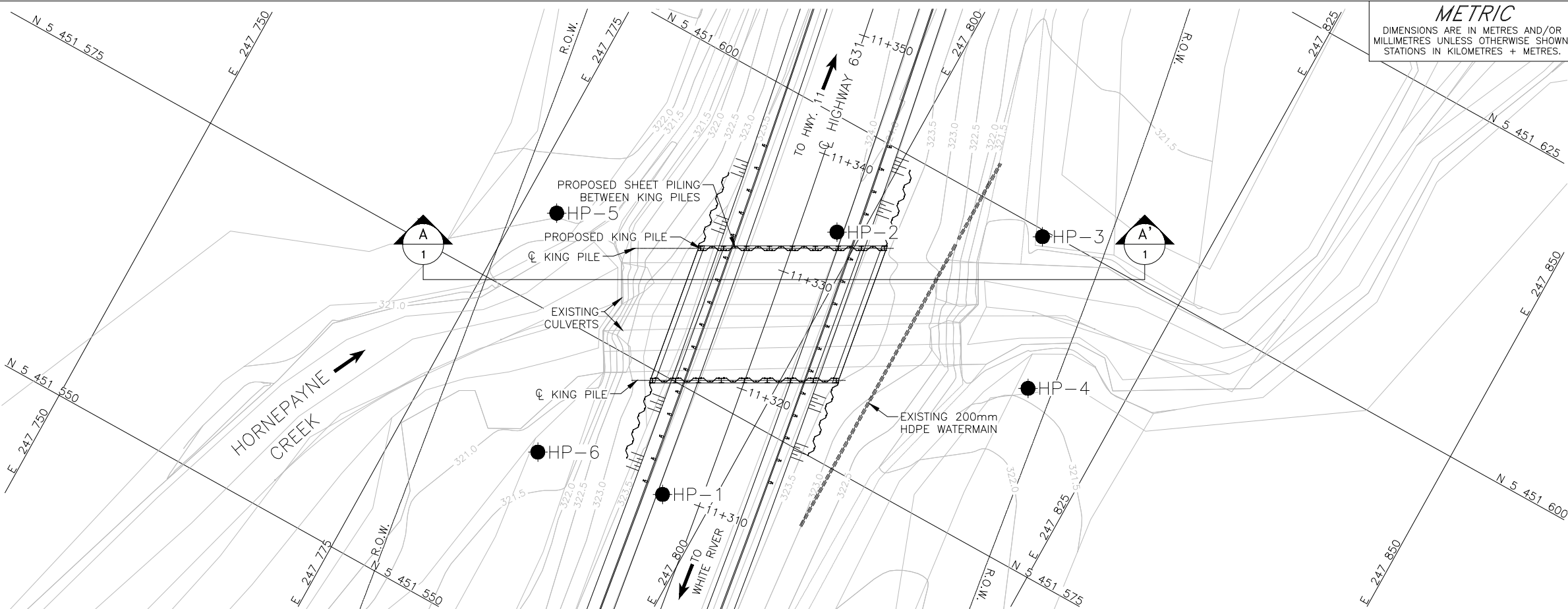
OPSS.PROV 206	Construction Specifications for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Dewatering
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
SP517F01	Temporary Flow Passage System

Ontario Water Resource Act

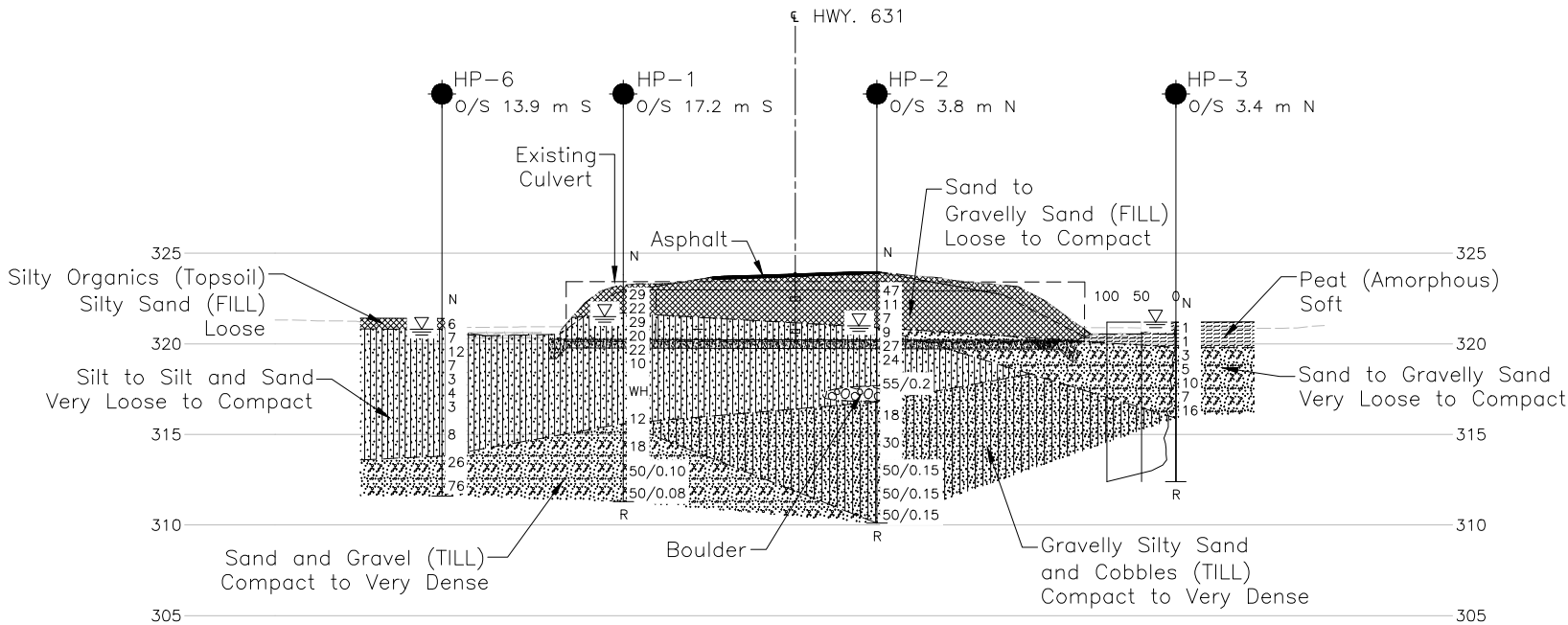
Ontario Regulation 903 Wells (as amended)

Ontario Occupational Health and Safety Act:

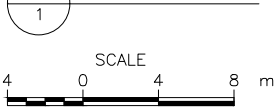
Ontario Regulation 213/91 Construction Projects (as amended)



PLAN



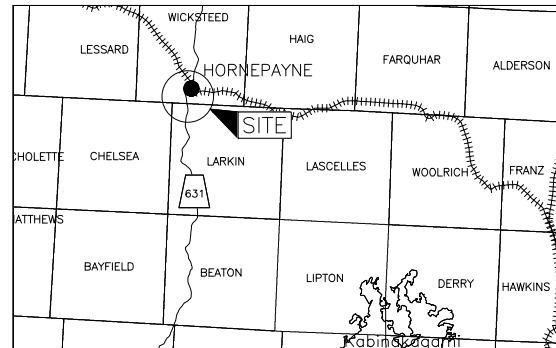
PROFILE



METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. DB 2021-5168
WP No.

HORNEPAYNE CREEK CULVERT REPLACEMENT
LAT. 49.199675, LONG. -84.782248
BOREHOLE LOCATIONS AND SOIL STRATA



KEY PLAN

SCALE



LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ≡ WL upon completion of drilling
- R Refusal

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 13)

No.	ELEVATION	NORTHING	EASTING
HP-1	323.8	5451566.6	247796.2
HP-2	324.0	5451591.8	247798.2
HP-3	321.2	5451599.5	247812.8
HP-4	321.2	5451588.3	247817.7
HP-5	321.7	5451582.2	247777.8
HP-6	321.4	5451564.7	247785.8



NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

- General arrangement provided in digital format by WSP, drawing file no. S221-09193-00-304-001GA.dwg, Received MARCH 7, 2023.
- Base plan provided in digital format by WSP, drawing file no. x17197 Hornepayne Base.dwg, Received DECEMBER 12, 2022.
- Alignment provided in digital format by WSP, drawing file no. x17197 Hornepayne ALI.dwg, Received DECEMBER 12, 2022.

NO.	DATE	BY	REVISION
1	6/14/2023	KJB	1
Geocres No. 42F-063			
HWY. 631	PROJECT NO. 22525553	DIST.	
SUBM'D. AB	CHKD.	DATE: 6/14/2023	SITE: 38N-0004/CO
DRAWN: TR	CHKD. AB	APPD. KJB	DWG. 1

**Photograph 1: Hornepayne Creek Culvert
East (Outlet) End (June 2017)**



**Photograph 2: Hornepayne Creek Culvert
West (Inlet) End (June 2017)**



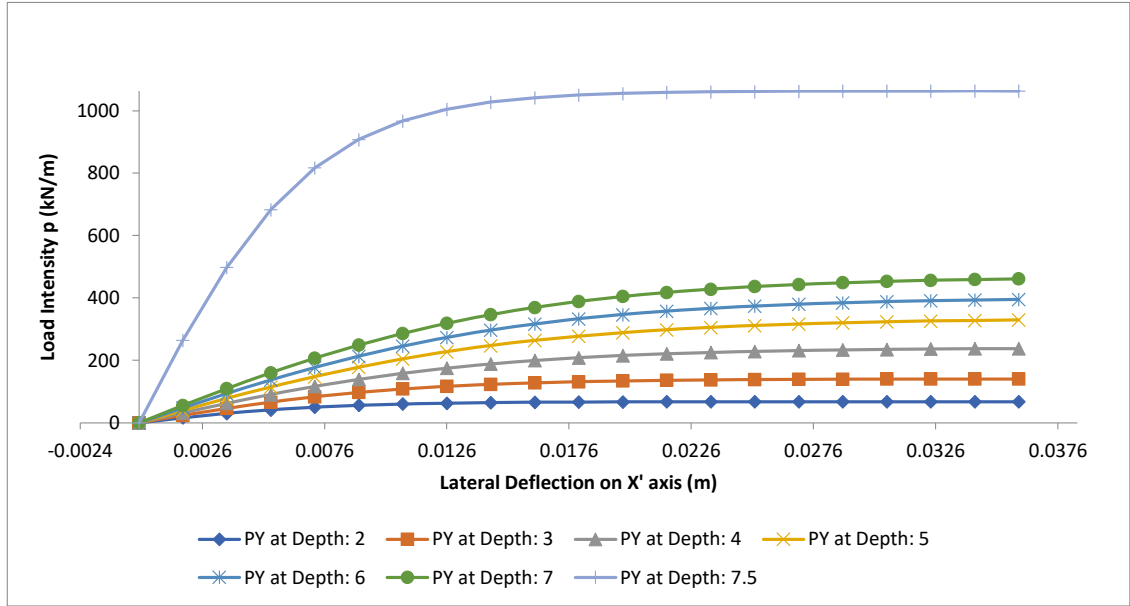
**Photograph 3: Hornepayne Creek Culvert
Looking Southwest Upstream (June 2017)**



**Photograph 4: Hornepayne Creek Culvert
Looking South Along Road from North of Culvert (June 2017)**



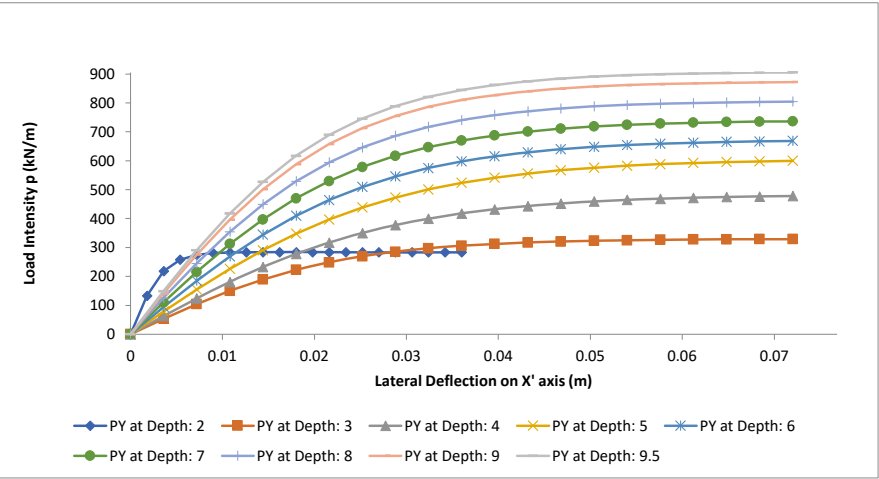
Figure 1 Model 1 - Deflection Towards Creek Bed (Single 360x132 HPIle)



PY at Depth: 2	Load Intensity p (kN/m)	PY at Depth: 3	Load Intensity p (kN/m)	PY at Depth: 4	Load Intensity p (kN/m)	PY at Depth: 5	Load Intensity p (kN/m)	PY at Depth: 6	Load Intensity p (kN/m)	PY at Depth: 7	Load Intensity p (kN/m)	PY at Depth: 7.5	Load Intensity p (kN/m)
0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.0018	15.55363087	0.0018	23.53652917	0.0018	31.49770073	0.0018	39.41713869	0.0018	47.30040876	0.0018	55.18366979	0.0018	264.344
0.0036	29.52832325	0.0036	45.78901875	0.0036	61.9318269	0.0036	77.75902503	0.0036	93.31065381	0.0036	108.8659375	0.0036	497.926
0.0054	40.92816371	0.0054	65.74053958	0.0054	90.37828154	0.0054	114.0666319	0.0054	136.8815016	0.0054	159.6964688	0.0054	682.806
0.0072	49.51610323	0.0072	82.79407604	0.0072	116.1520647	0.0072	147.578707	0.0072	177.0938582	0.0072	206.6091354	0.0072	816.8
0.009	55.60678415	0.009	96.78779375	0.009	138.8588217	0.009	177.794025	0.009	213.3541189	0.009	248.9143646	0.009	907.83
0.0108	59.74368196	0.0108	107.8923229	0.0108	158.3617727	0.0108	204.462586	0.0108	245.3569343	0.0108	286.2525833	0.0108	966.988
0.0126	62.47133368	0.0126	116.4643646	0.0126	174.7740146	0.0126	227.5710115	0.0126	273.0823045	0.0126	318.5933646	0.0126	1004.32
0.0144	64.23580709	0.0144	122.9523125	0.0144	188.3286444	0.0144	247.2549322	0.0144	296.7055787	0.0144	346.1571354	0.0144	1027.52
0.0162	65.36186131	0.0162	127.7761667	0.0162	199.3580501	0.0162	263.7998436	0.0162	316.556757	0.0162	369.3234688	0.0162	1041.68
0.018	66.07548384	0.018	131.33325	0.018	208.2291345	0.018	277.5491241	0.018	333.0511887	0.018	388.5623646	0.018	1050.32
0.0198	66.52451408	0.0198	133.9289167	0.0198	215.2880918	0.0198	288.8538999	0.0198	346.6235245	0.0198	404.3929688	0.0198	1055.56
0.0216	66.80717414	0.0216	135.8085208	0.0216	220.8656309	0.0216	298.074171	0.0216	357.6898123	0.0216	417.3057083	0.0216	1058.68
0.0234	66.98438373	0.0234	137.1693854	0.0234	225.2479458	0.0234	305.5599374	0.0234	366.6700521	0.0234	427.7797292	0.0234	1060.6
0.0252	67.09490824	0.0252	138.1422188	0.0252	228.6688425	0.0252	311.5934515	0.0252	373.9088947	0.0252	436.2246042	0.0252	1061.7
0.027	67.16397602	0.027	138.8443438	0.027	231.3283212	0.027	316.4335871	0.027	379.7223879	0.027	443.0107604	0.027	1062.4
0.0288	67.20781543	0.0288	139.3457604	0.0288	233.3970907	0.0288	320.3125965	0.0288	384.3744838	0.0288	448.4373438	0.0288	1062.82
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0.0324	67.25180918	0.0324	139.9605625	0.0324	236.2310845	0.0324	325.8738582	0.0324	391.0433264	0.0324	456.2130729	0.0324	1063.22
0.0342	67.26219187	0.0342	140.1485938	0.0342	237.1908238	0.0342	327.8283629	0.0342	393.3947237	0.0342	458.9613646	0.0342	1063.32
0.036	67.26888321	0.036	140.276625	0.036	237.922951	0.036	329.3817414	0.036	395.2607716	0.036	461.1300833	0.036	1063.4

Figure 2 Model 2 - Deflection Away from Creek Bed (Single 360x132 HPile)

Project # 22525553
May, 2023



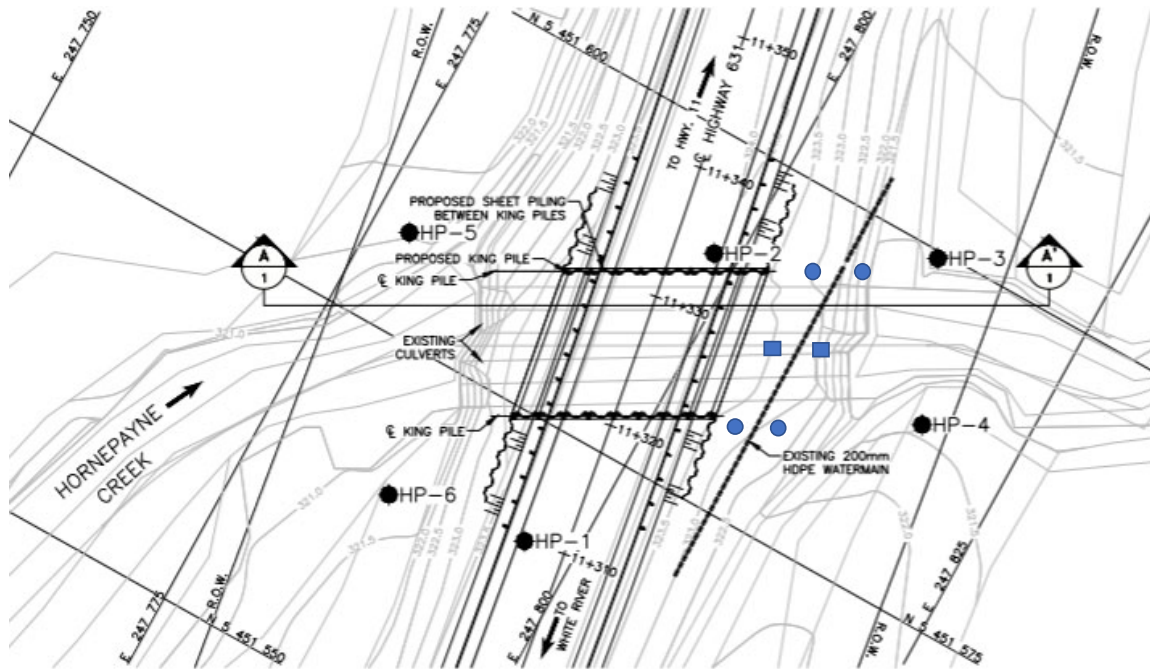
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0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.0018	132.8092593	0.0036	53.5233958	0.0036	62.99846856	0.0036	78.75203651	0.0036	94.41384077	0.0036	110.0580122	0.0036	125.6898986	0.0036	141.3057447	0.0036	149.1147667
0.0036	217.9720635	0.0072	104.3017691	0.0072	123.8949696	0.0072	154.8880527	0.0072	185.1833671	0.0072	215.3582556	0.0072	245.4303245	0.0072	275.4271733	0.0072	290.4060142
0.0054	258.1766138	0.0108	150.1325937	0.0108	180.844503	0.0108	226.1420284	0.0108	269.2187728	0.0108	311.9342394	0.0108	354.3616734	0.0108	396.5600912	0.0108	417.5938844
0.0072	274.4192593	0.0144	189.6583658	0.0144	232.4991176	0.0144	290.8033469	0.0144	344.3329006	0.0144	397.1251116	0.0144	449.3574544	0.0144	501.1543769	0.0144	526.9259838
0.009	280.5649735	0.018	222.454048	0.018	278.0658621	0.018	347.8872211	0.018	409.3919473	0.018	469.7356592	0.018	529.2110446	0.018	588.0200304	0.018	617.2310446
0.0108	282.8328042	0.0216	248.7796402	0.0216	317.2637525	0.0216	397.0484381	0.0216	464.2373225	0.0216	529.8363083	0.0216	594.2789351	0.0216	657.8591489	0.0216	689.4027282
0.0126	283.6605291	0.0252	269.3701349	0.0252	350.2798377	0.0252	438.5093915	0.0252	509.4166329	0.0252	578.3783367	0.0252	645.9776166	0.0252	712.5698784	0.0252	745.5759635
0.0144	283.967619	0.0288	285.1455472	0.0288	377.6031339	0.0288	472.8468357	0.0288	545.9226978	0.0288	616.8221704	0.0288	686.2202028	0.0288	754.5462918	0.0288	788.4044118
0.0162	284.076455	0.0324	297.0458771	0.0324	399.8656085	0.0324	500.8631643	0.0324	574.9807302	0.0324	646.7990872	0.0324	717.0598073	0.0324	786.2504863	0.0324	820.5442698
0.018	284.115873	0.036	305.9111394	0.036	417.8033266	0.036	523.4603448	0.036	597.8283367	0.036	669.8795132	0.036	740.4196755	0.036	809.9145593	0.036	844.3704665
0.0198	284.130582	0.0396	312.4613493	0.0396	432.1012069	0.0396	541.4931643	0.0396	615.6207302	0.0396	687.4943002	0.0396	757.9330527	0.0396	827.4145593	0.0396	861.8904665
0.0216	284.130582	0.0432	317.2665067	0.0432	443.4172819	0.0432	555.7788032	0.0432	629.361714	0.0432	700.8334483	0.0432	770.9898073	0.0432	840.276535	0.0432	874.6904665
0.0234	284.140582	0.0468	320.7766267	0.0468	452.3135193	0.0468	567.0296552	0.0468	639.9226978	0.0468	710.8825963	0.0468	780.6633164	0.0468	849.6804863	0.0468	883.993002
0.0252	284.140582	0.0504	323.3267166	0.0504	459.2728702	0.0504	575.8405071	0.0504	647.9998783	0.0504	718.4265314	0.0504	787.810071	0.0504	856.5364134	0.0504	890.7268053
0.027	284.140582	0.054	325.1817841	0.054	464.6953347	0.054	582.711359	0.054	654.1570588	0.054	724.0656795	0.054	793.0735801	0.054	861.5203647	0.054	895.5893408
0.0288	284.140582	0.0576	326.5268366	0.0576	468.919929	0.0576	588.062211	0.0576	658.8404361	0.0576	728.2748276	0.0576	796.933712	0.0576	865.1262918	0.0576	899.1018763
0.0306	284.140582	0.0612	327.4968816	0.0612	472.1905882	0.0612	592.2154564	0.0612	662.3952231	0.0612	731.4091886	0.0612	799.7704665	0.0612	867.7482675	0.0612	901.633144
0.0324	284.140582	0.0648	328.1969115	0.0648	474.7182961	0.0648	595.4387018	0.0648	665.0800101	0.0648	733.7435497	0.0648	801.8505984	0.0648	869.6422188	0.0648	903.4544118
0.0342	284.140582	0.0684	328.7069265	0.0684	476.672069	0.0684	597.9267343	0.0684	667.1186004	0.0684	735.4779108	0.0684	803.3739757	0.0684	871.0061702	0.0684	904.7656795
0.036	284.140582	0.072	329.071949	0.072	478.1828905	0.072	599.8547667	0.072	668.6571907	0.072	736.7574848	0.072	804.4873529	0.072	871.9961702	0.072	905.698215

Figure 3

Settlement Monitoring Point Locations



Project # 22525553
May, 2023



- Monitoring Points to be installed on subgrade about 3 m from existing culvert, about 1 m from watermain
- Monitoring Points to be installed on subgrade, about 1 m from watermain, if required

APPENDIX A

Record of Boreholes

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse	19 to 75	0.75 to 3
	Fine	4.75 to 19	(4) to 0.75
SAND	Coarse	2.00 to 4.75	(10) to (4)
	Medium	0.425 to 2.00	(40) to (10)
	Fine	0.075 to 0.425	(200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (<i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (<i>i.e.</i> , some sand)
≤ 10	trace (<i>i.e.</i> , trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (*q_t*), porewater pressure (*u*) and sleeve friction (*f_s*) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, w _L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
Y	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

COARSE-GRAINED SOILS

Compactness¹

Term	SPT 'N' (blows/0.3m) ²
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

- Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
- SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
- SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
$\ln x$	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_L or LL	liquid limit
w_P or PL	plastic limit
I_P or PI	plasticity index $= (w_L - w_P)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index $= (w - w_P) / I_P$
I_c	consistency index $= (w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
$C_{a(e)}$	secondary compression index
C_a	rate of secondary compression
$C_{a(e)}$	modified secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
c'	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q or q'	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

* Density symbol is ρ . Unit weight symbol is γ .
where $\gamma = \rho \cdot g$ (i.e., mass density multiplied by
acceleration due to gravity)

Notes: 1
2

$\tau = c' + \sigma' \tan \phi'$
shear strength = (compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING CLASSIFICATION

Fresh (W1): no visible sign of rock material weathering.

Slightly Weathered (W2): discoloration indicates weathering of rock mass material on discontinuity surfaces. **Less than 5%** of rock mass is altered or weathered.

Moderately Weathered (W3): less than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Highly Weathered (W4): more than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Completely Weathered (W5): 100% of the rock mass is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

Residual Soil (W6): all rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

AXJ Axial Joint	KV Karstic Void
BD Bedding	K Slickensided
BC Broken Core	LC Lost Core
CC Continuous Core	MB Mechanical Break
CL Closed	PL Planar
CO Contact	PO Polished
CU Curved	RO Rough
CT Coated	SA Slightly Altered
FLT Fault	SH Shear
FOL Foliation	SM Smooth
FR Fracture	SR Slightly Rough
GO Gouge	SY Stylolite
IN Infilled	UN Undulating
IR Irregular	VN Vein
JN Joint	VR Very Rough

ISRM Intact Rock Material Strength Classification

Grade	Description	Approx. Range of Uniaxial Compressive Strength (MPa)
R0	Extremely weak rock	0.25 – 1.0
R1	Very weak rock	1.0 – 5.0
R2	Weak rock	5.0 – 25
R3	Medium strong rock	25 – 50
R4	Strong rock	50 -100
R5	Very strong rock	100 -250
R6	Extremely strong rock	>250


PROJECT 1661607		RECORD OF BOREHOLE No HP-1				1 OF 2 METRIC						
W.P. 5165-13-01		LOCATION N 5451566.6; E 247796.2 MTM ZONE 13 (LAT. 49.199525; LONG. -84.782251)				ORIGINATED BY SA						
DIST HWY 631		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring				COMPILED BY AD						
DATUM GEODETIC		DATE June 7, 2017				CHECKED BY AB						
SOIL PROFILE			SAMPLES			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	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L			
323.8	GROUND SURFACE											
323.4	ASPHALT (60 mm) Gravelly sand (FILL)											
323.4	Sand, trace gravel, trace silt, trace organics (FILL) Compact Brown Moist Rootlets encountered in Sample 2.		1	SS	29		323					0 92 (8)
321.7			2	SS	22		322					
321.7	SILT and SAND, trace to some clay, trace organics in Samples 3 to 6 Loose to compact Brown to grey Wet		3	SS	29		321					
			4	SS	20		320					2 32 62 4
			5	SS	22		319					
			6	SS	10		318					0 2 92 6
318.2	SILT, trace sand, trace to some clay Very loose to compact Grey Wet		7	SS	WH		317					
			8	SS	12		316					
315.6	SAND to SAND and GRAVEL, trace to some silt, some cobbles, trace clay (TILL) Compact to very dense Grey Wet		9	SS	18		315					
			10	SS	50/0.10		314					
							313					2 86 9 3
							312					

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Continued Next Page

+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE



PROJECT <u>1661607</u>		RECORD OF BOREHOLE No HP-1		2 OF 2 METRIC	
W.P. <u>5165-13-01</u>		LOCATION <u>N 5451566.6; E 247796.2 MTM ZONE 13 (LAT. 49.199525; LONG. -84.782251)</u>		ORIGINATED BY <u>SA</u>	
DIST <u> </u> HWY <u>631</u>		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring</u>		COMPILED BY <u>AD</u>	
DATUM <u>GEODETIC</u>		DATE <u>June 7, 2017</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100	20	40	60					
311.3			11	SS	50/0.08															
12.5	END OF BOREHOLE SPLIT-SPOON REFUSAL Note: 1. Water level at a depth of 2.3 m below ground surface (Elev. 321.5 m) upon completion of drilling.																			

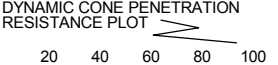


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+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

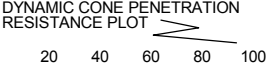
+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT 1661607		RECORD OF BOREHOLE No HP-3				1 OF 1 METRIC								
W.P. 5165-13-01		LOCATION N 5451599.5; E 247812.8 MTM ZONE 13 (LAT. 49.199822; LONG. -84.782020)				ORIGINATED BY MR								
DIST _____ HWY 631		BOREHOLE TYPE Portable Drill, NW Casing and Wash Boring				COMPILED BY AD								
DATUM GEODETIC		DATE June 8, 2017				CHECKED BY AB								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
321.2 0.0	GROUND SURFACE PEAT (Amorphous), trace sand, trace wood Very soft Black Wet		1	SS	1		321							
			2	SS	1		320							
319.8 1.4	SAND to Gravelly SAND, trace to some clay Very loose to compact Grey Wet		3	SS	3		319							
			4	SS	5		318							
			5	SS	10		317							
			6	SS	7		316							
			7	SS	16		315							
315.9 5.3	END OF BOREHOLE REFUSAL TO FURTHER CASING ADVANCEMENT						314							
312.4 8.8	END OF DCPT REFUSAL TO FURTHER PENETRATION Notes: 1. Water level at ground surface (Elev. 321.2 m) upon completion of drilling. 2. Split Spoon samples obtained by driving with a 1/2 weight hammer. SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer. 3. Advanced DCPT 0.6 m west of Borehole.						313							









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PROJECT 1661607			RECORD OF BOREHOLE No HP-4				1 OF 1 METRIC				
W.P. 5165-13-01		LOCATION N 5451588.3; E 247817.7 MTM ZONE 13 (LAT. 49.199722; LONG. -84.781959)				ORIGINATED BY MR					
DIST _____ HWY 631		BOREHOLE TYPE Portable Drill, NW Casing and Wash Boring				COMPILED BY AD					
DATUM GEODETIC		DATE June 9, 2017				CHECKED BY AB					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
321.2	GROUND SURFACE										
0.0	PEAT (Amorphous), trace sand Soft Black Wet		1	SS	3		321				
320.4	SAND, some gravel, trace silt, trace clay, trace organics in Samples 2 and 3 Very loose to compact Grey Wet		2	SS	1		320				9 88 (3)
0.8			3	SS	1		319				
			4	SS	2		318				17 72 7 4
			5	SS	7		317				
			6	SS	16		316				
			7	SS	21						17 75 (8)
315.9	END OF BOREHOLE REFUSAL TO FURTHER CASING ADVANCEMENT										
315.7	END OF DCPT REFUSAL TO FURTHER PENETRATION										
5.5	Notes: 1. Water level at ground surface (Elev. 321.2 m) upon completion of drilling. 2. Split Spoon samples obtained by driving with a 1/2 weight hammer. SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer. 3. Advanced DCPT 0.8 m west of Borehole.										

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTO\1661607 LEA_5015-E-0049_NE REGION02_DATA\GINT\1661607.GPJ GAL-MISS.GDT 11/3/17 TBAJL

PROJECT 1661607			RECORD OF BOREHOLE No HP-5				1 OF 1 METRIC				
W.P. 5165-13-01			LOCATION N 5451582.2; E 247777.8 MTM ZONE 13 (LAT. 49.199664; LONG. -84.782506)				ORIGINATED BY MR				
DIST _____ HWY 631			BOREHOLE TYPE Portable Drill, NW Casing and Wash Boring				COMPILED BY AD				
DATUM GEODETIC			DATE June 7, 2017				CHECKED BY AB				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
321.7	GROUND SURFACE										
0.0	Peat (Amorphous) (FILL) Black Wet		1	SS	3	▽					
0.1	Sand, trace to some gravel, trace organics (FILL) Very loose to compact Brown Wet		2	SS	12		321				
320.6	SILT, trace sand, trace gravel, trace rootlets Loose to compact Grey Wet		3	SS	11		320				0 3 90 7
1.1			4	SS	8		319				
			5	SS	6		318				0 6 87 7
			6	SS	5		317				
317.1	SILT and SAND, trace gravel, trace clay Dense Grey Wet		7	SS	31		316				5 43 48 4
316.5	END OF BOREHOLE REFUSAL TO FURTHER CASING ADVANCEMENT										
5.2											
314.6	END OF DCPT REFUSAL TO FURTHER PENETRATION										
7.1	Notes: 1. Water level at a depth of 0.5 m below ground surface (Elev. 321.2 m) upon completion of drilling. 2. Split Spoon samples obtained by driving with a 1/2 weight hammer. SPT "N" values have been adjusted to the inferred values that would be obtained using a standard weight hammer. 3. Advanced DCPT 0.8 m east of Borehole.										

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTM\1661607 LEA_5015-E-0049_NE REGION02_DATA\GINT\1661607.GPJ GAL-MISS.GDT 11/3/17 TBAJL

PROJECT 1661607			RECORD OF BOREHOLE No HP-6				1 OF 1 METRIC									
W.P. 5165-13-01		LOCATION N 5451564.7; E 247785.8 MTM ZONE 13 (LAT. 49.199507; LONG. -84.782393)				ORIGINATED BY MR										
DIST _____ HWY 631		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring				COMPILED BY AD										
DATUM GEODETIC		DATE June 9, 2017				CHECKED BY AB										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
321.4	GROUND SURFACE															
0.0	Silty topsoil (FILL)		1	SS	6		321								NP	0 1 90 9
0.1	Silty sand, trace organics (FILL)															
320.8	Loose Grey Wet															
0.6	SILT, trace sand, trace rootlets in Samples 2 to 4		2	SS	7		320									
	Very loose to compact Grey Wet															
			3	SS	12		319									
		4	SS	7	318											
		5	SS	3	317											
		6	SS	4	316											
		7	SS	3	315											
315.3	SAND, trace gravel		8	SS	8		315							47 38 13 2		
6.1	Loose Grey Wet															
313.8	SAND and GRAVEL, some silt, trace clay (TILL)		9	SS	26		314									
7.6	Compact to very dense Grey Wet															
		10	SS	76	313											
311.6	END OF BOREHOLE						312									
9.8	Note: 1. Water level at a depth of 0.6 m below ground surface (Elev. 320.8 m) upon completion of drilling.															

APPENDIX B

Laboratory Test Results

Table B1 - Summary of Analytical Testing of Soil Sample

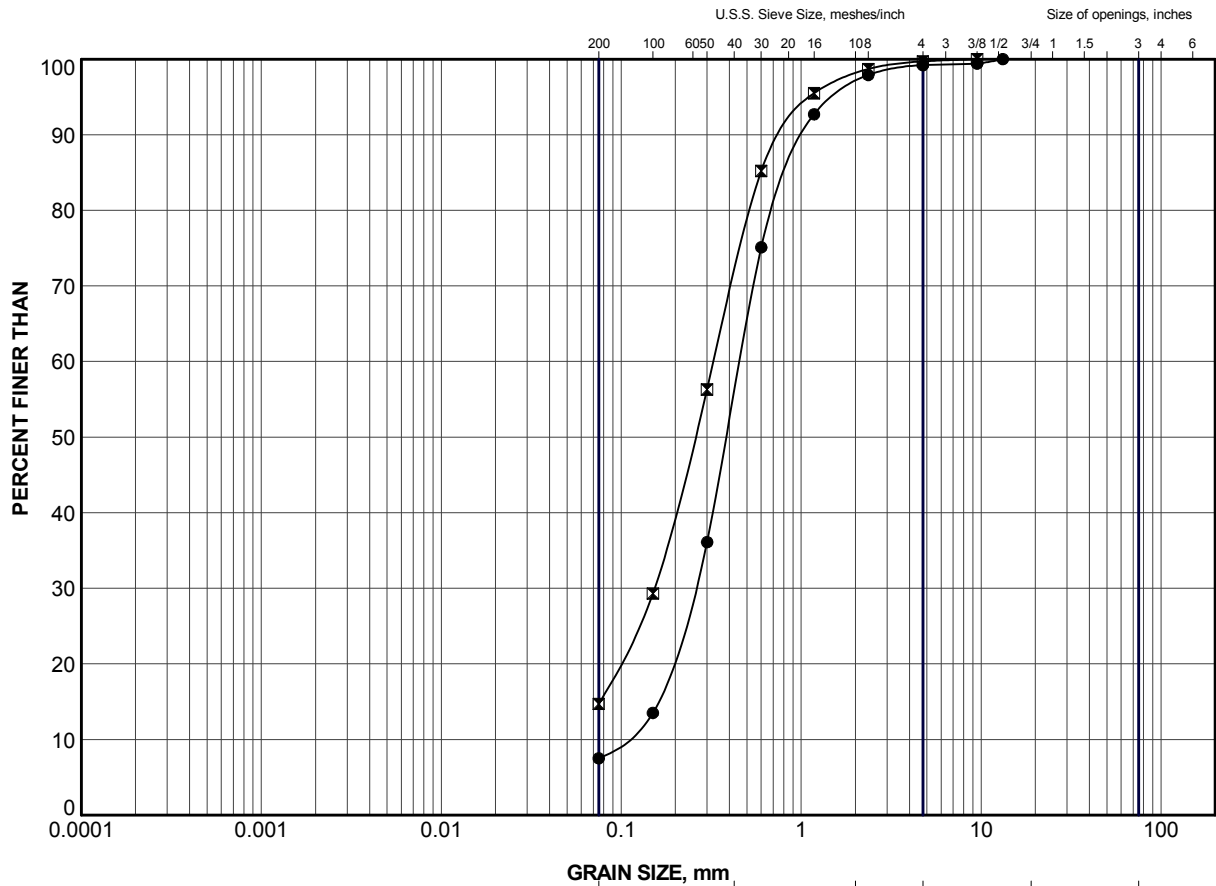
Parameter	Units	Borehole HP-2
Resistivity	ohm-cm	9800
Conductivity	µmho/cm	102
pH	pH	7.85
Sulphate	µg/g	Not Detected
Chloride	µg/g	Not Detected

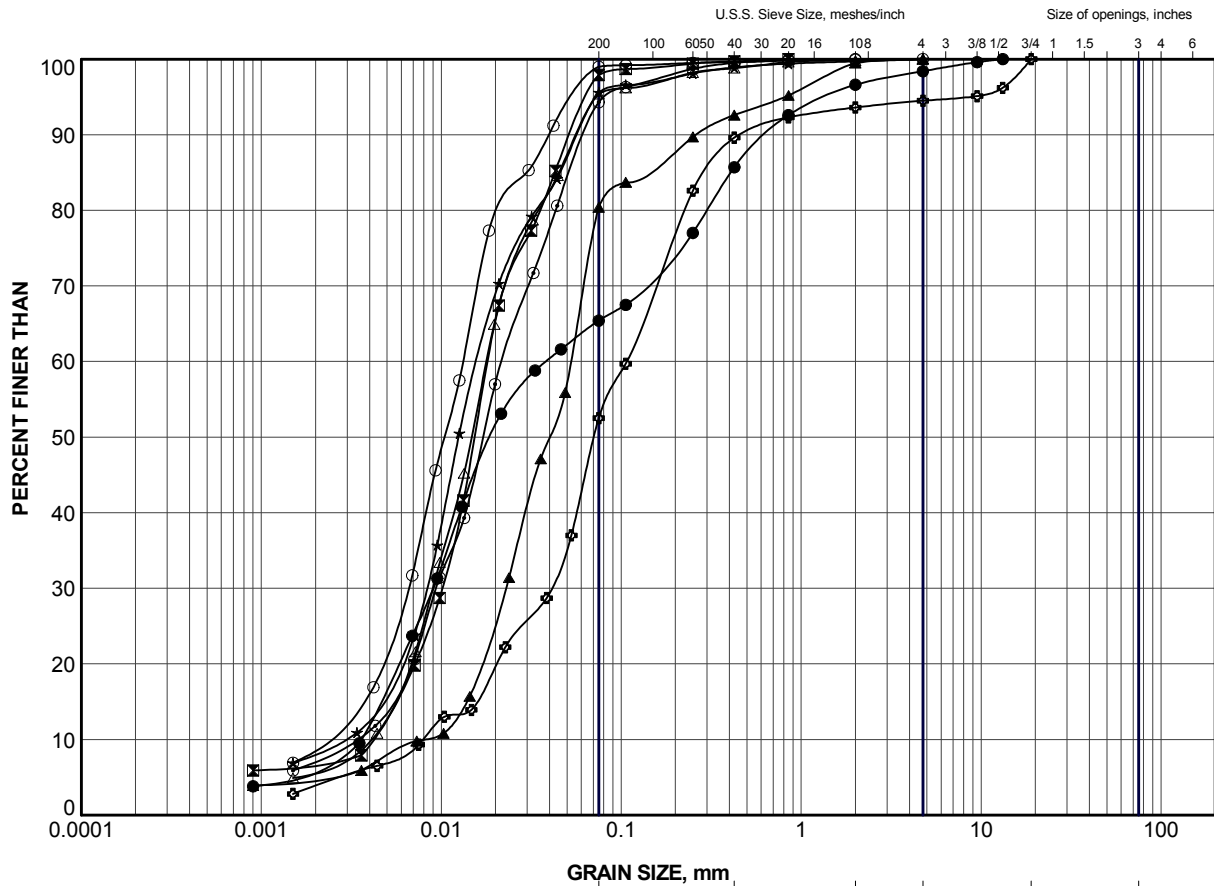
Notes:

1. Sample obtained 8 June 2017.

Prepared by: AD

Reviewed by: AB





CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	HP-1	4	320.4
⊠	HP-1	7	317.4
▲	HP-2	6	319.1
★	HP-5	3	319.9
⊙	HP-5	5	318.3
⊕	HP-5	7	316.8
○	HP-6	3	319.6
△	HP-6	6	317.3

PROJECT

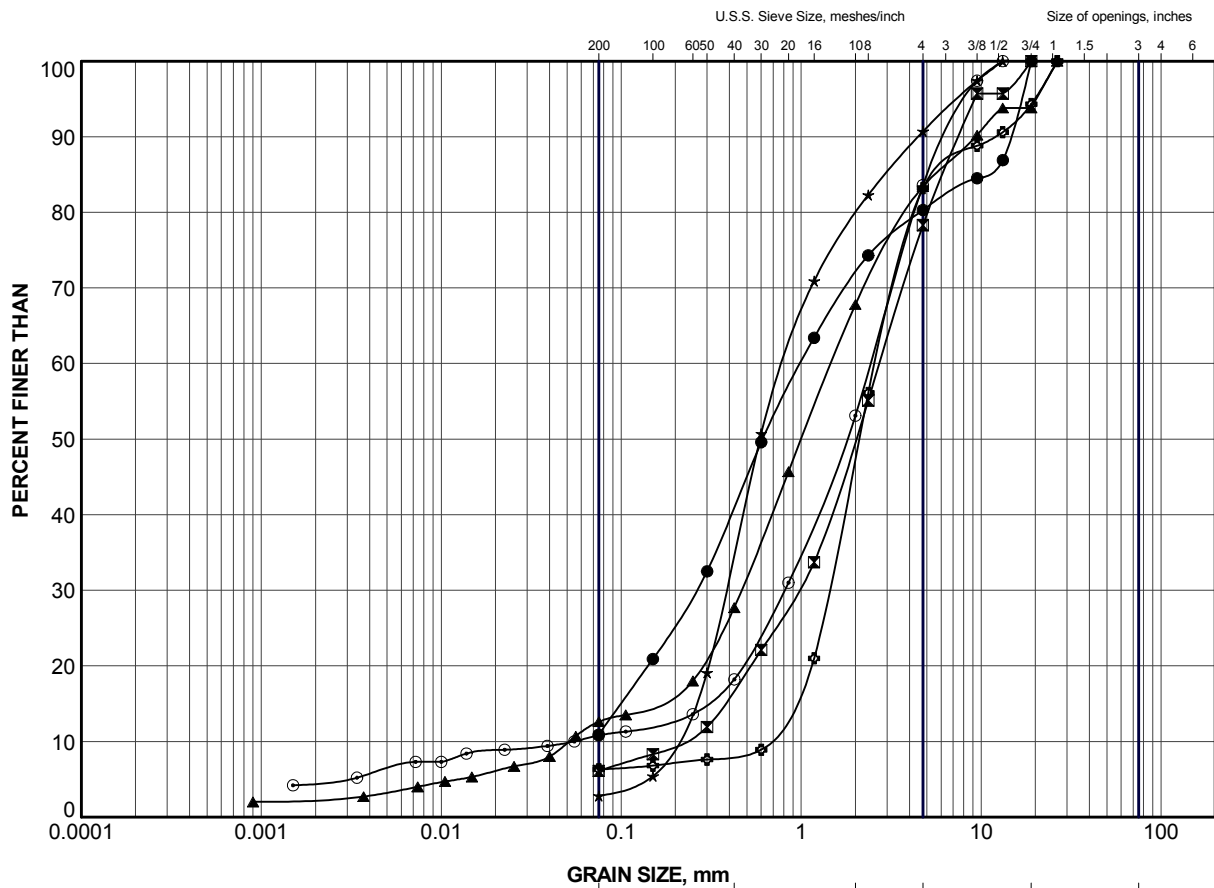
HIGHWAY 631
HORNEPAYNE CREEK CULVERT

TITLE

GRAIN SIZE DISTRIBUTION
SILT to SILT and SAND




PROJECT No. 1661607			FILE No. 1661607.GPJ		
DRAWN	TB	Nov 2017	SCALE	N/A	REV.
CHECK	AB	Nov 2017	FIGURE B2		
APPR	JMAC	Nov 2017			

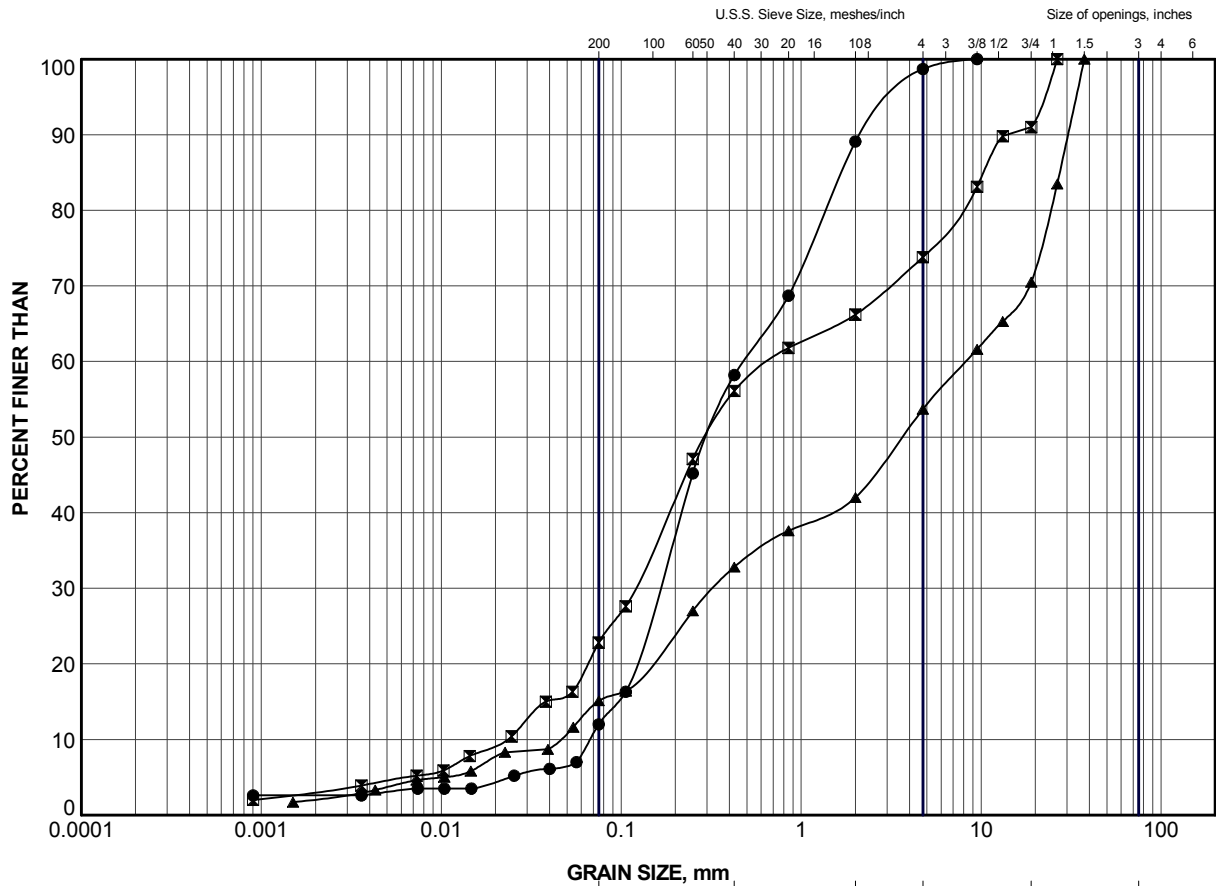


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	HP-2	4	320.6
⊠	HP-3	4	318.6
▲	HP-3	6	317.1
★	HP-4	2	320.1
⊙	HP-4	4	318.6
⊕	HP-4	7	316.3

PROJECT					
HIGHWAY 631 HORNEPAYNE CREEK CULVERT					
TITLE					
GRAIN SIZE DISTRIBUTION SAND to GRAVELLY SAND					
PROJECT No.		1661607		FILE No. 1661607.GPJ	
DRAWN	TB	Nov 2017	SCALE	N/A	REV.
CHECK	AB	Nov 2017			
APPR	JMAC	Nov 2017			
 Golder Associates SUDBURY, ONTARIO			FIGURE B3		



CLAY AND SILT	SAND SIZE			GRAVEL SIZE		Cobble Size
	fine	medium	coarse	fine	coarse	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	HP-1	10	313.0
■	HP-2	11	311.5
▲	HP-6	9	313.5

PROJECT					
HIGHWAY 631 HORNEPAYNE CREEK CULVERT					
TITLE					
GRAIN SIZE DISTRIBUTION SAND to GRAVELLY SILTY SAND to SAND and GRAVEL (TILL)					
PROJECT No.		1661607		FILE No. 1661607.GPJ	
DRAWN	TB	Nov 2017	SCALE	N/A	REV.
CHECK	AB	Nov 2017			
APPR	JMAC	Nov 2017			
			FIGURE B4		



APPENDIX C

Non Standard Special Provisions

AMENDMENT TO OPSS 903, APRIL 2016

Special Provision No. DBSP0903

OPSS 903, April 2016, is deleted in its entirety and replaced with the following:

CONSTRUCTION SPECIFICATION FOR DEEP FOUNDATIONS

903.01 SCOPE

This specification covers the requirements for the supply and installation of deep foundation units.

903.02 REFERENCES

This specification refers to the following specifications, standards, or publications:

Design-Build Special Provisions

DBSP 0904	Concrete Structures
DBSP 0909	Prestressed Concrete - Precast Girders
DBSP 1350	Concrete – Materials and Production

Ontario Provincial Standard Specifications, Construction

OPSS 904	Concrete Structures
OPSS 905	Steel Reinforcement for Concrete
OPSS 911	Coating Structural Steel Systems

Ontario Provincial Standard Specifications, Material

OPSS 1302	Water
OPSS 1440	Steel Reinforcement for Concrete
OPSS 1350	Concrete – Materials and Production

CSA Standards

G40.20/40.21-13 (R2018)	General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Steel
CAN3-056-1962 (R2006)	Round Timber Piles
O80 Series-08	Wood Preservation
W47.1-09 (R2014)	Certification of Companies for Fusion Welding of Steel
W48-18	Filler Metals and Allied Materials for Metal Arc Welding
W59-18	Welded Steel Construction (Metal Arc Welding)
W178.1-18	Certification of Welding Inspection Organizations
W178.2-18	Certification of Welding Inspectors

Canadian General Standards Board (CGSB)

48.9712-2006	Non-destructive Testing, Qualification and Certification of Personnel
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ASTM International

A 252-98(2007)	Welded and Seamless Steel Pipe Piles
A 328/A 328M-07	Steel Sheet Piling
D 1143/ D 1143M-07	Standard Test Methods for Deep Foundations under Static Axial Compressive Load
D 3689-07	Standard Test Methods for Deep Foundations under Static Axial Tensile Load
D 3966-07	Standard Test Method for Deep Foundations under Lateral Loads

American Petroleum Institute (API)

API 13A	Drilling Fluid Materials, 17th Edition, 10.00.08
RP 13B-1	Standard Procedure for Field Testing Water Based Drilling Fluids, 4th Edition,

Joint Publications of the Society for Protective Coatings (SSPC) and National Association of Corrosion Engineers (NACE)

SSPC-SP6/NACE No. 3-2007	Commercial Blast Cleaning
SP10/NACE No.2	Near-White Blast Cleaning

International Organization for Standardization/International Electrotechnical Commission (ISO/IEC)

17025	General Requirements for the Competence of the Testing and Calibration Laboratories
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903.03 DEFINITIONS

For the purpose of this specification, the following definitions apply:

Anvil means the component of a diesel hammer that acts as an impact block for the ram

Bedrock means a natural solid bed of the hard, stable, cemented part of the earth's crust, igneous, metamorphic, or sedimentary in origin that may or may not be weathered.

Caisson Pile means a cast in place deep foundation unit with or without an enclosing liner formed by placing concrete in a bored or excavated hole.

Cap Block means a material placed on top of the helmet to cushion the blow of the hammer and to attenuate the peak impact energy without causing excessive loss of the impact energy.

Casing means open ended enclosing cylindrical steel tubing or pipe permanently installed in the ground. Casings are structurally required and can be used to stabilize and excavated hole.

Deep Foundation Unit means a structural member, driven or otherwise, installed in the ground to transfer the loads from a structure to soil or rock and derives supporting resistance from the surrounding soil or rock or from the soil or rock strata below its tip or a combination of both.

Design Engineer means the Engineer retained by the Contractor who has sealed and signed the Issued for Construction Drawings and/or Working Drawings required to complete all or part of the work specified in the contract.

Design Engineer's Designee means a foundations Engineer who under the direct supervision of the design Engineer performs monitoring of the deep foundation work specified in the Contract Documents or as required by the design Engineer.

Displacement Caisson Pile means a pile formed in the ground by driving a casing or liner with a concrete plug or an expendable metal plate attached to it and replacing the displaced soil with unreinforced or reinforced concrete.

Driven Pile means one of the following pile types: steel H, tube, or sheet piles; wooden pile; or precast reinforced concrete pile that has been installed by means of a pile driver.

Driving Shoe means reinforcement attached to the bottom of the pile and designed to protect the pile during driving or to penetrate a hard stratum.

Driving to a Set means driving the pile to the requirement that satisfies pile driving criteria correlated to a required pile resistance.

Follower means a removable extension that transmits the hammer blows to the head of the pile.

Helmet means a formed steel cap that fits over the top of a pile head to retain in position a resilient cap block.

Jetting means the use of a jet of water at high pressure directed into the ground below the pile tip to assist its penetration

Liner means open ended enclosing steel tubing or pipe temporarily installed in the ground to facilitate the construction of caisson piles

Pile means a relatively slender structural element that is installed, wholly or partly in the ground by driving, drilling, auguring, jetting, or other means.

Pile Cap means a footing, or some other structural component used to transfer the load to the piles as well as maintaining them in position.

Pile Cushion means a pad of resilient material placed between the helmet and the top of a precast reinforced concrete or wooden pile to minimize damage to the head during driving.

Pile Group means the piles supporting a pile cap.

Pumped Concrete means a method of transporting concrete through hose or pipe by means of positive and continuous pressure.

Ram means the moving or driving part of an air, steam, diesel, or drop pile hammer that delivers an impact blow to an anvil and to the pile.

Retapping means verifying that the specified resistance previously attained has been sustained by imparting appropriate hammer energy to the pile and monitoring pile penetration.

Rock Points means a specially designed steel tip fitted to piles to enable them to be driven into hard, sound sloped bedrock.

Sheet Pile means a pile that is designed to interlock with adjacent piles and form a continuous wall for the purpose of resisting mainly lateral forces and to reduce seepage.

Slurry means a drilling fluid, consisting of water mixed with one or more of various solids or polymers, used to maintain the stability of the side walls and bottom of an excavation.

Tremie means a hopper with a vertical pipe used for placing concrete under water. The foot of the pipe is always submerged in concrete except during commencement of concreting and the upper level of the concrete in the pipe is always above water level.

903.04 DESIGN AND SUBMISSION REQUIREMENTS

903.04.01 Design Requirements

903.04.01.01 Concrete

The Contractor is responsible for providing plastic concrete with suitable characteristics for installation. The concrete shall be flowable, non-segregating concrete that does not exhibit rapid slump loss.

903.04.02 Submission Requirements

903.04.02.01 General

All submissions shall bear the seal and signature of the design Engineer experienced in the field of deep foundations.

When welded field splices are used, the Contractor shall submit the welding procedures to the Contract Administrator for the purpose of quality assurance and documentation. The Canadian Welding Bureau shall have approved the welding procedures.

903.04.02.02 Preconstruction Survey

Prior to commencing the work, the Contractor shall submit to the Contract Administrator, a condition survey of property and structures that may be affected by the work. The survey shall include the locations and conditions of adjacent properties; buildings; underground structures; Utility services; and structures, such as walls abutting the site.

903.04.02.03 Materials

903.04.02.03.01 Mill Certificates

The Contractor shall submit to the Contract Administrator at the time of delivery 1 copy of the mill certificates, indicating that the steel meets the requirements for the appropriate standards for H-piles, tube piles, and sheet piles.

Where mill test certificates originate from a mill outside Canada or the United States of America the Contractor shall have the information on the mill certificates verified by testing by a Canadian laboratory. The laboratory shall be certified by an organization accredited by the Standards Council of Canada to comply to comply with the requirements of ISO/IEC 17025 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the

specified material requirements. The stamp shall include the appropriate material specification number, the date (i.e., yyyy-mm-dd), and the signature of an authorized officer of the Canadian testing laboratory.

903.04.02.03.02 Concrete

The Contractor shall submit a suitable, site-specific concrete mix design that meets the requirements of the hardened concrete to the Contract Administrator 14 Days prior to construction, for the purpose of quality assurance and documentation.

903.04.02.03.03 Slurry

The Contractor shall submit the following to the Contract Administrator 14 Days prior to construction, for the purpose of quality assurance and documentation:

- a) The type, source, and physical and chemical properties of the bentonite or polymer.
- b) The source of water.
- c) Method of mixing slurry.
- d) The water solids ratio and the mass and volumes of the constituent parts, including any chemical admixtures or physical treatment employed to produce slurry with the required physical properties.
- e) Details of procedure to be used for monitoring the quality of the slurry.
- f) A test report showing the properties of the slurry and certifying that the slurry meets the requirements of API RP 13B-1.
- g) Method of disposal of the slurry.

903.04.02.04 Installation

903.04.02.04.01 Driven Piles

The Contractor shall submit the following to the Contract Administrator 14 Days prior to construction, for the purpose of quality assurance and documentation:

- a) A schedule of work identifying time and sequence of activities. A Request to Proceed to allow for quality assurance oversight by the Contract Administrator.
- b) Type of equipment, anvil, helmet, and hammer details, including the hammer energy assumed by the Contractor, stated potential energy (rated energy) of the hammer, operating efficiency, and weight of ram.
- c) Working Drawings of precast concrete piles showing the pile dimensions, concrete strength, tendon arrangement, working stresses and arrangement of steel reinforcement, schedules, elongation calculations, method and sequence of casting, complete specifications and details of the prestressing steel, and lift anchors and lifting point locations.
- d) The method of maintaining the steel reinforcement cages in position, when steel reinforcement cages are used in tube piles.

- e) Procedure for monitoring pile installation.
- f) Details of the method of attaching proprietary driving shoes.
- g) When load testing is specified in the Contract Documents, details of the full-scale test, including site preparation and the details of the load application, components, equipment, testing apparatus, and method of monitoring.
- h) Information pertinent to establishing the resistance of a pile when the wave equation analysis method is used.

903.04.02.04.02 Caisson Piles

The Contractor shall submit the following to the Contract Administrator 14 Days prior to construction, for the purpose of quality assurance and documentation:

- a) A schedule of work identifying time and sequence of activities. A Request to Proceed to allow for quality assurance oversight by the Contract Administrator.
- b) Detailed procedures for caisson excavation in overburden and rock.
- c) Detailed procedures for casing and liner installation and for the withdrawal of the liner.
- d) Detailed procedures for slurry displacement method of excavation, including disposal of slurry upon completion.
- e) Detailed procedures for tremie concrete, including the size of tremie delivery pipe.
- f) Detailed procedure for placing concrete in the dry.
- g) Method of maintaining the steel reinforcement cages in position in the caisson.
- h) Details of filling the annular void around a casing.
- i) Details of procedure to be used for monitoring installation.
- j) When load testing is specified in the Contract Documents, details of the full-scale test, including site preparation, details of the load application, components, equipment, testing apparatus, and method of monitoring.

903.04.02.04.03 Displacement Caisson Piles

The Contractor shall submit the following to the Contract Administrator 14 Days prior to construction, for the purpose of quality assurance and documentation:

- a) A schedule of work identifying time and sequence of activities. A Request to Proceed to allow for quality assurance oversight by the Contract Administrator.

- b) Type of equipment, anvil, helmet, and hammer details, including the hammer energy assumed by the Contractor, stated potential energy (rated energy) of the hammer, operating efficiency, maximum stroke or drop, and weight of the ram.
- c) Details of procedures used for installation of displacement caisson piles, including detailed procedures for liner installation and withdrawal.
- d) Method of maintaining the steel reinforcement cages in position in the pile.
- e) Details of procedure to be used for monitoring pile installation.
- f) When load testing is specified in the Contract Documents, details of the full-scale test, including site preparation, and the details of the load application, components, equipment, testing apparatus, and method of monitoring.

903.04.02.04.04 Steel Reinforcement Cages

The Contractor shall submit Working Drawings showing the fabrication details of the steel reinforcement cages, including the lifting points and lifting lugs, to the Contract Administrator 14 Days prior to fabrication, for the purpose of quality assurance and documentation.

903.05 MATERIALS

903.05.01 Wooden Piles

Wooden piles shall be according to CAN3-056 and shall be clean and peeled. Treated piles shall be pressure treated with creosote according to CAN/CSA O80.

Wooden piles shall be provided with collars sufficiently strong to prevent splitting of the head of the wooden pile during driving.

903.05.02 Steel Piles

903.05.02.01 H-Piles

Steel H-piles shall be according to CAN/CSA G40.20/G40.21, Grade 350 W.

903.05.02.02 Tube Piles

Steel tube piles shall be according to ASTM A 252, minimum Grade 2.

903.05.02.03 Sheet Piles

Steel sheet piles shall be according to ASTM A 328M.

903.05.02.04 Straightness Tolerance for Steel Piles, Casings, and Liners

Steel piles, casings, and liners shall conform to a straightness tolerance of 1.5 mm maximum per metre of length.

Steel sheet piles shall be sufficiently straight to prevent binding in the interlock during driving.

903.05.03 Driving Shoes and Rock Points

Rock points and driving shoes shall be as specified in the Contract Documents.

Driving shoes shall transfer the driving stresses to the pile over the full cross-sectional area of the pile.

Where precast concrete piles are driven into dense or hard material, a steel driving shoe cast into the concrete shall be provided.

Where wooden piles are driven into dense material, a steel plate driving shoe shall be provided to prevent damage to the bottom of the pile.

903.05.04 Casing for Caissons

Casings shall be according to ASTM A 252, Grade 2. If welded, they shall be welded by the electric arc method according to CSA W59.

The casing wall thickness specified is the minimum that shall be supplied. The wall thickness shall be increased as required to ensure the casing is not damaged during handling and installation.

903.05.05 Steel Reinforcement

Steel reinforcement shall be according to OPSS 1440.

903.05.06 Concrete

903.05.06.01 General

Concrete shall be according to DBSP 1350.

903.05.06.02 Tube Piles

Concrete shall have a slump of 150 to 180 mm.

903.05.06.03 Caisson Piles

Concrete shall have a slump of 150 to 180 mm. When approved by the Contractor's Engineer in writing, admixtures may be used. Where the liner is to be withdrawn, sufficient retarder shall be added to prevent arching of concrete during liner withdrawal and to prevent setting of concrete until after the liner is withdrawn.

903.05.07 Precast Concrete Piles

The production of precast reinforced concrete piles shall be according to DBSP 0904, OPSS 905, and DBSP 0909.

Steel reinforcement shall be placed such that direct loading during the ram stroke shall not occur.

Lifting anchors shall be at least 25 mm clear from reinforcement or prestressing steel in the pile.

Concrete in precast reinforced concrete piles shall be according to DBSP 1350 and have a nominal minimum 28-Day compressive strength of 45 MPa.

Concrete for precast reinforced concrete piles shall be cured according to DBSP 0904.

Concrete for precast reinforced concrete piles shall be placed in smooth mortar-tight forms that are supported to prevent excessive deformation or settlement during placing or curing.

Unformed surfaces shall be finished smooth.

When removed from the form, the pile shall present true, smooth, even surfaces free from honeycombs and voids. The pile shall be straight so that a line stretched from butt to tip on any face shall not be more than 25 mm from the face of the pile at any point.

Each precast reinforced concrete pile shall have the date of manufacture (i.e., yyyy-mm-dd) inscribed on it.

903.05.08 Slurry

903.05.08.01 Solids

Bentonite and polymers shall be according to API Spec 13A.

903.05.08.02 Water

Water shall be according to OPSS 1302.

903.05.08.03 Slurry Composition

The slurry shall consist of a stable colloidal suspension of pulverized solids or polymers thoroughly mixed with water. The density, viscosity, sand content, and pH of the slurry being used during excavation shall be according to API RP 13B-1.

903.06 EQUIPMENT

903.06.01 Hammers

Hammers shall be capable of installing the piles, casings, and liners to the depth or resistance specified in the Contract Documents, without damage to the portions that are not cut off.

The hammer used to chisel the rock point into the rock shall be capable of delivering a controlled blow in 10% increments ranging in energy from zero to the maximum hammer energy.

For precast reinforced concrete piles, the heaviest hammer practicable shall be employed and the stroke limited so as not to damage the piles. When choosing the size of the hammer, consideration shall be given to whether the pile is to be driven to a resistance or to a given depth.

903.06.02 Helmets and Striker Plates

The head of steel piles shall be protected by a striker plate or a helmet. Helmets shall have adequate and suitable cushioning material. Helmets and striker plates shall distribute the blow of the hammer evenly throughout the cross-section of the pile head.

903.06.03 Leads

Pile driver leads shall be built to afford freedom of movement for the hammer and shall be held in position at the top and bottom by guys, stiff braces, or other approved means to ensure support of the pile, casing, or liner while it is being driven. Swinging leads shall not be permitted.

Batter piles, casings, or liners shall be driven with leads aligned parallel to the axis of the pile, casing, or liner. The leads shall be equipped with a fixed, rigid, adjustable kicker.

903.06.04 Followers

When use of followers are specified in the Contract Documents, followers shall be of type, size, shape, length, and weight as to permit driving the pile, casing, or liner at the location and to the required depth or ultimate resistance specified in the Contract Documents. The follower shall be provided with a socket or hood carefully fitted to the top of the pile, casing, or liner to minimize loss of energy and to prevent damage to the pile, casing, or liner, and shall have sufficient rigidity to prevent "whip" during driving.

When followers are permitted, an identical follower shall be used when the set is being determined.

903.07 CONSTRUCTION

903.07.01 Transporting, Storing, and Handling Piles, Casings, Liners, and Reinforcing Steel Reinforcement Cages

903.07.01.01 General

Piles, casings, liners, and steel reinforcement shall be transported, stored, and handled in such a manner that damage is prevented and the strength of the components is not affected by deterioration or deformation.

Components shall be lifted and placed using appropriate lifting equipment, temporary bracing, guys, or stiffening devices so that the components are at no time overloaded, unstable, or unsafe.

Material shall be supported to prevent unequal settlement when stacked.

903.07.01.02 Wooden Piles

Cant hooks, dogs, pile pulls, or use of other lifting methods that might damage the integrity of the pressure treated surface shall not be used. Cuts or breaks in the surface of treated piling shall be given three brush coats of hot creosote oil. Bolt holes shall be treated with three applications of hot creosote oil applied with a bolt hole treater.

903.07.01.03 Handling Holes in Steel Piles

Unless otherwise approved by the design Engineer, holes shall only be made in the portion of the pile to be cut off or in the portion of the pile to be encased in concrete.

When other holes are approved to be cut in a pile, they shall be covered by splice plates placed on both sides of the section. The thickness and the mechanical properties of the plate material shall be at least equivalent to the pile material.

903.07.01.04 Precast Reinforced Concrete Piles

Precast concrete piles shall be handled only from the designated lifting points.

When lifting or transporting precast reinforced concrete piles lift anchors, slings, or other approved means shall be used. Care shall be taken when lifting and transporting to avoid any overstressing of the pile or cracking of the concrete.

Precast reinforced concrete piles shall be so handled to avoid breaking or chipping their edges.

Lift anchors shall be removed, and the holes filled with a non-shrink grout or epoxy installed according to the manufacturer's recommendations.

903.07.01.05 Caisson Casings and Liners

Casings and liners shall be handled and stored in such a manner to avoid damage or distortion to them. The casings and liners shall be maintained circular within $\pm 2\%$ of the casing or liner diameter.

903.07.02 Driven Piles

903.07.02.01 Pile Driving Requirements and Restrictions

Piles shall not be driven until embankment work or excavation work has been completed to the underside of the footing. When driving of the piles is completed, all material between the piles shall be removed to the correct elevation and any holes or voids created shall be filled to the correct elevation with compacted material approved by the Contractor's design Engineer or design Engineer's designee.

Piles shall be installed at the locations specified in the Contract Documents and to the set or depth specified without being damaged. Damage to the pile, casing, or liner during driving shall be prevented by limiting the drop or energy and number of blows of the hammer. The hammer, helmet, cap block, striker plate, and pile shall be coaxial and shall sit squarely upon each other.

A shorter stroke shall be used, and proper precaution shall be taken when there is a danger of damaging or over driving the piles, casing, or liners under conditions such as:

- a) In the early stages of driving a long pile where a hard layer near the ground surface has to be penetrated.
- b) Where there is very soft material of a considerable depth and a large penetration is achieved at each hammer blow.
- c) Where it is anticipated the pile shall meet refusal on rock or other impenetrable soil.
- d) When piles are driven onto sloping bedrock.

Damage to adjacent structures, Utilities, and fresh concrete shall be prevented during pile installation. Piles shall not be driven within a radius of 8 m of concrete that has been in place for less than 72 hours. Piles shall not be driven within a radius of 15 m of concrete that has been in place for less than 72 hours without the approval of the design Engineer.

The tops of all piles shall be either square to the longitudinal axis of the pile or horizontal as indicated in the Contract Documents.

Piles shall not be forced into their proper position using excessive manipulation. Pile damage due to excessive driving shall be avoided.

903.07.02.02 Driving Shoes and Rock Points

Driving shoes and rock points shall be installed in locations specified in the Contract Documents.

Driving shoes shall be welded in accordance with the Contract Documents.

When driving shoes are specified in the Contract Documents, the Titus H bearing pile point, standard model, may be substituted for the driving shoes.

When Oslo points are specified in the Contract Documents, the Titus H bearing pile point, rock injector model, may be substituted for the pile points.

Where proprietary driving shoes are used, they shall be welded or otherwise attached to the driven piles according to the manufacturer's specifications.

903.07.02.03 Splicing

903.07.02.03.01 General

Any damaged material shall be cut-off prior to splicing.

903.07.02.03.02 Wooden Piles

Wooden piles shall not be spliced.

903.07.02.03.03 H-Piles, Tube Piles, and Sheet Piles

Welding shall be according to CSA W59 and shall be done by a qualified welder employed by a firm certified according to CSA W47.1, Division 1 or Division 2.1.

Steel H-piles and steel tube piles may be spliced providing that the pieces being spliced are not less than 3 m long, except for piles at integral abutments for which the pieces being spliced shall not be less than 7 m long. Splices in piles located into a watercourse shall only be introduced under the low water level, unless the piles are encased in concrete.

Sheet piles shall not be spliced without approval by the design Engineer.

903.07.02.03.04 Precast Reinforced Concrete Piles

Precast reinforced concrete piles shall only be spliced when specified in the Contract Documents and the splices shall only be made with approved mechanical splicing devices.

903.07.02.04 Concrete in Steel Tube Piles

Concrete in steel tube piles shall be placed according to DBSP 0904.

903.07.02.05 Cutting Off Piles

903.07.02.05.01 General

Driven piles shall be cut to the elevation as specified in the Contract Documents.

The length of pile supplied shall be sufficient to ensure there is no damaged material below the cut off. Damaged material at the pile head shall be cut off.

Piles shall not be cut off until retapping, re-driving, and specified load testing are complete.

903.07.02.05.02 Wooden Piles

Where wooden piles are broomed, splintered, or otherwise damaged below the cut-off elevation, the pile shall be considered defective and shall be replaced.

903.07.02.06 Protective Coating for Steel H and Steel Tube Piles

Exposed steel H and steel tube piles shall have a coal tar epoxy protective coating applied from an elevation 600 mm below the low water level or finished ground surface up to the top of the exposed steel.

The steel surfaces shall be cleaned according to SSPC-SP10 prior to application of a coal tar epoxy system that shall be according to OPSS 911.

903.07.02.07 Monitoring Driven Piles

903.07.02.07.01 General

The Contractor shall submit a Request to Proceed to Contract Administrator. The driving of piles shall be carefully monitored and controlled and pile driving records produced for each pile under the direction of the Contractor's design Engineer or design Engineer's designee. A pile driving record shall be submitted to the Contract Administrator for the purpose of quality assurance and documentation.

The Contractor shall not overdrive the piles. When driving to a specified ultimate resistance, or driving to bedrock, the Contractor shall drive the piles to the anticipated tip elevation. If a pile does not reach set at the anticipated tip elevation, the Contractor shall notify the design Engineer for review and decision prior to proceeding with driving of that pile.

In soils where there is a possibility of piles moving upward due to ground heave, elevations of completed pile tops shall be measured at time intervals determined by the design Engineer or design Engineer designee while nearby piles are being installed. The readings shall be recorded and submitted to the design Engineer and Contract Administrator (for the purpose of quality assurance and documentation) as the work proceeds.

903.07.02.07.02 Driving to a Specified Elevation

Piles shall be driven to an elevation specified in the Contract Documents. Driving piles to other elevations shall only be done when approved in writing by the design Engineer.

903.07.02.07.03 Driving to a Specified Ultimate Resistance

903.07.02.07.03.01 General

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be determined using the High Strain Dynamic Test at end of initial driving as specified in the Contract Documents. If the specified ultimate resistance is not achieved, retap/restrike shall be conducted after initial driving as specified in the Contract Documents.

High Strain Dynamic Tests (HSDT) shall be performed on a minimum 25% of the king piles installed. The HSDT should be performed near the west, middle, and east portion of both the north and south wall foundation elements.

A Request to Proceed shall be submitted to the Contract Administrator after the design ultimate resistance is achieved.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

903.07.02.07.03.02 Driving to a Set

The founding elevation shall be established by driving to a set determined in accordance with the dynamic formula specified in the Contract Documents or by the application of the wave equation analysis procedure that verifies the pile resistance. This set shall be established on the first pile of every ten piles driven in a pile group.

The other piles shall be controlled by the pile penetration rate in blows per millimetre that correlates to the set.

When new conditions, such as change in hammer size, change in pile size, or change in soil material occur, new sets shall be determined.

903.07.02.07.03.03 Driving to Bedrock

When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile.

Where rock points are used, the rock points shall penetrate into the rock. Piles driven using rock points shall be driven to ensure adequate seating on the bedrock without damaging the pile.

Driving of piles on sloping bedrock shall be stopped when initial contact is made with the bedrock. The bedrock elevation shall be recorded. Driving shall then continue, commencing with energy of 10% of the maximum energy of the hammer. The pile shall be driven in sets of 20 blows at this energy until no penetration is observed. Twenty additional blows shall be applied, and, if no penetration is observed, the energy shall be increased by an additional 10% and the above procedure repeated.

Driving shall continue with these stepped increases in energy and with the same series of blows as described above, until the pile has been seated on the bedrock.

If unrealistic excessive penetration per blow is observed, driving shall be stopped, and this excessive penetration immediately reported to the design Engineer and Contract Administrator.

The design Engineer shall determine when the hammer energy can be increased and when the driving is complete for each pile.

903.07.02.07.04 Wave Equation Analysis

When requested by the Contract Administrator, the Contractor shall supply all equipment, material, and personnel to conduct the wave equation analysis procedure. The design Engineer shall review the results of the analysis and submit a report with recommendations to the Contract Administrator.

903.07.02.07.05 Hammer Performance

When requested by the Contract Administrator, the Contractor's design Engineer or design Engineer designee shall verify the hammer performance using the pile driving analyzer or other approved equivalent. Hammer performance shall be verified to ensure that the actual potential energy (rated energy) is not less than 90% of the stated potential energy. The Contractor shall provide all instrumentation, access, and assistance for the testing and monitoring. The Contractor shall provide a copy of the hammer performance verification results.

903.07.02.07.06 Retapping Tests on Piles

In each pile group, 10% of the piles rounded up to the next whole number, but no fewer than two piles, shall be retapped no sooner than 24 hours after installation of the individual pile to confirm that the ultimate axial resistance has been sustained.

Retapping of piles driven to bedrock is not required.

903.07.02.07.07 Retapping and Redriving Piles

When the retapping tests indicate that the ultimate axial resistance has not been achieved on any one pile, all piles in the group shall be retapped.

Where the retapping reveals that the ultimate axial resistance of the piles has not been achieved, the piles that have not achieved the ultimate axial resistance shall be redriven to the specified resistance.

Where piles have risen, the piles shall be redriven to the original depth.

903.07.02.08 Jetting

Jetting shall be carried out in such a manner that the resistance of the piles already in place and the safety of adjacent structures shall not be impaired. Jetting shall be stopped at least 1 m above the final expected pile-tip elevation and at least 1 m above the tip elevation of any piles previously driven within 2 m of the jet. Where piles are to be end bearing on rock, jetting may be carried to the rock surface.

The driving and jetting of precast reinforced concrete piles shall not be carried out simultaneously.

903.07.03 Caisson Piles

903.07.03.01 General

Caissons shall be constructed as specified in the Contract Documents.

The final bearing elevation shall be as specified in the Contract Documents or as determined by the design Engineer. When permanent casings are not specified, the caisson shall be constructed in a drilled hole with or without the use of a temporary liner or slurry as determined by the Contractor.

903.07.03.02 Excavation

903.07.03.02.01 General

Sidewall stability and basal stability shall be maintained throughout the excavation and concrete placement operation. Soil cave-in into the excavation hole shall be prevented.

The bottom of the excavation shall be cleaned before the start of concrete placement.

Excavation methods shall be such that the sides and bottom of the hole are straight and free of loose material that might prevent intimate contact of the concrete with undisturbed soil or bedrock.

Except when founded on sloping rock, the caisson bottom shall be level. On sloping rock, the caisson bottom may be stepped, with each step not greater than $\frac{1}{4}$ the diameter of the bearing area.

903.07.03.02.02 Casings

When an auger is used to excavate for a casing, the diameter of the auger shall be no greater than the outside diameter of the casing.

903.07.03.02.03 Liners

The diameter of the excavation for the installation of liners shall not exceed the diameter of the liner by more than 150 mm.

903.07.03.02.04 Slurry Method

The level of slurry in the excavation shall be sufficient to prevent the intrusion of water and to maintain a stable wall with no cave-in, sloughing, or basal heave.

Slurry shall be tested as specified in API RP 13B-1. The Contractor shall provide all test equipment required for the tests. A slurry sampler capable of obtaining samples at any depth within the caisson hole shall always be available.

At least 1 set of tests shall be completed every 4 hours during the slurry operation. Samples shall be taken from the mud tank and from within the caisson at a depth within 300 mm of the bottom.

903.07.03.03 Inspection of the Excavation

The bottom of excavations shall be visually inspected. The bottom of excavations shall be inspected with a Mini Shaft Inspection Device (Mini-SID) and/or Shaft quality inspection device (SQUID), lowered to the base of the shaft.

903.07.03.04 Dewatering

Where dewatering is required, the Contractor shall affect a dewatering scheme in such a manner as to prevent any disturbance to the base founding material. The dewatering shall not create subsidence or cause ground loss that may adversely affect the work or adjacent structures.

903.07.03.05 Backfilling Liners Left in Place

The annular space between a liner permanently left in place and shaft excavation shall be filled with concrete or fluid grout.

903.07.03.06 Steel Reinforcement

Steel reinforcement steel shall be installed according to OPSS 905. Steel reinforcement cages shall be checked to ensure conformance to the Working Drawings prior to installation and during placement of concrete.

The steel reinforcement cage shall be fabricated in one piece.

Welding of steel reinforcement and use of splices shall not be done unless specified in the Contract Documents.

The steel reinforcement shall not be displaced or distorted during the construction of the caisson.

903.07.03.07 Concrete

903.07.03.07.01 General

A Request to Place Structural Concrete shall be submitted to the Contract Administrator prior to concrete placement.

The placement of concrete shall not proceed until the Contract Administrator has issued a Notice to Proceed to the Contractor.

Concrete shall be placed in the caisson according to DBSP 0904, and as specified herein.

The reinforcement shall not be displaced or distorted during the construction of the caisson.

When casing or liner withdrawal is part of the design, arching of concrete during casing or liner withdrawal shall be prevented.

903.07.03.07.02 Concrete Placed in the Dry

The concrete may be placed free fall provided the fall is vertically down the centre of the opening and transverse ties, spacers or other objects do not impede the free fall. In the event of interference with the concrete free fall, an elephant trunk or other means shall be used to prevent concrete segregation.

Concrete shall be placed in a continuous operation from the bottom to the top of the caisson or, where columns are cast integral with the caisson, to the elevation of the bottom of the column steel reinforcement cage. The concrete shall be vibrated for the last 1.5 m of the pour.

903.07.03.07.03**Concrete Placed Under Water or Under Slurry**

Tremie or pumped concrete shall be carried out in one continuous operation. The tremie or pumping operation shall be a continuous flow of concrete that prevents the inflow of water or slurry.

Where tremie concrete is to be placed in a caisson under water, the Contractor shall maintain an adequate head of water within the excavations to prevent the inflow of water through the base or walls of the caisson as the concrete is being placed.

Where tremie is placed under slurry, the caisson shall be filled with concrete entirely by tremie and the method of deposition shall not be changed part way up the caisson.

When concrete placement is not started within 6 hours of acceptance of the excavation, the excavation shall be redrilled, cleaned, and the slurry tested before concrete placement commences.

903.07.03.07.04**Withdrawal of Liners**

Arching of concrete during withdrawal of the liner shall be prevented.

During withdrawal, the bottom of the liner shall have a minimum embedment into the concrete being placed and a sufficient head of concrete shall always be maintained above the bottom of the liner to prevent intrusion of soil and water into the hole.

During withdrawal, upward or downward movement of the steel reinforcement shall be monitored. Upward or downward movement shall be restricted to 150 mm.

A theoretical concrete level shall be calculated based on the quantity of concrete placed and the caisson dimensions, and this theoretical level shall be compared to the actual level of concrete in the caisson to provide a check for possible separation of shaft concrete during liner withdrawal.

903.07.03.07.05**Founding Elevation**

The final founding elevation shall be as specified in the Contract Documents or an elevation approved in writing by the design Engineer. When casings are not specified in the Contract Documents, the caisson shall be constructed in a drilled hole with or without the use of a liner or slurry as determined by the Contractor.

Mini-Sid and/or SQUID shall be used to verify the founding soil.

Except when founded on sloping unweathered bedrock, the caisson bottom shall be level. On sloping unweathered bedrock, the caisson bottom may be stepped, with each step not greater than one quarter the diameter of bearing area.

The bearing area of the caisson pile shall be approved by the design Engineer. A Request to Place Structural Concrete shall be submitted to the Contract Administrator prior to placing concrete. Complete access to inspect the bearing area of the caisson pile prior to the placement of concrete shall be given to the Contract Administrator. The placement of concrete shall not proceed until the Contract Administrator has issued a Notice to Proceed to the Contractor.

903.07.04 Displacement Caisson Piles

Work shall be carried out in accordance with the displacement caisson pile suppliers' installation procedures. A permanent liner shall be used when specified in the Contract Documents.

The sequence of installation shall be such as to prevent damage to any recently completed piles.

The pile shall not be founded above or below the specified pile tip elevation without approval in writing from the design Engineer.

The Contractor's design Engineer or design Engineer's designee shall witness the pile installation operation.

903.07.05 Tolerances

903.07.05.01 Driven Piles

- a) Cut-off elevation ± 25 mm.
- b) Deviation from vertical not more than 1H:50V, except in the case of a pile cap or footing supporting only a single row of piles the deviation shall not be more than 1H:75V in the direction of the span.
- c) The deviation from the specified inclination for battered piles shall not exceed 1H:25V.
- d) The centre of the pile at the junction with the pile cap shall be within 150 mm measured horizontally of that specified except in the case of a pile cap or footing supported on a single row of piles the deviation shall not be more than 75 mm measured horizontally in the direction of the span.

903.07.05.02 Caissons and Displacement Caisson Piles

- a) Cut-off elevation ± 25 mm.
- b) Horizontal location at cut-off not more than 5% of shaft diameter or 75 mm, whichever is less.
- c) Vertical alignment not more than 2% of the caisson length from vertical for vertical caissons, or 2% of the caisson length from the specified inclination for battered caissons.

903.07.06 Load Test

When a load test is specified in the Contract Documents, the testing shall be according to ASTM D 1143M for piles under vertical static load, ASTM D 3689 for piles under tensile load, and ASTM D 3966 for piles under lateral loads. The Contractor's design Engineer, or design Engineer's designee, shall organize and notify the Contract Administrator of the scheduled test. The Contractor's design Engineer, or design Engineer's designee, and the Contract Administrator shall witness the pile load test. All records and results of the pile load test shall be submitted to the Contract Administrator for the purpose of quality assurance and documentation.

The Contractor shall provide all necessary personnel, equipment, and material to make adjustments during the tests and shall have at least one skilled worker present for the complete duration of each test. The Contractor shall ensure that this worker shall have demonstrated experience in load testing of piles.

The Contractor shall do all necessary grading work to ensure a level dry working area at the test location and shall erect an adequate enclosure sufficient to provide complete protection from adverse weather conditions for the complete duration of the tests, including all temporary work required to obtain access to the site for the personnel, equipment, and materials.

On completion of the tests, the Contractor shall clear and restore the site to the satisfaction of the Contract Administrator. Piles that are not part of the finished work shall be cut off 1.2 m below ground level or 0.6 m below stream bed level. Any resulting void shall be backfilled with suitable fill material.

903.07.07 Repair of Welds

Any section of weld that does not meet the requirements of the Contract Documents shall be removed and rewelded.

903.08 Quality Assurance

903.08.01 Visual Inspection of Welds

Complete access to visually inspect the welds shall be given to the Contract Administrator.

All welds shall conform with the requirements of CSA W59 and the Contract Documents. A representative sample of splice welds, not less than 30% of the welds will be selected by the Contract Administrator for visual inspection. The selected splice welds shall be taken from different piles.

If the sample welds do not pass the visual inspection and need to be repaired, the visual inspection by the Contract Administrator may be increased up to 100% of the welds.

903.08.02 Non-Destructive Testing of Welds

The Contract Administrator shall be notified in writing, 48 hours in advance of installing piles which will require weld splicing. The Contract Administrator shall be immediately notified in writing, if there are any schedule changes for each pile requiring weld splicing.

A Request to Proceed shall be submitted to the Contract Administrator after the completion of splice welds for each construction stage of work.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

Radiographic or ultrasonic testing shall be carried out by the Contract Administrator using procedures according to CSA W59.

Ultrasonic or radiographic testing shall be carried out on the entire length of selected splice welds chosen at random by the Contract Administrator.

The welds selected for the random ultrasonic or radiographic testing shall be taken from different piles and shall include 10% of the splice welds, rounded to the next highest number, but no fewer than two.

If any welds do not pass the ultrasonic or radiographic-testing and need to be repaired, these non-destructive testing requirements may be increased up to 100% of the welds.

903.08.03 Repaired Welds

All welds that have been repaired shall be visually inspected and shall undergo non-destructive testing performed by the Contract Administrator.

903.08.04 Non-Destructive Test Reports and Visual Inspection Reports

Results from completed Visual Inspection Reports and Non-Destructive Test Reports will be provided upon request.

Costs associated with any required removals and replacement or repairs of defective welds, following the visual inspection or non-destructive testing by the Contract Administrator, shall be the Contractor's responsibility at no additional cost to the Owner. No additional payment will be made for labour and equipment provided by the Contractor, and the Contractor will pay the Owner \$500, for each weld requiring additional re-testing.

903.08.03 Displacement Caisson Piles

A Request to Proceed shall be submitted to the Contract Administrator before the installation of displacement caisson piles.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

903.09 MEASUREMENT FOR PAYMENT – Not Used

903.10 BASIS OF PAYMENT – Not Used

NOTES TO DESIGNER:

* Fill-in one of the following, as recommended by the foundations engineer in consultation with the MTO Foundations Office:

- Dynamic Formula, or
- High-Strain Dynamic Testing

WARRANT: In Design-Build contracts with Deep Foundations.

CUSTODIAN: Tony Sangiuliano, Senior Foundation Engineer, Foundations Section, Structural Standards and Specifications Office and Felipe Mendoza, Senior Contract Innovations Analyst, Special Planning Initiatives Office.

OBSTRUCTIONS

Notice to Contractor

The Contactor is hereby notified that the native soils at the site of the Hornepayne Creek Culvert are glacially derived and as such are very dense and should be expected to contain cobbles and boulders, as encountered at a number of boreholes advanced at this site, which could affect excavations and the installation of deep foundations and/or temporary shoring and roadway protection systems. Consideration of the presence of these obstructions must be made in selection of appropriate equipment and procedures for sub-excavation and installation of the foundation and temporary shoring and roadway protection systems.

Vibration Monitoring for Existing Watermain - Item No.

Non-Standard Special Provision

Scope

Vibration monitoring at the ground surface above the existing watermain located below the Hornepayne Creek should be completed during the driving of temporary protection systems, king piles and sheet piles. Vibration monitoring equipment shall be capable of measuring and recording ground vibration Peak Particle Velocity (PPV) up to 200 mm/s in the vertical, transverse, and radial directions. The equipment shall have been calibrated within the last 12 months either by the manufacturer or other qualified agent. Vibration monitoring threshold should be PPV of 50mm/s for the watermain. A vibration monitoring summary letter / memorandum shall be prepared which documents the vibrations recorded during sheet pile driving.

Basis of Payment

Payment at the contract price for the above item shall include all labour, equipment and material required to do the work.



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