



Foundation Investigation and Design Report

CPR at Hwy 17 Overpass Replacement at Martin, Kenora

Agreement No. 6021-E-0019

Work Item No. 10

GWP No. 6109-17-00

Geocres No.: 52G03-001

Latitude: 49.2436 Longitude: -91.0734

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1 FOUNDATION INVESTIGATION REPORT

1.1 Introduction

This report presents the results of the geotechnical investigation completed by EXP Services Inc. (EXP) for the replacement of the existing CPR Overpass/Bridge at Hwy 17 with a single span bridge. The existing structure is located approximately 8.6 km west of English River at Martin, Kenora, Ontario (Latitude: 49.2436; Longitude: -91.0734) in the Ministry of Transportation (MTO) Northwestern Region. The work was undertaken under Assignment No. 6021-E-0019, Work Item No. 10. The terms of reference (TOR) were provided by MTO in an email dated June 27, 2023.

The purpose of the investigation is to evaluate the subsurface conditions at each abutment of the proposed structure, and based on these data, to permit detailed design for the existing bridge replacement. The site-specific geotechnical investigation consisted of a field investigation program including visual inspections, drilling, soil sampling, and laboratory testing.

This foundation investigation report has been prepared specifically and solely for the project described herein. It contains the factual results of the investigation and the laboratory testing completed for this project. The factual results of the investigation and the laboratory testing completed for this project are presented below.

1.2 Site Description and Geological Setting

1.2.1 Site Description

The existing CP Rail overpass is located on Highway 17 in the township of Martin, 8.6 km west of English River at Martin, Kenora, Ontario. At the site, Highway 17 is a two-lane paved roadway with a speed limit of 90 km/h (unless otherwise posted). Highway 17 generally runs in the east-west direction, but at the site it runs in a southwest to northeast direction. The CPR tracks intersect the highway in approximately an east to west direction at a skew angle of about 29°. For the purpose of the report, the abutment/approach embankment northeast and southwest of the intersection with the CPR tracks will be labelled “north” and “south” abutment/approach embankment, respectively.

Based on the information provided by MTO, the existing bridge was constructed in 1963. The structure is a three-spanned concrete structure with two spans at approximately 15.5 m and 1 span at 14.6 m with a total length of about 45.7 m. The width of the bridge is estimated to be about 12.9 m. Drawings of the existing structure do not indicate the foundations details; however, it can be assumed the abutments and piers are supported on spread footings due to the shallow bedrock and extensive cobbles and boulders in the overburden. The CPR tracks run between the piers with the top of rail and bottom of rail ranging from approximately Elev. 474.74 m to 474.78 m and Elev. 474.60 m to 474.63 m, respectively. A 43.86 m long 750 mm CSP culvert runs underneath the approach embankment situated between the northern pier and north abutment. Additionally, a 48.34 m long 750 mm CSP culvert is located underneath the south approach embankment, approximately 144 m from the south abutment. A gabion wall, approximately 1.8 m high above the ground surface, runs along the bottom of the north abutment forward slope.

The drawing (Plan E-8080-1, Proposed Crossing at Canadian Pacific Railways and King’s Highway 17, dated 1987) provided by MTO shows that the existing elevation of the mid-bridge is about Elev. 483.4 m and the existing highway north and south approaches are at about Elev. 483.0 m. The north and south road approaches have approximately 8.7 m and 8.5 m high embankments (from top of embankment to the lowest point in front of the forward slope). The existing forward slope was estimated from the drawing to range from 1.7H:1V to 2.5H:1V above the gabion

retaining wall at the north abutment and 1.5H:1V to 3.3H:1V at the south abutment based on the drawing. At the north approach, the side slope was approximately 2H:1V on the west side and ranges from 1.8H:1V to 2.7H:1V on the east side. At the south approach, the side slope ranges from 1.2H:1V to 2H:1V on the west side and 3.5H:1V on the east side. Based on observations on the site, these forward abutment slopes appear to be stable (i.e., no visible sign of slope instability). The elevation of the natural ground surface is around 472 m to 474 m within the vicinity of the structure. A gravel access road that leads to a switch station is located on northeast side of the overpass just before the approach embankment.

Photographs 1 to 6 taken by EXP in October 2023 and presented in Appendix A show the site and structure. Photographs 1, 2 and 3 show the surface conditions of the existing bridge along the bridge deck, north approach embankment, and south approach embankment. The existing surface has been repaved at some point, however longitudinal and transversal cracking is present throughout the roadway. Extensive rutting along the highway can be seen as well. Photograph 4 shows a profile view of the existing substructure from the point of view of the NW corner. Photograph 5 shows the substructure at the south abutment and the southern pier. Bedrock outcrops can be seen under the pier/near the bottom of the forward slope. Power lines run perpendicular to the bridge structure closely adjacent to the south abutment. Photograph 6 shows the north abutment, northern pier, and the gabion retaining wall running along the bottom of the forward slope.

1.2.2 Geological Setting

According to the Ministry of Northern Development and Mines, Map 2554 (Quaternary Geology of Ontario, West-Central Sheet, 1991) the surface conditions in the vicinity of the project area consists of gravel and sand which includes proglacial river and deltaic deposits. According to Map 2542 (Bedrock Geology of Ontario, West-Central Sheet, 1991), the bedrock geology of the site is of gneissic tonalite suite: tonalite to granodiorite – foliated to gneissic – with minor supracrustal inclusions. The map also indicates massive granodiorite to granite (massive to foliated granodiorite to granite) and mafic metavolcanic and metasedimentary rocks within the vicinity of the site.

1.3 Previous Investigations

The previous report at the site available in the MTO GEOCRE library is:

- Geocres No. 53G-003. “Hwy. 17 Revision, Line C & C.P.R. Overhead, Approx. 6 Miles West of English River, W.P. 908-60. District #19. W.J. F-59-113” prepared by Department of Highways - Ontario, dated April 6, 1960.

1.4 Investigation Procedures

1.4.1 Site Investigation and Field Testing

The field investigation was performed between October 2 and 14, 2023 by EXP. The field program consisted of drilling six (6) sampled boreholes, numbered BH23-1 to BH23-6, and the performance of conventional standard penetration tests (SPT). Three (3) boreholes were drilled at either side of the existing structure, strategically staggered to facilitate the design of foundations for the abutments, pads for the temporary modular bridge, temporary protection systems, and approach embankments as per the TOR and MTO Guideline for Foundation Services (April 2022). The locations of boreholes drilled during current investigation are shown on Drawing 1 in Appendix B.

All boreholes drilled during this fieldwork were advanced using a truck mounted CME 750 drill rig equipped with hollow stem augers and standard soil sampling /bedrock sampling equipment and NW casing, operated by a

specialist drilling contractor, Maple Leaf Drilling Ltd. All the boreholes (BH23-1 to BH23-6) were advanced to a depth of between 4.4 m and 14.2 m below the ground surface.

For the drilling program, soil samples were obtained using a 51 mm outside diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586) at intervals ranging from 0.75 m to 1.5 m in depth as shown on the attached borehole logs (Appendix C). The original field (uncorrected) SPT “N” values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual (CFEM, pg. 40) and used to provide an assessment of compactness of cohesionless soils. When a hard stratum was reached, sampling of hard material was performed by diamond core drilling, using a 1.5 m long HQ double tube wireline core barrel. Core samples of the bedrock in BH23-1, BH23-3, BH23-4, BH23-5 and BH23-6 were obtained using a 1.5 m long HQ double tube wireline core barrel (core diameter ~65 mm).

Upon completion of the boreholes, groundwater level measurements were carried out in boreholes in accordance with MTO guidelines. A temporary standpipe piezometer was installed in BH23-3 to permit monitoring of the groundwater level at the site. The groundwater records after completion of drilling boreholes and in the piezometer were presented in the borehole log sheets in Appendix C. Upon completion of drilling and field testing, the boreholes were decommissioned by bentonite and auger cuttings. The borehole decommissioning was in general accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the Ontario Water Resources Act). The piezometer in borehole BH23-3 was removed and the borehole was decommissioned on October 13, 2023.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by EXP personnel using a GPS (Garmin Montana 680) and a basic level and survey rod, respectively, having an accuracy of 0.5 m to 2 m in the horizontal direction and 0.1 m in the vertical direction. The borehole coordinates obtained by the GPS were verified by measuring the distances from a known point on the bridge (i.e., the edge of north abutment) using a measuring tape. The bottom of rail at the overpass location was used for a benchmark (BM). Based on the AutoCAD drawings provided by MTO, the elevation of the BM was Elev. 474.6 m. The BM is shown on Drawing 1 in Appendix B.

The fieldwork was supervised by an EXP geotechnical representative who directed the drilling and sampling operation, logged borehole data in accordance with MTO and/or ASTM Standards for Soils Classification and retrieved soil samples for subsequent laboratory testing and identification. All recovered soil samples were placed in labelled moisture-proof bags and returned to EXP’s Thunder Bay laboratory for additional visual, textual, olfactory examination and selective testing.

Table 1.1. Summary of boreholes completed by EXP

Structure	Borehole No.	Location (MTM NAD 83 Zone 12)		Ground Surface Elevation ¹ (m)	Borehole Depth ² (m)	Bedrock Coring Length (m)
		Northing	Easting			
North Abutment	BH23-1	5456758.8	226656.9	483.0	12.2	0.8
	BH23-2	5456773.5	226682.9	483.0	7.6	-

Structure	Borehole No.	Location (MTM NAD 83 Zone 12)		Ground Surface Elevation ¹ (m)	Borehole Depth ² (m)	Bedrock Coring Length (m)
		Northing	Easting			
	BH23-3	5456756.4	226664.6	483.0	14.2	3.0
South Abutment	BH23-4	5456713.3	226618.6	483.0	4.4	1.0
	BH23-5	5456698.6	226592.6	482.8	7.5	0.6
	BH23-6	5456715.8	226610.7	483.0	9.4	4.1

Notes:

1. The referenced ground surface elevations are geodetic.
2. Depths are relative to ground surface.

1.4.2 Laboratory Testing

All soil and rock samples returned to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content on all soil samples and particle size distribution for approximately 25% of the collected soil samples. Two (2) soil samples were selected for chemical analysis and tested at a CALA-certified and accredited laboratory. Uniaxial compression tests were carried out on selected rock core samples.

1.5 Subsurface Conditions

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the borehole log sheets in Appendix C. Laboratory test results of grain size analyses are provided in Appendix D. The “Explanation of Terms Used in Report” preceding the borehole logs in Appendix C forms an integral part of and should be read in conjunction with this report.

A borehole location plan and cross section subsurface profiles are provided in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole log and cross section stratigraphic profiles are inferred from semi-continuous sampling, observations of drilling progress and results of Standard Penetration Tests. These boundaries typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Furthermore, subsurface conditions may vary between and beyond the borehole locations.

In general, the subsoil condition at the proposed bridge replacement location consists of asphalt over sand fill (embankment fill), followed by various native cohesionless layers comprised of silt, silt and sand, and sand, which was underlain by granite to granodiorite bedrock. Cobbles and boulders were frequently encountered throughout the fill and native soil.

A detailed description of the subsurface conditions encountered is discussed further in subsequent sections. It should be noted that the following sections are based on the geotechnical investigation conducted by EXP.

1.5.1 Subsoils

1.5.1.1 Asphalt

A pavement structure consisting of asphalt was encountered at the surface of all boreholes. The thickness of the asphalt ranged between approximately 130 mm and 180 mm.

1.5.1.2 Sand (Fill)

Sand fill was encountered below the asphalt in all boreholes. The approximate elevation of the surface and base of the fill and thickness as encountered in the boreholes are summarized below in Table 1.2.

Table 1.2. Summary of sand fill

Borehole	Elevation ¹ (m)		Layer Surface Depth ² (m)	Layer Thickness (m)
	Top	Bottom		
BH23-1	482.9	473.9	0.1	9.0
BH23-2	482.8	476.1	0.2	6.7
BH23-3	482.9	473.8	0.1	9.1
BH23-4	482.9	479.6	0.1	3.3
BH23-5	482.6	476.0	0.2	6.6
BH23-6	482.8	477.7	0.2	5.1

Notes:

1. The referenced ground surface elevations are geodetic.
2. Depths are relative to ground surface.

The fill layer predominantly consists of sand with trace gravel to gravelly, trace silt to silty, and trace clay. Cobbles and boulders were also commonly encountered throughout the entire fill layer. The material was generally brown in color and moist. The SPT “N” values obtained within this layer ranged from 16 to 108 blows per 0.3 m penetration (excluding tests where the spoon bounced on cobbles/boulders), suggesting that the sand fill is compact to very dense in compactness.

Laboratory testing performed on selected samples consisted of thirty-one (31) moisture content tests and eleven (11) grain size distribution tests. The test results are as follows:

Moisture Content:

- 1.3% to 12.3%

Grain Size Distribution:

- 2% to 35% gravel;
- 49% to 95% sand;
- 2% to 29% silt;

- 2% to 6% clay;
- 3% to 26% silt and clay;

The results of the moisture content and grain size distribution tests are included on the borehole logs in Appendix C. The results of the grain size distribution test are also provided on Figure 1 and Figure 2 in Appendix D.

1.5.1.3 Silt

A native silt layer was encountered below the sand fill in boreholes BH23-1 and BH23-2. The approximate elevation of the surface and base of the layer and thickness as encountered in the boreholes are summarized below in Table 1.3.

Table 1.3. Summary of silt layer

Borehole	Elevation ¹ (m)		Layer Surface Depth ² (m)	Layer Thickness (m)
	Top	Bottom		
BH23-1	473.9	473.1	9.1	0.8
BH23-2	476.1	476.0	6.9	0.1

Notes:

1. The referenced ground surface elevations are geodetic.
2. Depths are relative to ground surface.

The composition of this material consisted predominantly of silt with trace to some sand and trace clay. Occasional cobbles/boulders were also encountered. The material was brown in color and moist to wet. The SPT “N” values obtained within this layer ranged from 5 blows per 0.3 m penetration (BH23-1) to 3 blows per 150 mm (during seating of the split-spoon) followed by 50 blows bouncing on a suspected cobble/boulder (BH23-2) at which point the test was terminated. These blow counts suggest the native silt is loose in compactness.

Laboratory testing performed on selected samples consisted of two (2) moisture content tests and two (2) grain size distribution tests. The test results are as follows:

Moisture Content:

- 26.1% to 28.3%

Grain Size Distribution:

- 0% gravel;
- 9% to 14% sand;
- 84% to 88% silt;
- 2% to 3% clay;

The results of the moisture content and grain size distribution tests are included on the borehole logs in Appendix C. The results of the grain size distribution tests are also provided on Figure 3 in Appendix D.

1.5.1.4 Silt and Sand

A native silt and sand layer was encountered below the sand fill in boreholes BH23-3 and BH23-5. The approximate elevation of the surface and base of the layers and thickness as encountered in the boreholes are summarized below in Table 1.4.

Table 1.4. Summary of silt and sand layer

Borehole	Elevation ¹ (m)		Layer Surface Depth ² (m)	Layer Thickness (m)
	Top	Bottom		
BH23-3	473.8	473.1	9.2	0.7
BH23-5	476.0	475.9	6.8	0.1

Notes:

1. The referenced ground surface elevations are geodetic.
2. Depths are relative to ground surface.

The composition of this material consisted predominantly of silt and sand with trace clay, some gravel (BH23-5) and occasional cobbles/boulders. The material was brown in color and moist to wet.

In BH23-3, the SPT “N” value obtained within this layer was 13 blows per 0.3 m penetration. In BH23-5, the SPT “N” value obtained within this layer was 26 for the initial 75 mm of penetration during the seating of the split-spoon, followed by 50 bounces on a suspected cobble or boulder, at which point the test was terminated. These test results suggest that the layer is compact to dense in compactness.

Laboratory testing performed on selected samples consisted of one (1) moisture content test and one (1) grain size distribution test. The test results are as follows:

Moisture Content:

- 11.2% to 15.6%

Grain Size Distribution:

- 0% gravel;
- 48% sand;
- 52% silt and clay;

The results of the moisture content and grain size distribution test are included on the borehole logs in Appendix C. The results of the grain size distribution test are also provided on Figure 4 in Appendix D.

1.5.1.5 Sand

A native sand layer was encountered below the native silt layer in boreholes BH23-1 and BH23-2 and below the silt and sand layer in BH23-3. Borehole BH23-2 was terminated in this layer. The approximate elevation of the surface and base of the layer and thickness as encountered in the boreholes are summarized below in Table 1.5.

Table 1.5. Summary of sand layer

Borehole	Elevation ¹ (m)		Layer Surface Depth ² (m)	Layer Thickness (m)
	Top	Bottom		
BH23-1	473.1	471.6	9.9	1.5
BH23-2	476.0	475.4	7.0	0.6
BH23-3	473.1	471.8	9.9	1.3

Notes:

1. The referenced ground surface elevations are geodetic.
2. Depths are relative to ground surface.

The composition of this material consisted predominantly of fine-grained sand with a gravel content that ranges from some gravel to gravelly, some silt, trace clay, trace oxidation layers, and occasional cobbles/boulders. The material was brown in color and moist to wet.

The SPT “N” values obtained within this layer ranged from 45 to 50 blows per 0.3 m penetration (excluding tests where the split-spoon bounced on cobbles/boulders), suggesting that the layer is dense to very dense in compactness.

Laboratory testing performed on selected samples consisted of four (4) moisture content tests and two (2) grain size distribution tests. The test results are as follows:

Moisture Content:

- 7.3% to 8.9%

Grain Size Distribution:

- 20% to 29% gravel;
- 51% to 61% sand;
- 16% to 19% silt;
- 1% to 3% clay;

The results of the moisture content and grain size distribution tests are included on the borehole logs. The results of the grain size distribution tests are also provided on Figure 5 in Appendix D.

1.5.2 Bedrock

The presence of bedrock was proved in all boreholes by coring except in BH23-2 since the drilling in that borehole was terminated after SPT and auger refusal. The bedrock was confirmed using coring at all locations where it was encountered. The depth to bedrock ranged from approximately 11.2 m to 11.4 m (corresponding to Elev. 471.6 m to Elev. 471.8 m) to the north of the CPR tracks and a depth of 3.4 m to 7.5 m (corresponding to Elev. 475.9 m to Elev. 479.6 m) to the south of the CPR tracks. The bedrock surface depths and elevations encountered at these borehole locations are listed in Table 1.6. Photographs of rock cores are included in Appendix D.

Table 1.6. Depth and elevation of bedrock surface

Borehole	Depth Below Ground Surface ² (m)	Elevation ¹ (m)	Uniaxial Compressive Strength (MPa)	Comments
North Side of CPR Tracks				
BH23-1	11.4	471.6	-	Bedrock Cored
BH23-2	7.6	475.4	-	Bedrock Inferred from Refusal
BH23-3	11.2	471.8	128.4 (Run 1) 120.5 (Run 2)	Bedrock Cored
South Side of CPR Tracks				
BH23-4	3.4	479.6	-	Bedrock Cored
BH23-5	6.9	475.9	-	Bedrock Cored
BH23-6	5.3	477.7	70.1 (Run 3) 99.5 (Run 4)	Bedrock Cored

Notes:

1. The referenced ground surface elevations are geodetic.
2. Depths are relative to ground surface.

Based on the bedrock HQ cores (~ core diameter 65 mm) recovered, the bedrock at the site consists of granite to granodiorite. In general, the rock samples are described as fine to medium grained, grey/black to pink/white or grey/black with pink/white banding in colour, fractured to predominantly sound, and slightly weathered. The Rock Quality Designation (RQD) measured on the core samples ranged from approximately 68% to 100%, indicating a rock mass of fair to excellent, but typically good to excellent quality. The total core recovery (TCR) of bedrock cores ranged from 97% to 100%.

Uniaxial compression tests were performed on four (4) rock core samples, two (2) samples were from BH23-3 (Run 1 - sample taken between Elev. 471.4 m and 471.2 m and Run 2 - sample taken between Elev. 469.8 m and 469.6 m) and two (2) samples were from BH23-6 (Run 3 - sample taken between Elev. 476.4 m and 476.2 m and Run 4 - sample taken between Elev. 475.7 m and 475.5 m). The uniaxial compressive strength (UCS) was measured to be about 120.5 MPa to 128.4 MPa in BH23-3 and 70.1 MPa to 99.5 MPa in BH23-6, indicating strong to very strong (R4 to R5) rock according to the CFEM. The laboratory uniaxial compression tests results are presented on the borehole records in Appendix C, as well as, in Appendix D.

1.6 Groundwater and Surface Water Conditions

Groundwater was not encountered in any boreholes prior to the introduction of water used in the wash boring/coring process. A 50 mm diameter PVC piezometer was installed upon the completion of drilling BH23-3, where the groundwater reading was taken 2 days after installation.

A summary of the attempts to measure groundwater prior to wash boring/coring in open holes (including the depth where this measurement was taken) and in the temporary piezometer is provided in Table 1.7. It should be noted that fluctuations in the level of the groundwater may occur due to seasonal variations, (precipitation, snowmelt, rainfall), local soil permeability, construction remediation activities, and other related factors.

Table 1.7. Summary of observed groundwater levels

Borehole	Ground Surface Elevation ¹ (m)	Water level Depth ¹ / Elevation ² (m)	Date	Comment
BH23-1	483.0	Dry to depth ~3.7 m	Oct. 14, 2023	Open hole prior to wash boring
BH23-2	483.0	Dry to depth ~7.6 m (Elev. 475.4 m)	Oct. 13, 2023	Open hole
BH23-3	483.0	Dry to depth ~9.1 m	Oct. 6, 2023	Open hole prior to wash boring
		8.5 / 474.5	Oct. 13, 2023	Monitoring well ³
BH23-4	483.0	Dry to depth ~1.2 m	Oct. 5, 2023	Open hole prior to wash boring
BH23-5	482.8	Dry to depth ~6.9 m	Oct. 2, 2023	Open hole prior to coring
BH23-6	483.0	Dry to depth ~5.3 m	Oct. 2, 2023	Open hole prior to coring

Notes:

1. Depths are relative to ground surface.
2. The referenced ground surface elevations are geodetic.
3. Monitoring well installed October 11, 2023.

1.7 Chemical Analysis

Two soil samples were selected for chemical analysis during the current investigations performed by EXP. The soil samples collected by EXP were tested at a CALA-certified and accredited laboratory. The results of the corrosion potential chemical analysis testing including sulfide, chloride, sulfate, pH, electrical conductivity, and resistivity are summarized in Table 1.8.

Table 1.8. Summary of chemical analysis results

Borehole ID	Sample	Depth (m)	Chloride (ppm)	Sulphate (ppm)	pH	Electrical Conductivity (mS/cm)	Resistivity (ohm-cm)
BH23-1	S8	9.1 – 9.7	1100	75	5.67	1.8	560
BH23-3	S4	4.6 – 5.2	1600	58	6.56	2.8	360

2 ENGINEERING DISCUSSION & RECOMMENDATIONS

2.1 General

This section of the report provides geotechnical design recommendations for replacement of the CPR at Hwy 17 Overpass/Bridge located approximately 8.6 km west of English River at Martin, Kenora, Ontario (Latitude: 49.2436; Longitude: -91.0734) in the Ministry of Transportation (MTO) Northwestern Region. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site performed by EXP in October 2023. The compiled factual data is presented in **Part I-Foundation Investigation Report** of this report. The interpretation and recommendations provided are intended solely to permit designers to assess foundation alternatives and design the new bridge replacement. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

This report addresses the geotechnical design of the foundation for the proposed bridge structure by providing geotechnical design parameters at the Ultimate Limit State (ULS) and Serviceability Limit States (SLS) as well as other geotechnical parameters that may be required in accordance with the latest edition of the Canadian Highway Bridge Design Code (CHBDC) (CAN/CSA-S6-19), the Canadian Foundation Engineering Manual (CFEM) (2006), American Railway Engineering and Maintenance-of-Way Association (AREMA) (2023), Canadian Pacific Railway Track Monitoring Requirements for Third Party Projects (2010), the provisions in the Terms of Reference (TOR) and good practice. This structure has the potential to significantly affect alternate transportation corridors and is considered to be of “Typical Consequences Level” associated with exceeding Limit States Design (Section 6.5 and Commentary, CHBDC, 2019). A “Typical Degree of Site and Prediction Model Understanding” is considered appropriate based on the level of foundation investigation completed. Pertinent geotechnical resistance factors and consequence factors have been used in design.

This detailed report provides discussion about the structure foundation type as well as other geotechnical and construction considerations such as assessment of slope stability, site preparation, excavation, dewatering, and frost protection. The use of a temporary modular bridge for the redirection of traffic during construction and temporary protection systems is also addressed.

2.2 Expected Ground Conditions

The following ground conditions along the proposed bridge alignment are evident from the investigation:

- a) At the project site, Highway 17 is a two-lane paved roadway. Highway 17 generally runs in the east-west direction, but at the site it runs in a southwest to northeast direction (i.e., for the purpose of this report, the abutment/approach embankments were labelled “south” and “north, respectively). The CPR tracks intersect the highway in approximately an east to west direction at a skew angle of about 29°.
- b) The existing elevation of the mid-bridge is about Elev. 483.4 m and the existing highway north and south approaches are at about Elev. 483.0 m. The north and south road approaches have approximately 8.7 m and 8.5 m high embankments (from top of embankment to the lowest point in front of the forward slope). The existing forward slope is estimated to range from 1.7H:1V to 2.5H:1V above at gabion retaining wall at the north abutment and 1.5H:1V to 3.3H:1V at the south abutment based on the drawing provided by MTO.

At the north approach, the side slope was approximately 2H:1V on the west side and ranges from 1.8H:1V to 2.7H:1V on the east side. At the south approach, the side slope ranges from 1.2H:1V to 2H:1V on the west side and 3.5H:1V on the east side. Based on observations on the site, these forward abutment slopes appear to be stable (i.e., no visible sign of slope instability). The elevation of the natural ground surface is around 472 m to 474 m within the vicinity of the structure.

- c) At the north approach (BH23-1 to BH23-3), compact to very dense sand fill (~6.7 m to 9.1 m thick) was underlain by native loose silt or compact silt and sand (~0.1 m to 0.8 m thick) over dense to very dense sand (~0.6 m to 1.5 m thick). Cobbles and boulders were regularly encountered throughout the fill and native soils. The sand is underlain by strong to very strong granite to granodiorite bedrock (~11.2 m to 11.4 m below existing ground surface corresponding to Elev. 471.6 m to 471.8 m).
- d) At the south approach (BH23-4 to BH23-6), compact to very dense sand fill (~3.3 m to 6.6 m thick) was underlain by native dense silt and sand (BH23-5 only, ~0.1 m thick). Cobbles and boulders were also encountered throughout the fill and native soils. The fill/native soil is underlain by strong to very strong granite to granodiorite bedrock (~3.4 m to 6.9 m below existing ground surface corresponding to Elev. 475.9 m to 479.6 m).
- e) Groundwater was not encountered in any boreholes prior to the introduction of water using in the wash boring/coring process. Attempts were made to measure groundwater up to a depth 9.1 m (Elev. 473.9 m) at the north approach and 6.9 m (Elev. 475.9 m) at the south approach. A 50 mm diameter PVC piezometer was installed upon the completion of drilling BH23-3. Groundwater was encountered at a depth of 8.5 m below existing ground surface (corresponding to Elev. 474.5 m) in this temporary piezometer. It should be noted that fluctuations in the level of the groundwater may occur due to seasonal variations, (precipitation, snowmelt, rainfall), local soil permeability, construction remediation activities, and other related factors.

2.3 Existing Structure

Based on the information provided by MTO, the existing bridge was constructed in 1963. The structure is a three-spanned concrete structure with two spans at approximately 15.5 m and 1 span at 14.6 m with a total length of about 45.7 m. The width of the bridge is estimated to be about 12.9 m and supported two lanes of traffic. Drawings of the existing structure do not indicate the foundations details; however, it can be assumed the abutments and piers are supported on spread footings due to the shallow bedrock and extensive cobbles and boulders in the overburden. The CPR tracks run between the piers with the top of rail and bottom of rail ranging from approximately Elev. 474.74 m to 474.78 m and Elev. 474.60 m to 474.63 m, respectively. A 43.86 m long 750 mm CSP culverts runs underneath the approach embankment situated between the northern pier and north abutment. Additionally, a 48.34 m long 750 mm CSP culvert is located underneath the south approach embankment, approximately 144 m from the south abutment. A gabion retaining wall, approximately 1.8 m high above the ground surface, runs along the bottom of the north abutment forward slope. A gravel access road that leads to a switch station is located on northeast side of the overpass just before the approach embankment.

Select photographs of the site and existing bridge are presented in Appendix A. The site plan, soil stratigraphy cross-sections and profiles for the proposed bridge replacement are shown on the drawings attached in Appendix B.

2.4 Proposed Structure

As per the TOR, the existing CPR overpass is to be replaced by single span Bridge. Additional details of the new structure are not provided at the time of writing this report. However, it is assumed that the final grade of the bridge deck and approach embankments will remain relatively unchanged.

2.5 Structure Foundations

2.5.1 Bridge Foundation Alternatives

Based on the sub-surface conditions encountered at the proposed bridge site, shallow and deep foundation options were considered at this site from a geotechnical/foundations perspective to support the proposed bridge replacement. Table 2.1 shows the advantages and disadvantages of the considered shallow options.

Table 2.1. Evaluation of foundation alternatives

Options/Rank		Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Shallow Foundation Options					
Spread Footings	<p>North Abutment: Spread footing supported on a Granular A pad over existing fill underlain by dense native sand followed by bedrock</p> <p>South Abutment: Spread footing supported on mass concrete or Granular A pad over bedrock</p> <p>Rank: 1</p>	<ul style="list-style-type: none"> ▪ Straightforward construction ▪ Existing fill is competent for new footing. ▪ Founding on Granular A pad or mass concrete over bedrock provides high geotechnical resistance. 	<ul style="list-style-type: none"> ▪ Additional BH/TP's required at center and 4 corners of footing location to probe bedrock (south abutment) ▪ Lower geotechnical resistances than deep foundations 	<ul style="list-style-type: none"> ▪ Low 	<ul style="list-style-type: none"> ▪ Susceptible to differential settlements
Deep Foundation Options					
Steel H-Piles	<p>North Abutment: Steel H-piles driven to bedrock</p> <p>South Abutment: Not Applicable</p> <p>Rank: 2</p>	<ul style="list-style-type: none"> ▪ Higher geotechnical resistance available ▪ Negligible or minimum settlement 	<ul style="list-style-type: none"> ▪ High cost for mobilization of pile driving equipment ▪ Different depth of competent soil on the south and north abutment ▪ Possible obstruction due to presence of boulders, and potential "hung up" 	<ul style="list-style-type: none"> ▪ Relatively higher than spread footings 	<ul style="list-style-type: none"> ▪ Risk of pile tip damage, should be adequately protected while driving through overburden with boulders ▪ Variation in pile tip elevations (sloping bedrock)

Options/Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Caissons North Abutment: Caissons socketed into bedrock South Abutment: Not Applicable Rank: 3	<ul style="list-style-type: none"> ▪ High axial capacity would result in fewer units than H-Piles ▪ Possible elimination of the pile cap 	<ul style="list-style-type: none"> ▪ Not suitable for integral bridge abutments ▪ Difficulty in socketing the large diameter caissons within sloping bedrock and achieving an adequate seal ▪ Temporary liners and tremie concrete required ▪ Cobbles/boulders may pose difficulties during the advancing of caissons/ temporary liners 	<ul style="list-style-type: none"> ▪ High 	<ul style="list-style-type: none"> ▪ Risk of cave-in, especially below groundwater table during drilling

From a geotechnical/foundations perspective, spread footings are the most feasible/economical option for supporting the proposed bridge replacement.

2.5.2 Bridge Shallow Foundations

The spread footings for the proposed structure can be founded on an engineered granular pad, 1.0 m thick, placed on the existing sand fill at the north abutment and mass concrete over bedrock or a Granular A pad, 0.9 m to 1.0 m thick, over compact to very dense sand fill/bedrock at the south abutment.

2.5.2.1 Footing Elevation

Based on the results of the geotechnical investigation and a requirement for adequate protection against frost penetration in the project area (i.e. a minimum 2.5 m below the lowest surrounding area, see Section 2.5.2.4), the following founding elevations of spread footings presented in Table 2.2 are recommended:

Table 2.2. Recommendations for footing depth for the proposed bridge replacement

Option/Structure Unit (Relevant BH)	Material at Founding Level	Footing Elevation ¹ (m)	Elevation and Depth of Excavation for Footing Below Existing Grade
North Abutment (BH23-1 & BH23-3)	Existing compact to very dense sand fill underlain by dense native sand followed by bedrock	480.5	479.5 (~3.5 m excavation of compact to very dense sand fill; min 1.0 m thick leveling Granular A pad required)
South Abutment (BH23-4 & BH23-6)	0.9 m (east side) to 2.8 m (west side) thick mass concrete over strong to very strong sloping granodiorite to granite bedrock	480.5	479.6 to 477.7 ² (~3.4 m to 5.3 m excavation of compact to very dense sand fill)

Option/Structure Unit (Relevant BH)	Material at Founding Level	Footing Elevation ¹ (m)	Elevation and Depth of Excavation for Footing Below Existing Grade
	0.9 m (east side) to 1.0 m (west side) thick Granular A pad over compact to very dense sand fill /strong to very strong sloping granodiorite to granite bedrock	480.5	479.6 to 479.5 ² (~3.4 m to 3.5 m excavation of compact to very dense sand fill)

Notes:

1. Below frost line of 2.5 m (see Section 2.5.2.4)
2. Anticipated top of sound sloping bedrock.

2.5.2.2 Geotechnical Resistances

Spread footings placed on the properly prepared subgrade at the design level proposed in Table 2.3 should be designed based on the factored geotechnical resistances at ULS and factored serviceability geotechnical resistances (SLS, for 25 mm of settlement) as given in Table 2.3 below. The geotechnical resistances provided in sections below were factored with a typical consequence factor of 1.0 at ULS and SLS; typical degree of understanding (factor of 0.5 at ULS) and high degree of understanding (factor of 0.9 at SLS) in accordance with Table 6.1 and 6.2 of the CHBDC S6-19. The geotechnical resistances provided are for vertical loading conditions only; load eccentricity and load inclination effects should be addressed in accordance with the CHBDC and its commentary. A footing width of 5 m at the north abutment and 4 m for the south abutment is assumed based on the type of structure and encountered subsurface conditions.

Table 2.3. Recommended shallow foundation design parameters for bridge structure

Structure Unit	Soil at Founding Level	Founding/ Excavation Elevations (m)	Width of Footing (m)	Factored Geotechnical Resistance at ULS (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
North Abutment (BH23-1 & BH23-3)	1.0 m Granular A pad ¹ over existing compact to very dense sand fill	480.5 ² /479.5	~5	600	360
South Abutment (BH23-4 & BH23-6)	0.9 m (east side) – 2.8 m (west side) mass concrete over strong to very strong sloping bedrock ^{3,4}	480.5/479.6 to 477.7 ⁵	~4	3000	NA ⁶
	0.9 m (east side) – 1.0 m (west side) Granular A pad over compact to very dense sand fill/strong to very strong sloping bedrock ^{3,4}	480.5/479.6 ⁵ to 479.5		750	450

Notes:

1. *The granular material used for the granular pad shall be granular 'A' conforming to OPSS 1010 and compacted to 100% SPMDD.*
2. *Founding Elevation below frost line of 2.5 m (see Section 2.5.2.4)*
3. *BHs/TPs (center and 4 corners of the footing) are required within the shallow foundation zone as per MTO Guideline for Foundation Engineering Services.*
4. *Mass concrete can be used to create a level surface for footing placement*
5. *Anticipated top of sound sloping bedrock.*
6. *Geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS; therefore, ULS conditions will govern.*

Prior to placing footings, the exposed native subgrade should be inspected according with OPSS.PROV 902. A Qualified Geotechnical Engineer should check that the design foundation elevation is achieved and all unsuitable soils including fill, organics and those soils with the USCS classification of CH, OH, MH, OL or PT have been removed. It should be also checked that the entire footing is placed on the competent foundation soil.

2.5.2.3 Resistance of Footing to Lateral Loads

Resistance to lateral forces/sliding resistance between the subgrade and concrete should be calculated in accordance with Section 6.10.5 of the CHBDC/CSA S6-19. The unfactored values of the coefficient of friction, $\tan \delta$, between the base of cast-in-place concrete footing and the granular subgrade soils below the frost level/bedrock are presented in Table 2.4. A factor of 0.8 should be applied in calculation of the horizontal resistance in accordance with the CHBDC.

Table 2.4. Recommendations for coefficient of friction

Interface	Coefficient of Friction, $\tan \delta$
Granular A and cast-in-place concrete	0.60
Mass concrete and cast-in-place concrete	0.65

2.5.2.4 Frost Protection

Ontario Provincial Standard Drawing (OPSD) 3090.100 indicates that the frost penetration for the Martin, Ontario area is about 2.5 m. Footings situated on native soil should be provided with a minimum of 2.5 m of earth cover or equivalent approved insulation for frost protection. Equivalent protection could be provided by using polystyrene as suggested by the "Canadian Foundation Engineering Manual 2006, Section 13.5.2. page 196". It is usually accepted that 25 mm of polystyrene provides a protection which is equivalent to 600 mm of soil. If the footing is placed on bedrock, the frost protection is not required.

2.5.3 Bridge Deep Foundations

2.5.3.1 Driven Steel Piles

Considering the site specific conditions, steel H-piles (HP 310 x 79 or HP 310 x 110) driven to bedrock can be used to support a bridge designed with integral or semi- integral abutments. The piles will be installed through compact to dense sandy fill and dense to very dense native sand, both with occasional cobbles and boulders, and are expected to terminate on bedrock. It is anticipated that pile cap elevations would be below a frost depth of 2.5 m. Based on

the depth to bedrock encountered in boreholes drilled adjacent to proposed abutment locations, it appears that the termination depths for the piles could be variable.

Deep foundations are not recommended for the south abutment due to the shallow sloping bedrock (i.e. ~3 to 5 m depth from the top of the road). At the north abutment, the H-piles could be driven through the overburden to refusal on the underlying bedrock at ~ 11 m depth from the top of the road. Steel H-piles have advantages as they can be driven into a relatively strong (hard or dense) stratum offering relatively high carrying capacity, can be readily lengthened or cut to size, and they can be relatively roughly handled during delivery with little hazard of damage.

2.5.3.1.1 Geotechnical Axial Resistances of H-Piles

The factored geotechnical axial resistances at ULS and geotechnical axial reactions at SLS for 25 mm of displacement for the recommended driven piles are presented in Table 2.5. These values represent the structural limitation for the pile rather than a geotechnical limitation. It is anticipated that for H-piles driven and seated on the underlying unyielding bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be much greater than the factored axial resistance at ULS; as such, ULS conditions will govern for this foundation type and the SLS value would not apply.

Table 2.5. Factored geotechnical resistances for H-piles

Structure Element	Pile Founding Stratum	Estimated Tip Elevation (m)	Approx. Design Pile Length (m)	Factored Geotechnical Axial Resistance at ULS (kN/pile) ^{1,2}		Factored Serviceability Geotechnical Axial Resistance (kN/pile)	
				HP 310 x 79	HP 310 x 110	HP 310 x 79	HP 310 x 110
North Abutment	Bedrock	~471.6	~9	1,450	2,000	N/A	
South Abutment	Deep foundations are not recommended due to shallow bedrock (< 5 m depth from the top of the road)						

Notes:

1. Pile Resistance governed by structural capacity, based on MTO Structure Office Memorandum 98-01 "Actual Resistances of H-Piles", April 3, 1998; however as per MTO Bridge Office Memorandum "Bridge Office Design Bulletins: Capacity of Steel H-Piles", April 29, 2013: Designers shall follow the CHBDC to establish structural capacity of pile.
2. These structural capacities should be confirmed by a structural engineer.

2.5.3.1.2 Negative Skin Friction (Downdrag Loads) on Steel H-Piles

Since the approach embankments are not going to be raised and the foundation soils are cohesionless (non-compressible), the negative skin friction (or downdrag load) will not need to be taken into consideration during design of the piles supporting the integral abutment.

2.5.3.1.3 Steel H-Pile Installation

Piles should be installed in accordance with OPSS.PROV 903 as amended by SP109F57. The presence of the cobbles and boulders layer encountered in boreholes must be considered for the proper pile installation. In view of this, the piles should be fitted with a driving shoe section (Titus 'H' Bearing Pile point, APF Hard Bite bearing points or similar) offering some protection against buckling at the toe as the piles are driven through the cobbles and boulders or the piles should be stiffened as per OPSD 3000.100, Type I to minimize damage to the piles in anticipation of heavy driving conditions. Care must be taken to avoid overdriving and damaging the pile tip (i.e., the structural capacity of the piles should not be exceeded).

Prior to driving piles, a wave equation (WEAP) analysis should be performed to assess the driving stresses and the anticipated penetration resistance required to develop the required pile capacity. The final driving resistance required to achieve the design load can be determined by the Pile Driving Analyzer (PDA). Dynamic testing (PDA testing) on several piles with the Pile Driving Analyzer must be performed near the beginning of the pile driving phase of construction to confirm the pile capacities. Ten percent of the piles, but no fewer than three per site, should be tested to confirm pile capacities have been achieved. Alternatively, static load tests can be performed, although these are typically much more difficult to set up and are costlier.

MTO permits the control of pile installation using the 'Hiley Formula'. If this method is chosen to control the pile installation, the 'Hiley Formula' can apply in similar settings as shown on MTO standard drawings SS103-11 'Pile Driving Control'.

Alternatively, for the piles driven to bedrock, the set criteria can be established at the time of construction after the piling equipment is known to ensure that the piles are not overdriven and to avoid possible damage to the piles. As the upper portion of the bedrock could be weathered, it is anticipated that the piles will penetrate nominally into the bedrock. Assuming a hammer energy of 60 kJ, a set criterion of 10 blows for penetration of less than 20 mm is recommended for driving into the bedrock to achieve the geotechnical resistances given above. Alternatively, if or when "refusal" is met on stronger interbeds, if applicable, it is generally accepted practice to reduce the hammer energy after abrupt peaking is met in the bedrock, and then to gradually increase the energy over a series of blows to seat the pile in the bedrock.

All piles should be visually monitored by experienced personnel during installation to check for plumbness, set, internal damage, etc. All damaged piles should be rejected, or if the damage is considered to be minor, the pile can be tested to determine the available pile capacity.

Piles in groups should be spaced no closer than three (3) pile diameters. All piles in a group should be checked for heaving during the driving of the adjacent piles.

2.5.3.1.4 Resistance of Piles to Lateral Loads

The resistance to the lateral load will have to be derived from the soil in front of the vertical piles. The resistance to lateral load in front of a vertical pile may be calculated using subgrade reaction theory (Broms' Method) where the coefficient of lateral subgrade reaction, K_h (kPa/m), is based on the following equations (Terzaghi, 1955 and Davisson, 1970):

For cohesionless soils:

$$K_h = n_h(z/d)$$

For cohesive soils:

$$K_h = 67C_u/d$$

where,

K_h =coefficient of horizontal subgrade reactions (kPa/m)

d =pile diameter/ width (m)

n_h =constant of horizontal subgrade reaction (kPa/m)

C_u =undrained shear strength (kPa)

z =depth below ground surface (m)

The following Table 2.6 presents the estimated soil properties and their coefficients of subgrade reaction on the location of north and south abutments. The data presented in the table can be used for lateral load analyses using the L-pile software.

The notations (other than those explained above) used in the table are defined below:

N_{SPT}	Standard Penetration Test, N-value
γ	bulk unit weight (kN/m ³)
ϕ	internal friction angle (deg)
δ	friction angle between steel pile and soils (deg)
ϵ_{50}	strain corresponding to 50% of the maximum principal stress difference
K_p	coefficient of passive earth pressure

Group action for lateral loading should be considered by Reese method using reduction factors on the single pile capacity depending on the geometry of the pile layout.

The reduction factors are as follows:

1. Reduction factors for the piles in a row

$$e = 1 \text{ for } s/b \geq 3.75$$

$$e = 0.64 (s/b)^{0.34} \text{ for } 1 \leq s/b < 3.75$$

2. Reduction factors for leading piles in a line

$$e = 1 \text{ for } s/b \geq 4.0$$

$$e = 0.7 (s/b)^{0.26} \text{ for } 1 \leq s/b < 4.0$$

3. Reduction factors for trailing piles in a line

$$e = 1 \text{ for } s/b \geq 7.0$$

$$e = 0.48 (s/b)^{0.38} \text{ for } 1 \leq s/b < 7.0$$

Table 2.6. Parameters for lateral load analyses

Strata	Elevation (m)	N _{SPT}	γ (kN/m ³)	ϕ (°)	c _u (kPa)	δ (°)	K _{py} (MN/m ³)		ϵ_{50}	n _h (MN/m ³)	K _p
							Static	Cyclic			
Engineered Fill	-	-	21.0	32	-	20	10	10		6.6	3.26
Granular A/B type II	-	-	22.8	35	-	23	10	10	-	7.5	3.69
North Abutment (BH23-1 & BH23-3)											
Sand Fill (compact to very dense)	483.0 – 473.8	28- 108	21	33	-	11	10	10	-	6.6	3.39
Silt and Sand to Silt (loose to compact)	473.8 – 473.1	5 - 13	19	29	-	14	5	5	-	1.5	2.88
Sand (dense to very dense)	473.1 – 471.6	45 – 50+	20	34		17	61	61			

Lateral loading could be resisted fully or partially by use of battered piles. The piles could be installed at a batter of up to 4 vertical to 1 horizontal by simply tilting the pile-driver leads.

2.5.3.2 Caissons

Alternatively, the north abutment may also be supported on caissons socketed into the bedrock. However, this scheme is not suitable for integral abutment design. However, for completeness the design data for caissons are still included in this report. The high axial capacity of caissons would result in fewer units being required to support the bridge abutments than that required for the H –piles, as well as the possible elimination of the pile cap. There will, however, be difficulty in socketing the large diameter caissons within sloping bedrock and achieving an adequate seal. Temporary liners and tremie concrete will be required to install caissons at this site.

Table 2.7 below provides the factored geotechnical axial resistances for different caisson diameters socketed a minimum of 2 m into the bedrock. The given values for caissons were mainly from the shaft resistance of the bedrock socket. The end-bearing will be neglected due to the difficulties in cleaning and inspecting the base of sockets.

Table 2.7. Caisson's geotechnical resistance

Structure Element	Pile Founding Stratum	Estimated Tip Elevation (m)	Approx. Design Pile Length (m)	1.2 m Diameter Caisson		1.5 m Diameter Caisson	
				Factored ULS (kN)	SLS (kN)	Factored ULS (kN)	SLS (kN)
North Abutment	Bedrock	~469.6	~11 to 12	6,500	N/A	8,000	N/A

Structure Element	Pile Founding Stratum	Estimated Tip Elevation (m)	Approx. Design Pile Length (m)	1.2 m Diameter Caisson		1.5 m Diameter Caisson	
				Factored ULS (kN)	SLS (kN)	Factored ULS (kN)	SLS (kN)
South Abutment	Deep foundations are not recommended due to shallow bedrock (< 5 m depth from the top of the road)						

Note:

N/A-not applicable since for caissons socketed into the bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS, and ULS conditions will govern.

To verify the soundness/structural integrity of the caissons, one of the following non-destructive evaluation tests may be performed:

- a) Cross-hole acoustic testing and backscatter gamma ray (gamma-gamma) tests through access tubes installed within the caissons during the placement of the concrete; or
- b) Sonic echo tests. The advantage of these tests is that they do not require preparation during construction of the caissons. The disadvantage is that these tests do not identify all imperfections in a caisson, but provides information about continuity, defects, such as cracks, necking, soil incursions, changes in cross section and approximate pile lengths, unless the pile is very long or the skin friction is too high.

Static load tests to confirm the bearing capacity of the caissons may also be completed as described in ASTM D1143-81 (compression test quick method) and ASTM D3966-90 (Lateral Test) or as per designer's specification.

Giving the uncertainties associated with cleaning and inspection of the caisson base, this foundation type is not the preferred option. As indicated, it is not suited for integral abutment designs.

2.5.4 Retaining Wall in Front of North Abutment

It is assumed that the existing gabion wall at the toe of the north forward slope will be replaced during the construction of the new north abutment. However, there are no details currently available regarding a new retaining wall. However, RSS walls and concrete cantilever walls are considered as viable options, so geotechnical resistances for these two types of retaining walls founded on native soils are tabulated in Table 2.8 below.

Table 2.8. Recommended shallow foundation design parameters for retaining wall in front of north abutment

Structure Unit at North Abutment	Soil at Founding Level	Founding/Excavation Elevations ¹ (m)	Width of Footing (m)	Factored Geotechnical Resistance at ULS (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
RSS Wall	300 mm Granular A pad over dense to very dense sand	473.22/472.9	>2.0 ³	400	210
Concrete Cantilever Wall	100 mm Granular A pad or mass concrete over strong to very strong bedrock	471.7 ² /471.6	>1.0	3500	NA ⁴

Notes:

1. Assumed ground surface at Elev. 474.2 m in front of north abutment forward slope.
2. The embedment depth of RSS wall defined as the distance from the top of the levelling pad to the top of the adjoining finished grade at the toe of the wall is estimated to be 1.0 m (0.4 * frost depth of 2.5 m; as per MTO RSS Design Guidelines). The embedment depth of Concrete Cantilever is equal to frost depth (~2.5 m)
3. RSS wall height above ground surface assumed to be similar to existing gabion wall (~1.8m)
4. Geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS; therefore, ULS conditions will govern.

2.5.5 Lateral Earth Pressure on Structures

2.5.5.1 Static Earth Pressure

The abutment stems and temporary shoring, if any for excavation should be designed to resist lateral earth pressure. Where the abutment stems can be drained effectively to eliminate hydrostatic pressure on the walls, earth pressures equation can be simplified in accordance with the CHDBC. The expression for calculating lateral earth pressure is given by:

$P = K(\gamma h + q)$ for non-braced cut, or $K(0.65\gamma H + q)$ for braced support

where:

P = earth pressure intensity at depth h , kPa

K = earth pressure coefficient

γ = unit weight of retained soil, kN/m³

q = surcharge near wall, kPa

h = depth to point of interest, m

H = total depth of excavation, m

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation (rotation of 0.002 about the base of vertical walls (horizontal displacement divided by wall height) or translation of 0.001 times wall height or a combination of these). Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in design.

The effect of compaction surcharge should be taken into account in the calculations of active and at-rest earth pressures. The lateral pressure due to compaction should be taken as at least 12 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at-rest) pressure is equal to 12 kPa. This pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent oversteering. For design purposes, the unfactored static earth pressure parameters given in Table 2.9 can be used (assuming wall friction is neglected, the back wall is vertical, and the ground surface is horizontal both on the retained side as well as in front of the toe).

Table 2.9. Material types and earth pressure properties under static conditions

Material	Unfactored Friction Angle ϕ' (°)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure At- Rest (K_0)	Unit Weight γ (kN/m ³)
Granular A/B Type II	35	0.27	3.69	0.43	22.8
Engineered Fill	32	0.31	3.26	0.47	21
Existing Sand Fill (compact to very dense)	32	0.31	3.26	0.47	21
Silt (loose)	29	0.35	2.88	0.52	19
Silt and Sand (compact)	30	0.33	3.00	0.50	19
Sandy Silt (dense)	32	0.31	3.26	0.47	20
Sand (dense to very dense)	34	0.28	3.54	0.44	21

2.5.5.2 Lateral Earth Pressures for Seismic Design

The total lateral earth pressure should be calculated considering the static ($K(\gamma z + q)$) and seismic ($((K_{ae} - K_a) \gamma (h - z))$) components using the following equation below:

$$P = K(\gamma z + q) + (K_{ae} - K_a)\gamma(h - z)$$

where,

P = earth pressure intensity at depth z , kPa

K = earth pressure coefficient, K_a for yielding walls, K_0 for non-yielding walls

γ = unit weight of retained soil, kN/m³

q = surcharge near wall, kPa

h = total height of wall, m

z = depth to point of interest, m

Seismic lateral earth pressure parameters yielding and non-yielding walls are provided in Sections 2.9.1.2.1 and 2.9.1.2.2.

2.5.5.2.1 Yielding Walls

Seismic loading should be taken into account in the design in accordance with Section 6.14.7 of the CHBDC. These estimates are based on the Mononobe-Okabe (M-O) pseudo-static method of analysis. The M-O method produces seismic loads that are more critical than the static loads that act prior to an earthquake. The M-O method of seismic lateral earth pressure coefficients for the structural design can be estimated in accordance with Section 6.14.7.2 and C6.14.7.2 of the CHBDC and its Commentary, respectively.

When calculating seismic lateral earth pressures on walls that are capable of moving 25 to 50 mm using the M-O formulation, the seismic horizontal acceleration coefficient (k_h) should be taken as half of the site-adjusted PGA, where, the site-adjusted PGA estimated at ground surface is given as $F(PGA) \cdot PGA$, where, $F(PGA)$ is the PGA-based amplification factor that corresponds to the applicable Site Class as defined in Table 4.8 of the Code. Seismic design values for the site are provided in Section 2.7.1. A site-adjusted PGA of 0.0558 g (for Site Class C), earthquake having a 2% probability of exceedance in 50 years (1 in 2,475-year return period) can be used in the calculation of the seismic active pressure coefficients. k_h is estimated to be 0.0279 g and was used for lateral earth pressures for seismic design.

The effect of the seismic vertical acceleration coefficient (k_v) should be ignored when calculating the seismic lateral earth pressure coefficients. However, the minimum peak vertical acceleration coefficient can be taken as two-thirds of the peak horizontal acceleration coefficient, in accordance with Section 4.4.3.6 of the CHBDC when calculating the seismic lateral earth load.

It should be noted that in the computation of seismic earth pressure coefficients, the wall back-face geometry, backfill slope, and wall friction effects need to be addressed. For design purposes, the following unfactored seismic lateral earth pressure parameters in Table 2.10 can be used (assuming wall friction is neglected, a level ground surface in front of the wall and on the retained side and the back face of the wall is vertical).

Table 2.10. Material types and earth pressure properties under seismic conditions for yielding walls

Material	Unfactored Friction Angle ϕ' (°)	Coefficient of Seismic Active Earth Pressure (K_{ae})	Coefficient of Seismic Passive Earth Pressure (K_{pe})	Unit Weight γ (kN/m ³)
Granular A/B Type II	35	0.29	3.64	22.8
Engineered Fill	32	0.32	3.20	21

2.5.5.2.2 Non-Yielding Walls

For walls that are restrained against lateral movement, the seismic lateral earth pressures should be obtained using the M-O formulation and using a seismic horizontal acceleration coefficient (k_h) equal to the site-adjusted PGA, where, the site-adjusted PGA estimated at the ground surface, given as $F(PGA) \cdot PGA = 0.0558$ g (see Section 2.7.1). The same values for $F(PGA)$ and PGA are used from Section 2.9.1.2.1. The acceleration coefficient determined at the original ground surface should be the acceleration coefficient acting at the wall base. The seismic vertical acceleration coefficient (k_v) can be ignored when calculation the seismic lateral earth pressure coefficient. For design purposes, the following unfactored seismic lateral earth pressure parameters for non-yielding walls are provided in Table 2.11.

Table 2.11. Material types and earth pressure properties under seismic conditions for non-yielding walls

Material	Unfactored Friction Angle ϕ' (°)	Coefficient of Seismic Active Earth Pressure (K_{ae})	Coefficient of Seismic Passive Earth Pressure (K_{pe})	Unit Weight γ (kN/m ³)
Granular A/B Type II	35	0.30	3.58	22.8
Engineered Fill	32	0.34	3.15	21

2.6 Approach Embankments

2.6.1 General

As noted, there are no details regarding the replacement structure at the time of writing this report and that it is assumed that there is no/minimal widening of the embankment or change in elevation to the final grade. The forward and side slopes should be no steeper than 2H:1V and the slopes of the embankment should be provided with adequate erosion protection against surface water runoff.

2.6.2 Slope Stability

Preliminary slope stability analyses were performed to assess the global stability of the forward and side slopes of the new overpass structure to check if a minimum Factor of Safety of 1.5 for static and 1.1 for seismic conditions is achieved as per the CHBDC and MTO criteria for typical degree of understanding (Provincial Engineering Memorandum, Material Engineering and Research Office #2020-01, March 23, 2020). The static and seismic slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium. The SLOPE/W computer program developed by GeoSlope International was employed for computation.

The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report. The seismic properties given in Appendix F (Section 2.7) were obtained from the Natural Resources Canada website, 2020 NBC, using the site location coordinates. Tabulated below in Table 2.12 are the soil parameters used for the slope stability analyses.

Table 2.12. Soil properties used in slope stability analyses

Material Type	Effective Stress Parameters		
	ϕ' (degrees)	c' (kPa)	γ (kN/m ³)
Granular A/B Type II	35	0	22
Existing Sand Fill (compact to very dense)	32	0	20
Silt to Silt and Sand (loose to compact)	29	0	19
Sand (dense to very dense)	34	0	21
Abutment/RSS Wall	45	100	23
Bedrock	Impenetrable		

Based on the borehole information, the subsoils encountered at the work area consist of cohesionless fill and native cohesionless soils. Therefore, effective stress (drained/long-term conditions) analyses of the slopes were performed

taking into consideration the subsoil conditions encountered at the site. The analyses assumes that all organic material (if encountered) will be removed prior to construction. In addition, a traffic surcharge pressure of 16 kPa was adopted in the slope stability assessments.

The north abutment forward and side slopes were considered the critical cases since the south abutment/approach is located on shallow bedrock. Details of the new bridge are not provided at the time of writing this report. The geometry of bridge replacement at the site used in the analyses was based on the following assumptions:

- Abutments will be used and founded at the elevations provided in Table 2.2
- An RSS wall will replace the existing gabion wall at the north abutment, with the face and top of wall at approximately the same location as the existing wall.
- The forward slope and side slope will be constructed at 2H:1V
- All backfill will consist of OPSS.PROV 1010 Granular A or B Type II

Table 2.13 summarizes the results of performed slope stability analyses. The SLOPE/W graphical printout for the analyses is included in Appendix E (Figures E1 – E8).

Table 2.13. Summary of results of forward slope stability analyses

Location		Max Height (m)	Conditions	Min FOS
Forward Slope	Failure Behind Abutment	~8.8	Drained long-term conditions, static condition	1.6 (Figure E1)
			Drained long-term conditions, seismic condition	1.5 (Figure E2)
	Failure in front of abutment (RSS Wall)		Drained long-term conditions, static condition	1.7 (Figure E3)
			Drained long-term conditions, seismic condition	1.5 (Figure E4)
Side Slope	West Side	~7.2	Drained long-term conditions, static condition	1.5 (Figure E5)
			Drained long-term conditions, seismic condition	1.4 (Figure E6)
	East Side	~6.8	Drained long-term conditions, static condition	1.6 (Figure E7)
			Drained long-term conditions, seismic condition	1.4 (Figure E8)

As can be seen, the calculated minimum factors of safety of critical slip surfaces (i.e., calculated min FOS \geq 1.5 for static and min FOS \geq 1.1 for seismic) meet the design criteria for static and seismic conditions given above. Therefore, based on these results, the abutments of the replaced bridge option can safely be constructed with 2H:1V forward slopes with a retaining wall on the north side. However, as stated, the analyses that were performed are conceptual and are based on several assumptions. The stability analysis of the embankments must be re-evaluated once the final design geometry of the footings and embankments is made available.

2.6.3 Embankment Settlement

Considering the presence of relatively shallow bedrock and that it is not planned to change the existing embankment grade nor widening at the bridge location, it is anticipated that there should be negligible additional settlements under the embankment fill.

2.7 Seismic Potential Consideration

2.7.1 Seismic Hazard Site Classification and Values

Seismic characterization of the site should be compliant with the CHBDC, CAN/CSA-S6-19. Table 4.1 in CHBDC (see Clause 4.4.3.2) shows site classification for seismic site response based on average soil properties in the top 30 m.

At this site, the subsoil generally consists of asphalt over compact to very dense sand (some gravel to gravelly) fill, followed by native soils ranging from loose to dense silt to silt and sand, and dense to very dense sand underlain by bedrock (~depth of 11.2 m to 11.4 m below existing ground to the north of the CPR tracks and depth of 3.4 m to 7.5 m to the south of the CPR tracks). Groundwater was encountered in BH23-3 at 8.5 m below the ground surface. Based on soil characteristics, the site class for this site is estimated to be Class "C" according to Table 4.1 of the CHBDC.

From the Natural Resources Canada website, 2020 NBC seismic hazard values are obtained using the site location coordinates and the site-adjusted damped reference spectral accelerations for the project site are shown in Table 2.14 below:

Table 2.14. Seismic design values

Probability of Exceedance in 50 Years (Return Period)	Sa(0.2) (g)	Sa(0.5) (g)	Sa(1.0) (g)	Sa(2.0) (g)	PGA (g)
(Latitude: 49.2436 Longitude: -91.0734)					
10% (1 in 475-year)	0.0308	0.0179	0.00748	0.00255	0.0144
5% (1 in 975-year)	0.056	0.0323	0.0143	0.00521	0.0272
2% (1 in 2475-year)	0.109	0.0628	0.0288	0.0111	0.0558

These values are associated with an earthquake having 10%, 5% and 2% probability of exceedance in a 50-year period (1 in 475-year, 1 in 975-year and 1 in 2475-year events respectively) for Site Class C are also shown on the seismic hazard calculation data sheet for this site attached in Appendix F.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the reference peak ground acceleration (PGA_{ref}). The PGA_{ref} is $0.8 \cdot PGA$ if $Sa(0.2)/PGA < 2.0$, which holds true in this case. Therefore, as per Tables 4.2 to 4.8 of the CHBDC (CAN/CSA-S6-19), the site coefficients $F(0.2)$, $F(0.5)$, $F(1.0)$, $F(2.0)$ and $F(PGA)$, for this site (Seismic Site Class C and PGA_{ref} of $0.8 \cdot PGA$) are all equal to 1.00.

2.7.2 Liquefaction Potential

Considering the levels of bedrock and groundwater relative to the foundation level it is anticipated that liquefaction of foundation soil is negligible.

2.8 Construction Considerations

2.8.1 Construction Alternatives

Construction alternatives considered for the bridge replacement are listed in Table 2.15. The table summarizes advantages and disadvantages of these alternatives and assessed risk/consequences and relative costs of the considered methods.

Table 2.15. Evaluation of bridge replacement alternatives

Options/Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Temporary Modular Bridge Along New Alignment Rank: 2	<ul style="list-style-type: none"> Allows replacement of bridge without staging of traffic. Less shoring required compared to half-and-half method. 	<ul style="list-style-type: none"> Drilling of additional BH's at TMB footing locations required. Additional cost of TMB and it's foundations Embankment construction/ widening required, large amount of fill required Potential settlement due to construction of embankment widening/new embankment Access road on east side may limit embankment widening/new embankment construction to the west side 	<ul style="list-style-type: none"> High due to cost of bridge and construction of new embankment and its removal 	<ul style="list-style-type: none"> Risk of cost overrun and inability to finish job: moderate
Half-and-half Bridge Replacement Rank: 1	<ul style="list-style-type: none"> Half of the bridge can be used to divert traffic. Boreholes at TMB footing location not required. Doesn't required construction of embankment widening/new embankment. No settlement since there is no new earth embankment fill. 	<ul style="list-style-type: none"> High cost of shoring Additional challenges during installation of shoring due to cobbles and boulders and shallow sloping bedrock. Sheet piles may not be viable at this site. Need to decommission the shoring system. 	<ul style="list-style-type: none"> Moderate 	<ul style="list-style-type: none"> Risk of cost overrun and inability to finish job: moderate to high

Based on the above list of advantages and disadvantages of possible construction methods, replacement of the overpass using the half-and-half staged bridge replacement method is considered the most economical and feasible option from a geotechnical perspective.

2.8.1.1 Half-and-half Staged Bridge Replacement

The half-and-half construction method could be utilized to maintain the flow of the traffic on Highway 17. In this method, one lane of the existing bridge will be used to maintain the local traffic while the other half of the existing bridge will be dismantled, and half of the new bridge will be constructed. Upon completion of that half of the new bridge, the traffic will be moved onto the new bridge and the process will be repeated to complete the construction of the other half.

The temporary excavation at the site required to remove a half of the existing embankment at the abutment location would be up to about 3.5 m high at the north side and 3.4 m to 5.3 m high at the south side. Therefore, temporary shoring such as a soldier pile and lagging system will be required as a roadway protection system to allow staging excavation/construction. The sheet piling will be prevented by cobbles and couders encountered in the embankment fill and native soils, as well as the relatively shallow bedrock. It will be the Contractor responsibility to design a suitable temporary support system for the MTO review prior to installation. The Contractor is to follow OPSS.PROV 903, AREMA Manual for Railway Engineering, and CP Rail Track Monitoring Requirements for Third Party Projects regarding excavations for structures, and OPSS.PROV 539, regarding temporary protection systems. Recommendations for a temporary roadway protection are given in Section 2.8.2. The roadway protection can take the form of reversible shoring. Once one-lane is completed the supports can be reversed and the other lane constructed in similar fashion. The shoring system would likely be decommissioned in place. Temporary surface water flow control must be developed by the Contractor.

2.8.1.2 Detour with Temporary Modular Bridge

Creating a detour along a new alignment using a Temporary Modular Bridge (TMB) is considered as a viable construction option at this site. The major advantages of this bridge replacement approach are (i) possibility to maintain traffic flow at the site during construction, (ii) the TMB placed at a new alignment will provide clearance for construction of new bridge and abutments, and (iii) no need for construction of shoring system at centreline of approach embankments which may be difficult due to cobbles and boulders and shallow bedrock. On the other hand, the major disadvantages are (i) traffic interruption with one-way traffic, (ii) construction and removal of new embankment fill, (iii) cost of the TMB and (iv) required boreholes at footing locations.

As per the MTO Guideline for Foundation Engineering Services, additional boreholes at the location of the TMB footings will be required to provide additional subsurface information for design of the TMB if this option is selected. A minimum Factor of Safety (FOS) of 1.3 is required as the design criteria for TMB abutment forward slopes in static conditions. All details of the design of temporary footings for the TMB including slope stability of the temporary excavation slopes in front and adjacent to the TMB abutment footings, determination of bearing capacity, and determination of bearing elevation for abutment footings and safe footing set back distance from the open excavation are the responsibility of the Contractor. The Contractor must retain a Professional Engineer, experienced in bridge design to design the TMB. In addition, the Contractor shall retain a Geotechnical Consultant as their Subcontractor registered in MTO's Consultant Registry, Appraisal and Qualifications System (RAQS), to undertake the geotechnical engineering analyses for slope stability and bearing capacity for the detailed design of the TMB.

2.8.2 Temporary Protection Systems

For temporary excavations including for the half-and-half bridge replacement construction option, temporary shoring will be required as a roadway protection system. A shoring system such as steel sheet piles or soldier piles and lagging system can be employed for temporary excavations at this site, depending on where the shoring will be required. It should be noted that cobbles and boulders were encountered throughout the fill and native soils, and that the exist approach embankments are built on shallow bedrock (particularly on the south approach). Therefore, where shoring is required, a soldier piles and lagging system is most likely to be used. It will be the Contractor's responsibility to design a suitable temporary support system for the MTO review prior to installation. The Contractor is to follow OPSS.PROV 539, AREMA Manual for Railway Engineering, and CP Rail Track Monitoring Requirements for Third Party Projects. The shoring should be designed using the soil parameters recommended in Table 2.9 of this report.

The Contractor should be responsible for the complete design, construction, monitoring and removal of the installed protection system. The protection system shall be designed to provide protection for excavations as required by the OHSA, at locations specified in the contract, and at any locations where the stability, safety or function of an existing structures and/or utility may be impaired by construction work. Decommissioning of temporary shoring must be consistent with good practice to avoid interference with highway systems and utilities, if any. The protection system shall be designed for the Performance Level 2 (for small, less important sections). The minimum requirements for monitoring should include the survey measurements of 6 m apart scaled targets attached to the shoring wall at the elevations specified. If movement approaches the allowable limit of 25 mm (Performance Level 2), suitable measures should be taken to ensure stability of the protection system and to ensure that the movement does not exceed the performance level specified.

2.8.3 Excavation

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety (OHSA) and good construction practice. The existing fill and native soils which should be excavated for construction of the abutments (i.e., compact to very dense cohesionless fill, loose silt, compact to dense silt and sand, and dense to very dense sand) are considered as Type 3 soils above the groundwater table and Type 4 soils below the groundwater table. Temporary excavations (i.e., those that are open only for a short period) above the groundwater table may be made with side slopes not steeper than about 1H:1V, while the temporary slopes below the groundwater table have to be formed at 3H:1V unless a suitable dewatering system is installed to lower the water level below the base of the excavation.

2.8.4 Dewatering

During the geotechnical investigation, groundwater was encountered only in BH23-3 at 8.5 m depth below the ground surface (Elev. 474.5 m). Considering the recommended excavation depths for bridge abutments in Table 2.2, it appears the groundwater level should be below the excavation bottoms. However, since the soils encountered within potential excavation depths for the structures consist of sand fill and native silt, sandy silt, and silt and sand, some perched water within the embankment fill is anticipated during the excavation works for abutments. Water seepage into the excavations should be expected to be heavier during periods of sustained precipitation. All surface water should be directed away from the excavations. If enter the excavation, any surface or groundwater seepage should be removed from the excavation prior to any bedding material being placed. In general, pumping using properly filtered sumps, and/or filtered drains placed along the base of the excavation should provide sufficient groundwater control during foundation works.

Dewatering should be carried out in accordance with OPSS.PROV 902 as modified by NSSP FOUN0003 (Dewatering

of Structure Excavation) attached in Appendix G as well as OPSS.PROV 517. It is the responsibility of the Contractor to propose a suitable dewatering system based on the time of construction and water levels.

2.8.5 Abutment Stems Construction

The following recommendations are made concerning the abutment stems in accordance with the CHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' Type II but with less than 5 percent passing the No. 200 sieve should be used as backfill behind the wall. This fill should be compacted in accordance with OPSS 501.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to subdrains and frost tapers should be in accordance with OPSD 3101.150, 3190.100, and 3121.150. The outlets for these subdrains should not be subject to freezing or flooding.
- Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of 1.0 meter away from walls where the backfill soils are being placed. Hand-operated compaction equipment should be used to compact backfill soils within a 1.0 meter zone adjacent to the walls. Other surcharge should be accounted for in the design, as required.
- The granular fill may be placed in a zone with width equal to 1.8 m behind the back of the abutment stem (Case (a) on Figure C6.20 of the Commentary to the CHBDC) with a frost taper should be included as per OPSD 3101.150 or within the wedge shaped zone defined by a line drawn at 1.5H:1.0V extending up and back from the rear face of the footing (Case (b) on Figure C6.20 of Commentary to the CHBDC). As an alternative OPSD 3101.150 standard drawing can be used.

2.8.6 Corrosion Protection

Two (2) soil samples were selected for chemical analyses during this investigation. The testing was completed to determine the potential degradation of the concrete in the presence of soluble sulphates and the potential of corrosion of exposed steel used in foundations and buried infrastructure. The analyses results are summarized in Table 1.9 of this report.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH value measured at the site ranged from 5.67 to 6.56 which is within the normal range of soil pH of 5.5 to 8.5 (AASHTO, 2000/MTO Gravity Pipe Design Guidelines, May 2007). The chemical data indicates low ($R < 2000$ ohm-cm) resistivity of the tested soil which suggests severe potential for corrosion of buried metallic elements as per Table 3.2 of the MTO Gravity Pipe Design Guideline. The measured chloride content ranged from 1100 to 1600 ppm ($\mu\text{g/g}$) which also indicates a low potential for additional corrosion (Molinas and Mommandi, 2009).

Based on these results, some level of corrosion protection for buried metallic elements is required, depending upon the material type. However, coating of steel H Piles is not done in general practice. It is up to the designer to determine the requirements of appropriate protective coating measures to ensure that all aspects of CHBDC 2014,

Section 2 "Durability" requirements are followed. The test results provided in Table 1.9 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

Consideration should be given by the designer to designing for a « C » type of exposure class of concrete as defined by CSA A23.1:19 Table 1, since the structure will be exposed to de-icing salt. The maximum water-soluble sulphate content of the soils tested is 75 ppm ($\mu\text{g/g}$), i.e. 0.0075%, and being less than 0.10% (as per CSA A23.1:19, Table 3) does not require sulphate resistant cement. The data supports our local experience.

2.8.7 Winter Condition

In the event of construction during freezing temperatures, the foundation stratum should be protected from freezing by the use of loose straw, tarpaulins, propane heaters or other suitable means. In this regard, the base of the excavation should be insulated from sub-zero temperatures immediately upon exposure and until such time the footings are protected with sufficient soil cover to prevent freezing at the foundation level.

2.8.8 Obstructions

Cobbles and boulders were frequently encountered throughout the fill and native soils during the investigation. Therefore care (i.e. pile flange reinforcement or piles should be fitted with a driving shoe, as explained in Section 7.2.2.5) has to be taken during excavation and/or installation of piles. It is recommended that a NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils. An example of a NSSP is included in Appendix G.

Another possible obstacle could be two culverts present at the site; one is a 43.86 m long 750 mm CSP which runs along the bottom of the forward slope at the north abutment, and the other is a 48.34 m long 750 mm CSP which runs below the south approach embankment about 144 m south of the bridge. The culvert at the north abutment might need to be replaced during the construction of the retaining wall. If widening of the embankment is required to accommodate the TMB, the extension of the south culvert might also be necessary.

3 CLOSURE

The recommendations made in this report are in accordance with our present understanding of the project and are provided solely for the team responsible for the design of the works described herein.

We recommend that we be retained to review our recommendations as the design nears completion to ensure that the final design is in agreement with the assumptions on which our recommendations are based and that our recommendations have been interpreted as intended. If not accorded this review, EXP will assume no responsibility for the interpretation and use of the recommendations in this report.

A subsurface investigation is a limited sampling of a site; the subsurface conditions have been established only at the test hole locations. Should conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to assess this additional information and our recommendations, as appropriate. It may then be necessary to perform additional investigations and analyses.

Contractors bidding on or undertaking any proposed work at this site should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as their own interpretations of the factual results provided herein, so they may draw their own conclusions as to how the subsurface conditions may affect them.

This Foundation Investigation and Design Report has been prepared by Daniel Mroz, M.E.Sc., EIT, and Silvana Micic, Ph.D., P.Eng. It was reviewed by TaeChul Kim, M.E.Sc., P.Eng. and by Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact. The field investigation was supervised by Kole Pitkanen and Kristin McLean-Nunn. Traffic control was provided by Ahmad Masoumi, Kristin McLean-Nunn, and Kaden Thorne.

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REFERENCES

- Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.
- Canadian Pacific Railway, 2010. Track Monitoring Requirements for Third Party Projects.
- Canadian Standards Association (CSA), 2019. Canadian Highway Bridge Design Code and Commentary on CAN/CSA-S6-19. CSA Special Publication.
- Ministry of Transportation, April 2022. Guideline for MTO Foundation Engineering Services, Version 03
- Ministry of Northern Development and Mines, Map 2554. Quaternary Geology of Ontario, West-Central Sheet, 1991
- Ministry of Northern Development and Mines Map 2542. Bedrock Geology of Ontario, West-Central Sheet, 1991
- Ministry of Transportation, April 2014. MTO Gravity Pipe Design Guidelines. Circular Culverts and Storm Sewers.
- Molinas, A., and Mommandi, A., 2009. Development of New Corrosion/Abrasion Guidelines for Selection of Culvert Pipe Materials, Report No. CDOT-2009-11. Colorado Department of Transportation, DTD Applied Research and Innovation Branch.
- Terzaghi, K., 1955. Evaluation of Coefficients of Subgrade Reaction. Geotechnique, Vol. 5, No. 4, 297-326.

ASTM International:

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Ontario Provincial Standard Specifications (OPSS):

OPSS.PROV 501 Construction Specification for Compacting

OPSS.PROV 517 Construction Specification for Dewatering

OPSS.PROV 539 Construction Specification for Temporary Protection Systems

OPSS.PROV 902 Construction Specification for Excavating and Backfilling – Structures

OPSS.PROV 903 Construction Specification for Deep Foundations

OPSS.PROV 1010 Material Specification for Aggregates - Base, Subbase, Select Subgrade, And Backfill Material

Ontario Provincial Standard Drawings (OPSD):

OPSD 3000.100 Foundation, Piles, Steel H-Pile, Driving Shoe

OPSD 3090.100 Foundation Frost Depths for Northern Ontario

OPSD 3101.150 Walls, Abutment, Backfill, Minimum Granular Requirement

OPSD 3190.100 Walls, Retaining and Abutment, Wall Drain

OPSD 3121.150 Walls, Retaining, Backfill, Minimum Granular Requirement

*Foundation Investigation and Design Report
Hwy 17 CPR Overpass Replacement at Martin, Kenora
Agreement No. 6021-E-0019; Work Item No. 10
Date: February 29, 2024*

Special Provisions (SP):

NSP FOUN0003 Amendment to OPSS.PROV 902

SP109F57 Amendment to OPSS.PROV 903

Ontario Water Resources Act:

R.R.O 1990, Regulation 903 Wells, under Ontario Water Resources Act, R.S.O. 1990, c. O.40

Ontario Occupational Health and Safety Act (OHSA):

Ontario Regulation 213/91 Construction Projects

LIMITATIONS AND USE OF REPORT

BASIS OF REPORT

This report ("Report") is based on site conditions known or inferred by the geotechnical investigation undertaken as of the date of the Report. Should changes occur which potentially impact the geotechnical condition of the site, or if construction is implemented more than one year following the date of the Report, the recommendations of EXP may require re-evaluation.

The Report is provided solely for the guidance of design engineers and on the assumption that the design will be in accordance with applicable codes and standards. Any changes in the design features which potentially impact the geotechnical analyses or issues concerning the geotechnical aspects of applicable codes and standards will necessitate a review of the design by EXP. Additional field work and reporting may also be required.

Where applicable, recommended field services are the minimum necessary to ascertain that construction is being carried out in general conformity with building code guidelines, generally accepted practices and EXP's recommendations. Any reduction in the level of services recommended will result in EXP providing qualified opinions regarding the adequacy of the work. EXP can assist design professionals or contractors retained by the Client to review applicable plans, drawings, and specifications as they relate to the Report or to conduct field reviews during construction.

Contractors contemplating work on the site are responsible for conducting an independent investigation and interpretation of the borehole results contained in the Report. The number of boreholes necessary to determine the localized underground conditions as they impact construction costs, techniques, sequencing, equipment and scheduling may be greater than those carried out for the purpose of the Report.

Classification and identification of soils, rocks, geological units, contaminant materials, building envelopment assessments, and engineering estimates are based on investigations performed in accordance with the standard of care set out below and require the exercise of judgment. As a result, even comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations or building envelope descriptions involve an inherent risk that some conditions will not be detected. All documents or records summarizing investigations are based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated. Some conditions are subject to change over time. The Report presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, these should be disclosed to EXP to allow for additional or special investigations to be undertaken not otherwise within the scope of investigation conducted for the purpose of the Report.

RELIANCE ON INFORMATION PROVIDED

The evaluation and conclusions contained in the Report are based on conditions in evidence at the time of site inspections and information provided to EXP by the Client and others. The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose as communicated by the Client. EXP has relied in good faith upon such representations, information and instructions and accepts no responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of any misstatements, omissions,

misrepresentation or fraudulent acts of persons providing information. Unless specifically stated otherwise, the applicability and reliability of the findings, recommendations, suggestions or opinions expressed in the Report are only valid to the extent that there has been no material alteration to or variation from any of the information provided to EXP.

STANDARD OF CARE

The Report has been prepared in a manner consistent with the degree of care and skill exercised by engineering consultants currently practicing under similar circumstances and locale. No other warranty, expressed or implied, is made. Unless specifically stated otherwise, the Report does not contain environmental consulting advice.

COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment form part of the Report. This material includes, but is not limited to, the terms of reference given to EXP by its client ("Client"), communications between EXP and the Client, other reports, proposals or documents prepared by EXP for the Client in connection with the site described in the Report. In order to properly understand the suggestions, recommendations and opinions expressed in the Report, reference must be made to the Report in its entirety. EXP is not responsible for use by any party of portions of the Report.

USE OF REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. No other party may use or rely upon the Report in whole or in part without the written consent of EXP. Any use of the Report, or any portion of the Report, by a third party are the sole responsibility of such third party. EXP is not responsible for damages suffered by any third party resulting from unauthorised use of the Report.

REPORT FORMAT

Where EXP has submitted both electronic file and a hard copy of the Report, or any document forming part of the Report, only the signed and sealed hard copy shall be the original documents for record and working purposes. In the event of a dispute or discrepancy, the hard copy shall govern. Electronic files transmitted by EXP have utilized specific software and hardware systems. EXP makes no representation about the compatibility of these files with the Client's current or future software and hardware systems. Regardless of format, the documents described herein are EXP's instruments of professional service and shall not be altered without the written consent of EXP.

Appendix A – Site Photographs



Photograph A1. Bridge deck surface conditions, looking Northeast (October 2023)



Photograph A2. Surface conditions at top of north approach embankment, looking Northeast (October 2023)



Photograph A3. Surface conditions at top of south approach embankment, looking Southwest (October 2023)



Photograph A4. Bridge substructure at northwest corner (October 2023)



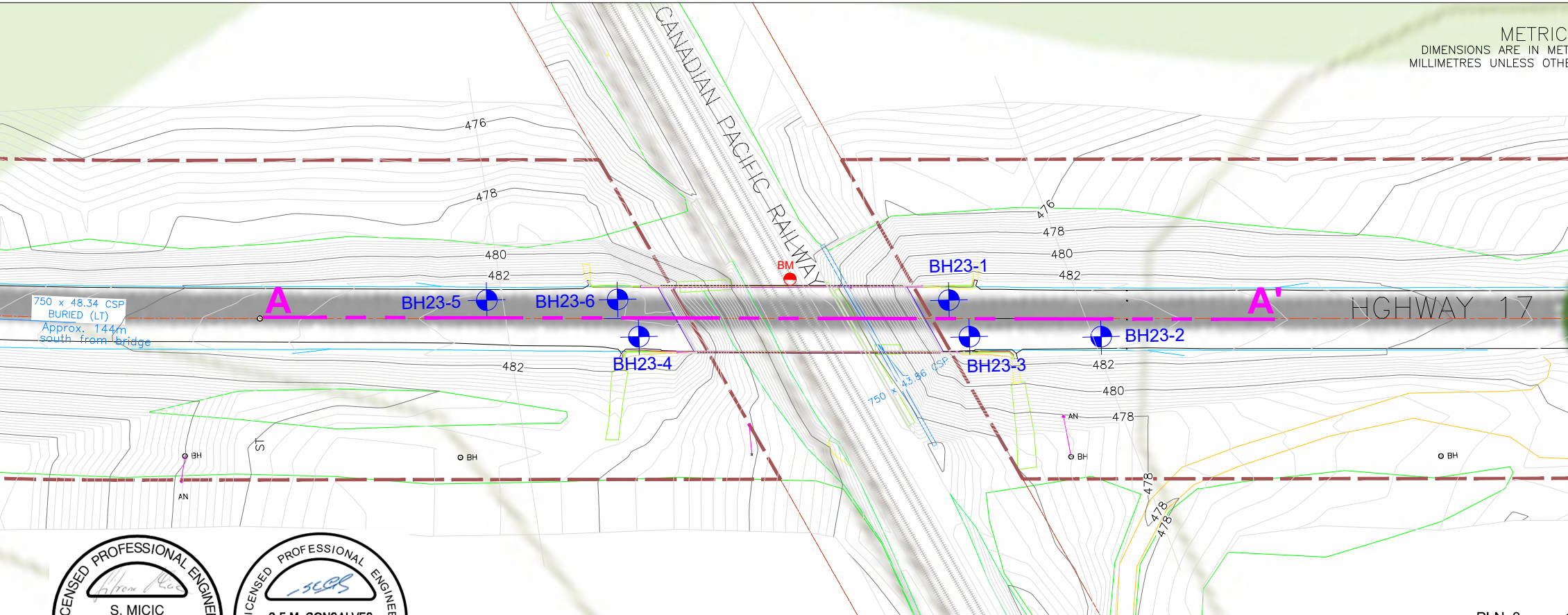
Photograph A5. Bridge substructure at south abutment, facing Southwest (October 2023)



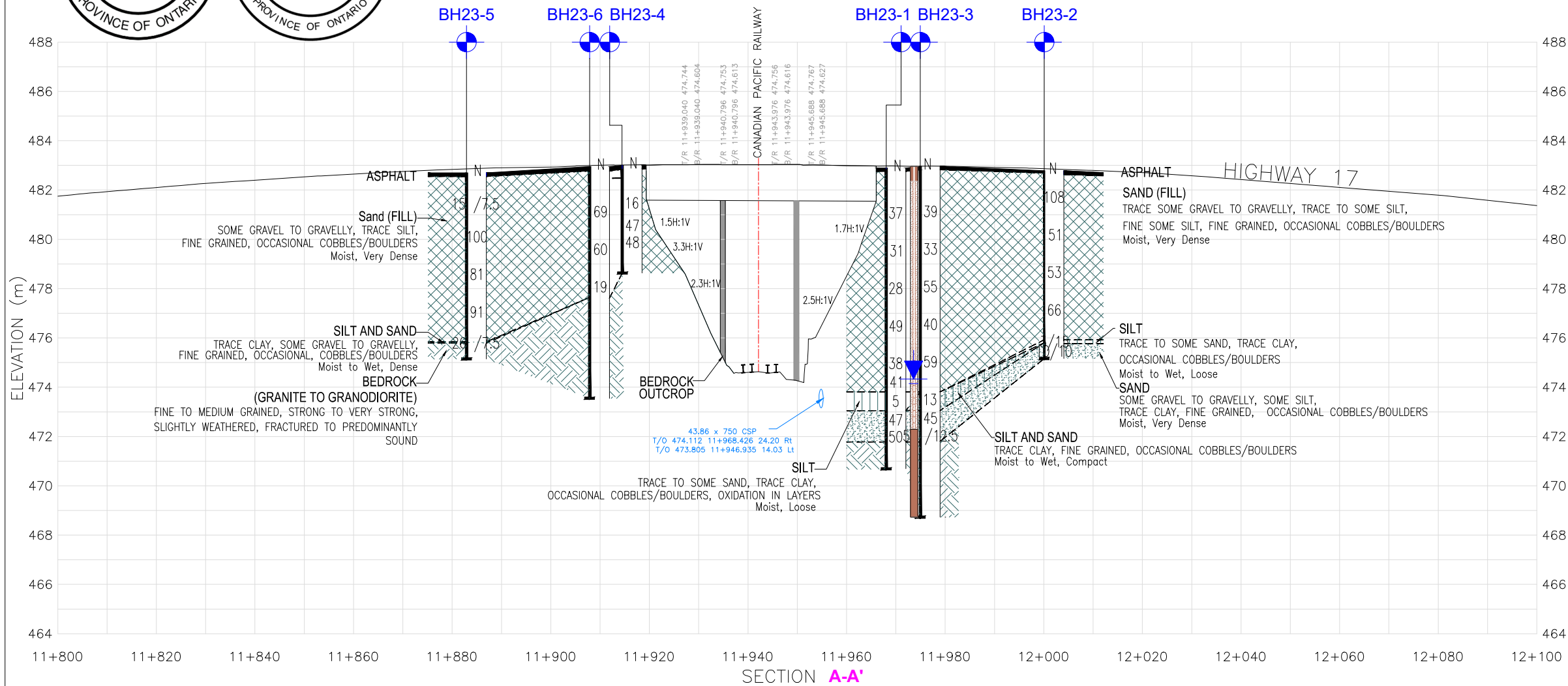
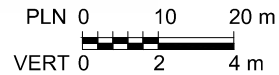
Photograph A6. Bridge substructure at north abutment, facing Northeast (October 2023)

Appendix B –
Borehole Location Plan and Soil Strata

FILE NAME: \\PBRMFS0001\Data_Zeus\2003-Brampton\Proposals\Projects\International\WTO Projects\Retainer NWR\6021-E-0019\A 10 - Hwy 17 bridge replacement Kenora\working drawings\A 10 - Hwy 17 bridge replacement Kenora\plan and profile.dwg
MODIFIED: 2024-01-16 14:37

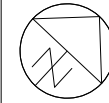


PLAN



SECTION A-A'

CONT No. 2021-E-0019
ASSIG No. 10
GWP No. 6109-17-00



CPR at Hwy 17 Overpass Replacement at Martin,
8.6 km west of English River at Martin, Kenora
Latitude: 49.2436°; Longitude: -91.0734°

SHEET

1



EXP SERVICES INC.



KEY PLAN
N.T.S.

LEGEND

- Borehole Location
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level in Piezometer (most recent) (W. L. STABILIZED)
- Piezometer
- Benchmark (rail line B/R) Elev. 474.6m Based on MTO Provided drawing (Plan E-8080-1 Dated May 1987)

SOIL STRATA SYMBOLS

- ASPHALT
- FILL
- SILT
- SAND
- SAND & SILT
- BEDROCK

BOREHOLE COORDINATES/ NAD 83/ MTM ON-15

BH No.	ELEV.	NORTHING	EASTING
BH23-1	483.0	5456758.8	226656.9
BH23-2	483.0	5456773.5	226682.9
BH23-3	483.0	5456756.4	226664.6
BH23-4	483.0	5456713.3	226618.6
BH23-5	482.8	5456698.6	226592.6
BH23-6	483.0	5456715.8	226610.7

NOTES

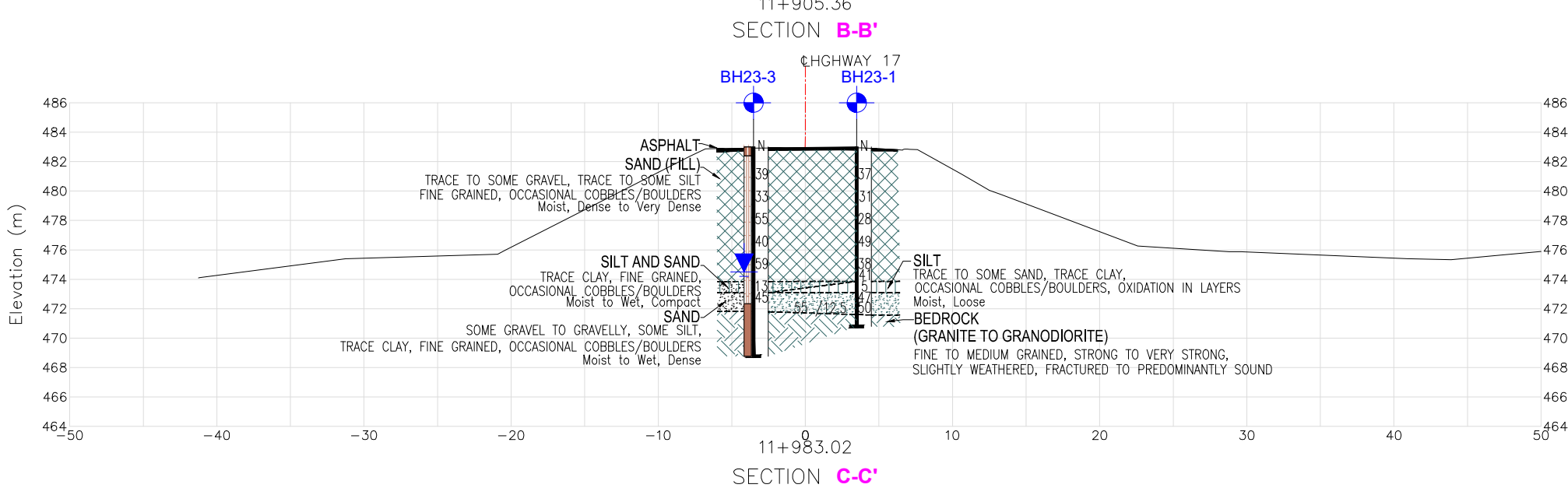
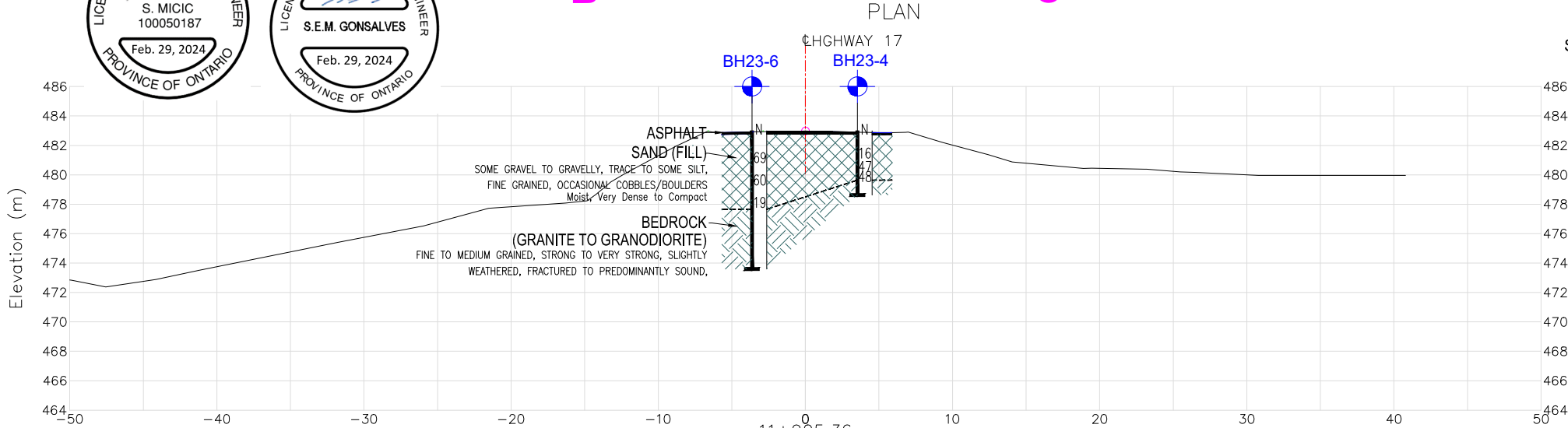
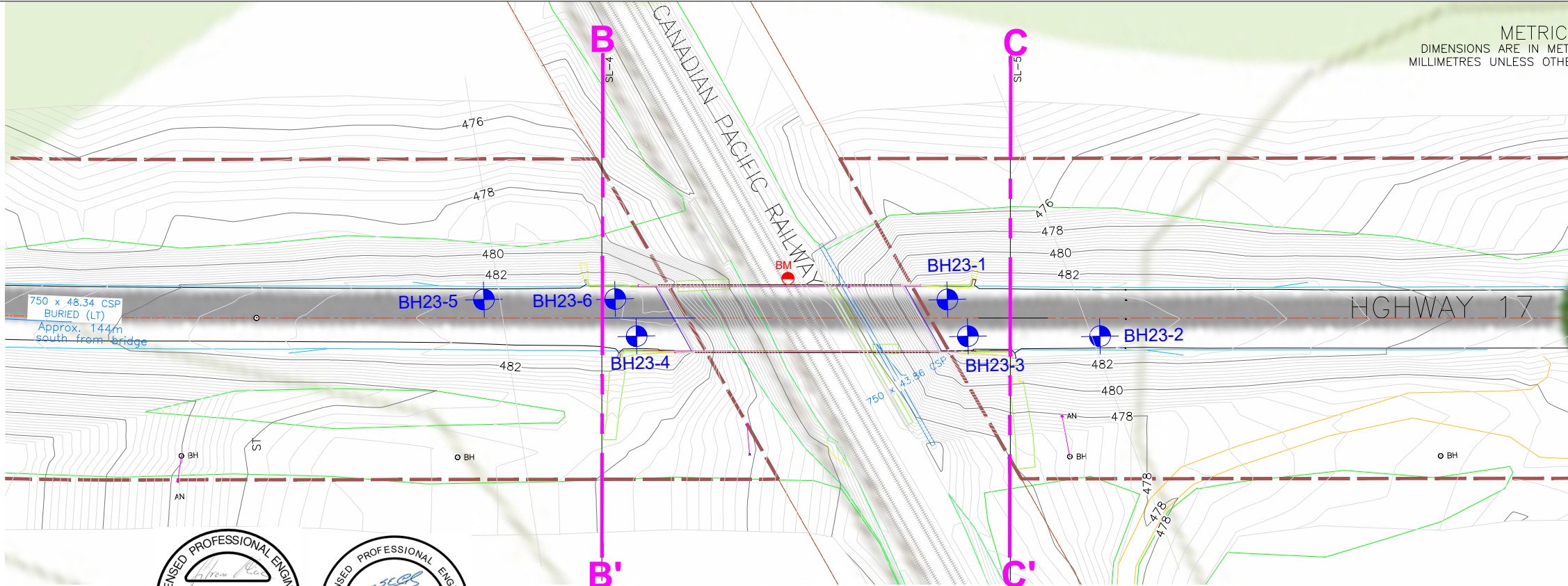
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

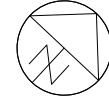
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in the report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

SUBMISSION FOR MTO REVIEW			
NO	DATE	BY	DESCRIPTION
PROJECT No.	ADM-22006096-A2	GEOCREs No.	-
SUBM'D SH	CHKD. SM	DATE	DEC. 16, 2024 SITE-
DRAWN SH	CHKD. TC	APPRD SG	DWG 01

FILE NAME: \\PBRMFS0001\Data_Zeus\2003-Brampton\Proposals\Projects\International\MTD Projects\Retainer NWR\6021-E-0019\A 10 - Hwy 17 bridge replacement Kenora\working drawings\A 10 - Hwy 17 bridge replacement Kenora\plan and profile.dwg
MODIFIED: 2024-01-16 15:12



CONT No. 2021-E-0019
ASSIG No. 10
GWP No. 6109-17-00



CPR at Hwy 17 Overpass Replacement at Martin,
8.6 km west of English River at Martin, Kenora
Latitude: 49.2436°; Longitude: -91.0734°

SHEET
2



EXP SERVICES INC.



KEY PLAN
N.T.S.

LEGEND

- Borehole Location
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level in Piezometer (most recent)
(W. L. STABILIZED)
- Piezometer
- Benchmark (Rail Line B/R) Elev. 474.6m Based
on MTO Provided Drawing (Plan E-8080-1
Dated May 1987)

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BH23-5	482.8	5456698.6	226592.6
BH23-6	483.0	5456715.8	226610.7

NOTES

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SUBMISSION FOR MTO REVIEW			
NO	DATE	BY	DESCRIPTION
PROJECT No.	ADM-22006096-A2	GEOCRES No.	-
SUBM'D SH	CHKD. SM	DATE	DEC. 16, 2024
DRAWN SH	CHKD. TC	APPRD SG	DWG 02

Appendix C – Borehole Logs

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

Topsoil: mixture of soil and humus capable of supporting good vegetative growth.

Peat: fibrous fragments of visible and invisible decayed organic matter.

Fill: where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc.; none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

Till: the term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Terminology describing soil structure:

Desiccated: having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.

Stratified: alternating layers of varying material or color with the layers greater than 6 mm thick.

Laminated: alternating layers of varying material or color with the layers less than 6 mm thick.

Fissured: material breaks along plane of fracture.

Varved: composed of regular alternating layers of silt and clay.

Slickensided: fracture planes appear polished or glossy, sometimes striated.

Blocky: cohesive soil that can be broken down into small angular lumps which resist further breakdown.

Lensed: inclusion of small pockets of different soil, such as small lenses of sand scattered through a mass of clay; not thickness.

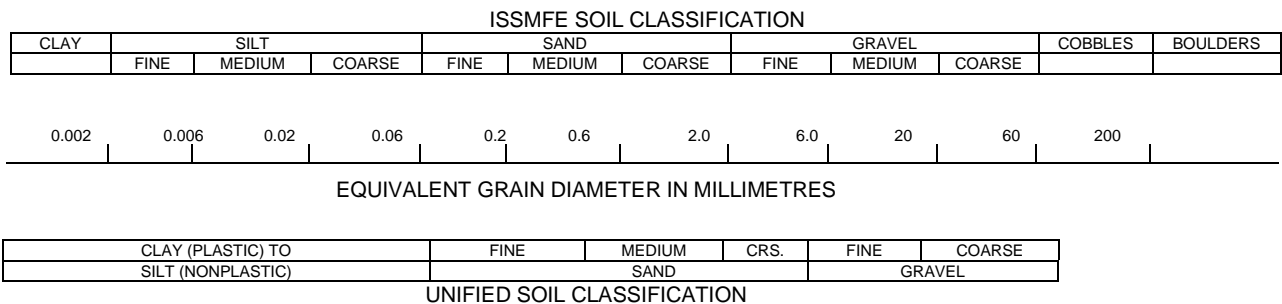
Seam: a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

Homogeneous: same color and appearance throughout.

Well Graded: having wide range in grain sized and substantial amounts of all predominantly on grain size.

Uniformly Graded: predominantly on grain size.

All soil sample descriptions included in this report follow generally the ASTM D2487-11 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) with some modification to reflect current MTO practices. The system divides soils into three major categories: (1) coarse grained, (2) fine-grained, and (3) highly organic. The soil is then subdivided based on either gradation or plasticity characteristics. The system provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification. The classification excludes particles larger than 76 mm. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually in accordance with ASTM D2488-09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems. Others may use different classification systems; one such system is the ISSMFE Soil Classification.



Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present and as described below in accordance with Canadian Foundation Engineering Manual (CFEM):

Table a: Percent or Proportion of Soil

Term	Description	Criteria
"trace"	trace gravel, trace sand, etc.	1% - 10%
"some"	some gravel, some sand, etc.	10% - 20%
Adjective	gravelly, sandy, silty and clayey	20% - 35%
"and"	and gravel, and sand, etc.	>35%
Noun	gravel, sand, silt, clay	>35% and main fraction

The standard terminology to describe cohesionless soils includes the compactness as determined by the Standard Penetration Test 'N' value:

Table b: Apparent Density of Cohesionless Soil

	'N' Value (blows/0.3 m)
Very Loose	N<5
Loose	5≤N<10
Compact	10≤N<30
Dense	30≤N<50
Very Dense	50≤N

The standard terminology to describe cohesive soils includes consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests or similar field and laboratory analysis, Standard Penetration Test 'N' values can also be used to provide an approximate indication of the consistency and shear strength of fine grained, cohesive soils:

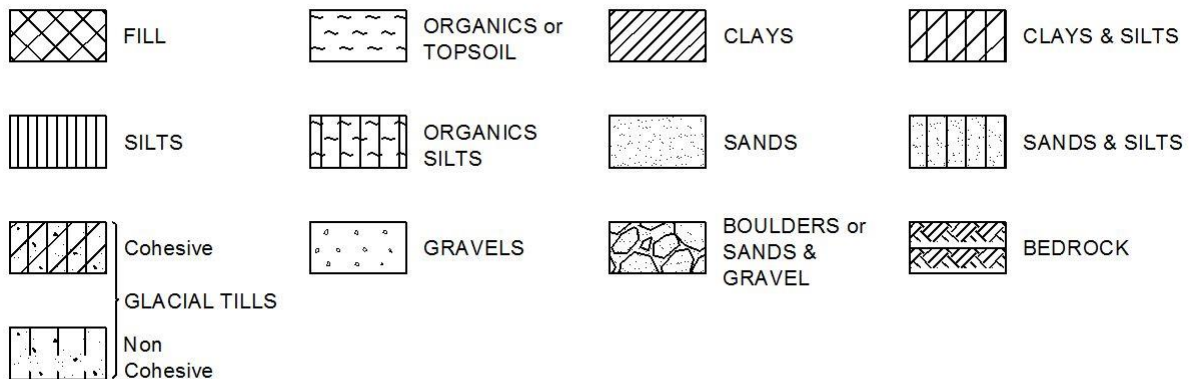
Table c: Consistency of Cohesive Soil

Consistency	Vane Shear Measurement (kPa)	'N' Value
Very Soft	<12.5	<2
Soft	12.5-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

Note: 'N' Value - The Standard Penetration Test records the number of blows of a 140 pound (64kg) hammer falling 30 inches (760mm), required to drive a 2 inch (50.8mm) O.D. split spoon sampler 1 foot (305mm). For split spoon samples where full penetration is not achieved, the number of blows is reported over the sampler penetration in meters (e.g. 50/0.15).

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:



WATER LEVEL MEASUREMENT



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	Split spoon sample (obtained from the Standard Penetration Test)
WS	Wash sample
BS	Bulk sample
TW	Thin wall sample or Shelby tube
PS	Piston sample
AS	Auger sample
VT	Vane test
GS	Grab sample
HQ, NQ, etc.	Rock core samples obtained with the use of standard size diamond drilling bits

STRESS AND STRAIN

u_w	kPa	Pore water pressure
r_u	1	Pore pressure ratio
σ	kPa	Total normal stress
σ'	kPa	Effective normal stress
τ	kPa	Shear stress
$\sigma_1, \sigma_2, \sigma_3$	kPa	Principal stresses
ε	%	Linear strain
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	Principal strains
E	kPa	Modulus of linear deformation
G	kPa	Modulus of shear deformation
μ	1	Coefficient of friction

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	Coefficient of volume change
c_c	1	Compression index
c_s	1	Swelling index
c_r	1	Recompression index
c_v	m ² /s	Coefficient of consolidation
H	m	Drainage path
T_v	1	Time factor
U	%	Degree of consolidation
σ'_{v0}	kPa	Effective overburden pressure
σ'_p	kPa	Preconsolidation pressure
τ_f	kPa	Shear strength
c'	kPa	Effective cohesion intercept
ϕ'	—°	Effective angle of internal friction
c_u	kPa	Apparent cohesion intercept
ϕ_u	—°	Apparent angle of internal friction
τ_R	kPa	Residual shear strength
τ_r	kPa	Remoulded shear strength
S_t	1	Sensitivity = c_u/τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m ³	Density of solid particles
γ_s	kN/m ³	Unit weight of solid particles
ρ_w	kg/m ³	Density of water
γ_w	kN/m ³	Unit weight of water
ρ	kg/m ³	Density of soil
γ	kN/m ³	Unit weight of soil
ρ_d	kg/m ³	Density of dry soil
γ_d	kN/m ³	Unit weight of dry soil
ρ_{sat}	kg/m ³	Density of saturated soil
γ_{sat}	kN/m ³	Unit weight of saturated soil
ρ'	kg/m ³	Density of submerged soil
γ'	kN/m ³	Unit weight of submerged soil
e	1, %	Void ratio
n	1, %	Porosity
w	1, %	Water content
S_r	%	Degree of saturation
W_L	%	Liquid limit
W_P	%	Plastic limit
W_s	%	Shrinkage limit
I_p	%	Plasticity index = $(W_L - W_P)$
I_L	%	Liquidity index = $(W - W_P)/I_p$
I_C	%	Consistency index = $(W_L - W)/I_p$
e_{max}	1, %	Void ratio in loosest state
e_{min}	1, %	Void ratio in densest state
I_D	1	Density index = $(e_{max} - e)/(e_{max} - e_{min})$
D	mm	Grain diameter
D_n	mm	N percent - diameter
C_u	1	Uniformity coefficient
h	m	Hydraulic head or potential
q	m ³ /s	Rate of discharge
v	m/s	Discharge velocity
i	1	Hydraulic gradient
k	m/s	Hydraulic conductivity
j	kN/m ³	Seepage force

Brampton, Ontario

RECORD OF BOREHOLE No BH23-1

1 OF 1

METRIC

W.P. GWP No. 6109-17-00 LOCATION CPR on Hwy 17 ~8.6 km West of English River MTM ON-16 226675E 5456765N ORIGINATED BY KM
 DIST Kenora HWY 17 BOREHOLE TYPE CME 750 Rubber Tire / HSA / HWT COMPILED BY KM
 DATUM Geodetic DATE 2023.10.13 - 2023.10.14 LATITUDE 49.243937 LONGITUDE -91.073305 CHECKED BY JK/SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X P. PENETROMETER												
483.0								20	40	60	80	100								
480.0	ASPHALT [140 mm]																			
0.1	SAND (FILL) - trace gravel, trace silt, trace clay, fine grained, compact to dense, moist, brown, occasional cobbles/boulders		S1	AUGER			482						○				2 95 (3)			
			S2	SS	37		481						○							
							480													
	- some gravel to gravelly						480													
	- auger refusal on cobbles/boulders		S3	SS	31		479						○				35 61 2 2			
	- auger refusal at about 3.7 m depth, wash boring techniques initiated						479													
							479													
	- some silt to silty, trace clay		S4	SS	28		478						○				10 64 (26)			
							478													
							477													
	- trace silt		S5	SS	49		477						○							
							476													
							476													
			S6	SS	38		475						○							
							475													
	- some gravel to gravelly, trace silt		S7	SS	41		475						○							
							474						○							
473.9							474													
9.1	SILT - trace to some sand, trace clay, loose, moist, brown, oxidation in layers, occasional cobbles/boulders		S8	SS	5		473							○			0 14 84 2			
473.1							473													
9.9	SAND - some gravel to gravelly, some silt, trace clay, fine grained, dense to very dense, moist to wet, brown, oxidation in layers, occasional cobbles/boulders		S9	SS	47		473						○				20 61 16 3			
							473													
			S10	SS	50		472						○							
							472													
471.6							472													
11.4	BEDROCK (GRANITE TO GRANODIORITE) - fine to medium grained, strong to very strong, slightly weathered, sound, grey/black to pink/white		R1	CORE			471										Recovery=99% RQD=95%			
470.8							471													
12.2	End of Borehole																			
	- Borehole moved 1.0 m east upon auger refusal at 3.4 m depth																			
	- No groundwater encountered prior to wash boring at 3.7 m depth. Groundwater level was not measured in borehole after water was introduced for wash boring / coring process																			

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

ONTARIO MTO ADM-21019842-J0 - MTO 10 - HWY 17 - HWY 17 OVERPASS REPLACEMENT, MARTIN, KENORA, ON_V3.GPJ ONTARIO MTO.GDT 1/12/24

Brampton, Ontario

RECORD OF BOREHOLE No BH23-2

1 OF 1

METRIC

W.P. GWP No. 6109-17-00 LOCATION CPR on Hwy 17 ~8.6 km West of English River MTM ON-16 226688E 5456779N ORIGINATED BY KM
 DIST Kenora HWY 17 BOREHOLE TYPE CME 750 Rubber Tire / HSA / HWT COMPILED BY KM
 DATUM Geodetic DATE 2023.10.11 - 2023.10.14 LATITUDE 49.244065 LONGITUDE -91.072887 CHECKED BY JK/SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X P. PENETROMETER									
483.0								20	40	60	80	100					
480.8	ASPHALT [150 mm]																
0.2	SAND (FILL) - trace gravel, trace to some silt, fine grained, very dense, moist, brown, occasional cobbles/boulders - some gravel to gravelly below ~0.8 m depth		S1	AUGER													
			S2	SS	108		482										23 67 (10)
							481										
			S3	SS	51		480										
							479										27 63 7 3
			S4	SS	53		478										
							477										
			S5	SS	66		476										0 9 88 3
476.1			S6	SS	3 for 6" (50 bounce)												
476.0	SILT - trace to some sand, trace clay, loose, moist to wet, brown, occasional cobbles/boulders		S7	SS	10 for 4" (50 bounce)												
7.0																	
475.4	SAND - some gravel to gravelly, some silt, trace clay, fine grained, very dense, moist, brown, occasional cobbles/boulders																
7.6	End of Borehole - SPT and auger refusal - No groundwater observed prior to backfill																

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

ONTARIO MTO ADM-21019842-J0 - MTO 10 - HWY 17 - HWY 17 OVERPASS REPLACEMENT, MARTIN, KENORA, ON V3.GPJ ONTARIO MTO.GDT 1/12/24

Brampton, Ontario

RECORD OF BOREHOLE No BH23-3

1 OF 2

METRIC

W.P. GWP No. 6109-17-00 LOCATION CPR on Hwy 17 ~8.6 km West of English River MTM ON-16 226664E 5456760N ORIGINATED BY KM
 DIST Kenora HWY 17 BOREHOLE TYPE CME 750 Rubber Tire / HSA / HWT COMPILED BY KM
 DATUM Geodetic DATE 2023.10.05 - 2023.10.11 LATITUDE 49.243891 LONGITUDE -91.073211 CHECKED BY JK/SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X P. PENETROMETER										
483.0							20	40	60	80	100	20	40	60				
482.0	ASPHALT [140 mm]																	
0.1	SAND (FILL) - trace to some gravel, trace to some silt, fine grained, dense to very dense, moist, brown, occasional cobbles/boulders		S1	AUGER														
			S2	SS	39													
			S3	SS	33													
			S4	SS	55													
			S5	SS	40													
			S6	SS	59													
473.8																		
9.2	SILT AND SAND - trace clay, fine grained, compact, moist to wet, brown, occasional cobbles/boulders		S7	SS	13													
473.1	- SPT & Auger refusal at about 9.7 m depth, wash boring techniques initiated		S8	SS	45													
9.9	SAND - some gravel to gravelly, some silt, trace clay, fine grained, dense, moist to wet, brown, occasional cobbles/boulders		S9	SS	55 for 5" (50 bounce)													
471.8																		
11.2	BEDROCK (GRANITE TO GRANODIORITE) - fine to medium grained, strong to very strong, slightly weathered, fractured to predominantly sound, grey/black with pink/white banding		R1	CORE														
			R2	CORE														
468.8																		
14.2	End of Borehole																	
	- 50 mm diameter PVC piezometer installed upon completion. Screened from about 7.6 m to 10.7 m below ground surface. Removed October 13, 2023																	
	- No groundwater encountered prior																	

Continued Next Page

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

ONTARIO MTO ADM-21019842-J0 - MTO 10 - HWY 17 - HWY 17 OVERPASS REPLACEMENT, MARTIN, KENORA, ON_V3.GPJ ONTARIO MTO.GDT 1/12/24

Brampton, Ontario

RECORD OF BOREHOLE No BH23-3

2 OF 2

METRIC

W.P. GWP No. 6109-17-00 LOCATION CPR on Hwy 17 ~8.6 km West of English River MTM ON-16 226664E 5456760N ORIGINATED BY KM
 DIST Kenora HWY 17 BOREHOLE TYPE CME 750 Rubber Tire / HSA / HWT COMPILED BY KM
 DATUM Geodetic DATE 2023.10.05 - 2023.10.11 LATITUDE 49.243891 LONGITUDE -91.073211 CHECKED BY JK/SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p W W _L				WATER CONTENT (%)
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X P. PENETROMETER						
	to wash boring at 9.1 m depth. Groundwater level was not measured in borehole after water was introduced for wash boring / coring process - Groundwater measured in temporary piezometer at 8.5 m below ground surface on October 13, 2023													

ONTARIO MTO ADM-21019842-J0 - MTO 10 - HWY 17 - HWY 17 OVERPASS REPLACEMENT, MARTIN, KENORA, ON_V3.GPJ ONTARIO MTO.GDT 1/12/24




Brampton, Ontario

RECORD OF BOREHOLE No BH23-4

1 OF 1

METRIC

W.P. GWP No. 6109-17-00 LOCATION CPR on Hwy 17 ~8.6 km West of English River MTM ON-16 226610E 5456710N ORIGINATED BY KM
 DIST Kenora HWY 17 BOREHOLE TYPE CME 750 Rubber Tire / HSA / HWT COMPILED BY KM
 DATUM Geodetic DATE 2023.10.05 - 2023.10.05 LATITUDE 49.243435 LONGITUDE -91.073942 CHECKED BY JK/SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X P. PENETROMETER										
483.0								20	40	60	80	100						
480.0	ASPHALT [130 mm]																	
0.1	SAND (FILL) - some gravel to gravelly, trace to some silt, fine grained, compact to dense, moist, brown, occasional cobbles/boulders - auger refusal [#1] on cobbles/boulders - auger refusal [#3] on cobbles/boulders at about 0.6 m depth, wash boring techniques initiated - auger refusal [#2] on cobbles/boulders at about 1.2 m		S1	AUGER														
			S2	SS	16												24 70 (6)	
			S3	SS	47													
	- gravelly, cobbles		S4	SS	48													
479.6	BEDROCK (GRANITE TO GRANODIORITE) - fine to medium grained, strong to very strong, slightly weathered, fractured to predominantly sound, grey/black to pink/white		R1	CORE													Recovery=98% RQD=84%	
3.4																		
478.6	End of Borehole																	
4.4	- No groundwater encountered prior to wash boring at 1.2 m depth. Groundwater level was not measured in borehole after water was introduced for wash boring / coring process - Borehole moved 1.0 m southwest upon auger refusal [#1] at 0.4 m depth - Borehole moved 1.0 m southwest upon auger refusal [#2] at 1.2 m depth - Borehole moved 1.0 m southwest upon auger refusal [#3] at 0.6 m depth																	

ONTARIO MTO ADM-21019842-J0 - MTO 10 - HWY 17 - HWY 17 OVERPASS REPLACEMENT, MARTIN, KENORA, ON V3.GPJ ONTARIO MTO.GDT 1/12/24

Brampton, Ontario

RECORD OF BOREHOLE No BH23-5

1 OF 1

METRIC

W.P. GWP No. 6109-17-00 LOCATION CPR on Hwy 17 ~8.6 km West of English River MTM ON-16 226588E 5456696N ORIGINATED BY KM
 DIST Kenora HWY 17 BOREHOLE TYPE CME 750 Rubber Tire / HSA / HWT COMPILED BY KM
 DATUM Geodetic DATE 2021.12.15 - 2021.12.17 LATITUDE 49.243305 LONGITUDE -91.074236 CHECKED BY JK/SM

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X P. PENETROMETER							
482.8	ASPHALT [150 mm]														
480.6	SAND (FILL) - some gravel to gravelly, trace silt, fine grained, very dense, moist, brown, occasional cobbles/boulders		S1	AUGER			482								20 73 (7)
0.2	- auger refusal [#1] on cobbles/boulders		S2	SS	15 for 3" (50 ounce)		481								
	- SPT and auger refusal [#2] on cobbles/boulders														
	- SPT and auger refusal [#3] on cobbles/boulders		S3	SS	100		480								
	- SPT and auger refusal [#4] on cobbles/boulders														
							479								
			S4	SS	81		478								
	- some gravel, some silt to silty		S5	SS	91		477								12 64 19 5
476.0	- silty		S7	AUGER			476								16 49 29 6
476.8	SILT AND SAND - trace clay, some gravel to gravelly, fine grained, dense, moist to wet, brown, occasional cobbles/boulders		S6	SS	26 for 3" (50 ounce)										
6.9			R1	CORE											Recovery=100% RQD=82%
475.3	BEDROCK (GRANITE TO GRANODIORITE) - fine to medium grained, strong to very strong, slightly weathered, fractured to predominantly sound, grey/black to pink/white														
7.5	End of Borehole														
	- No groundwater encountered prior to coring at 6.9 m depth. Groundwater level was not measured in borehole after water was introduced for wash boring / coring process														
	- Borehole moved 1.0 m southwest upon auger refusal [#1] at 1.2 m depth														
	- Borehole moved 1.0 m southwest upon SPT and auger refusal [#2] at 1.2 m depth														
	- Borehole moved 1.0 m southwest upon SPT and auger refusal [#3] at 2.3 m depth														
	- Borehole moved 1.0 m southwest upon SPT and auger refusal [#4] at 2.3 m depth														

ONTARIO MTO ADM-21019842-J0 - MTO 10 - HWY 17 - HWY 17 OVERPASS REPLACEMENT, MARTIN, KENORA, ON V3.GPJ ONTARIO MTO.GDT 1/12/24

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

Brampton, Ontario

RECORD OF BOREHOLE No BH23-6

1 OF 1

METRIC

W.P. GWP No. 6109-17-00 LOCATION CPR on Hwy 17 ~8.6 km West of English River MTM ON-16 226611E 5456716N ORIGINATED BY KM
 DIST Kenora HWY 17 BOREHOLE TYPE CME 750 Rubber Tire / HSA / HWT COMPILED BY KM
 DATUM Geodetic DATE 2023.10.02 - 2023.10.04 LATITUDE 49.243489 LONGITUDE -91.073927 CHECKED BY JK/SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X P. PENETROMETER											
483.0							20	40	60	80	100	20	40	60		GR SA SI CL			
480.8	ASPHALT [180 mm]																		
0.2	SAND (FILL) - some gravel to gravelly, trace silt, fine grained, very dense, moist, brown, occasional cobbles/boulders		S1	AUGER															
	- some gravel		S2	SS	69														
	- trace to some silt		S3	SS	60											19 70 (11)			
	- compact		S4	SS	19														
477.7																			
5.3	BEDROCK (GRANITE TO GRANODIORITE) - fine to medium grained, strong to very strong, slightly weathered, fractured to predominantly sound, grey/black with pink/white banding to pink/white		R1	CORE												Recovery=97% RQD=68%			
			R2	CORE												Recovery=100% RQD=96%			
			R3	CORE												Recovery=100% RQD=97% UCS test=70.1 MPa			
			R4	CORE												Recovery=100% RQD=100% UCS test=99.5 MPa			
			R5	CORE												Recovery=100% RQD=94%			
473.6																			
9.4	End of Borehole																		
	- No groundwater encountered prior to coring at 5.3 m depth. Groundwater level was not measured in borehole after water was introduced for wash boring / coring process																		

ONTARIO MTO ADM-21019842-J0 - MTO 10 - HWY 17 - HWY 17 OVERPASS REPLACEMENT, MARTIN, KENORA, ON_V3.GPJ ONTARIO MTO.GDT 1/12/24

Appendix D –
Laboratory Data and Bedrock Coring Photograph



Photograph D1. Rock cores from BH23-1, Run 1 – 11. 4 m to 12.2 m (October 2023)



Photograph D2. Rock cores from BH23-3, Run 1 and Run 2 – 11. 2 m to 14.2 m (October 2023)



Photograph D3. Rock cores from BH23-4, Run 1 – 3.4 m to 4.4 m (October 2023)



Photograph D4. Rock cores from BH23-5, Run 1 – 6.9 m to 7.5 m (October 2023)

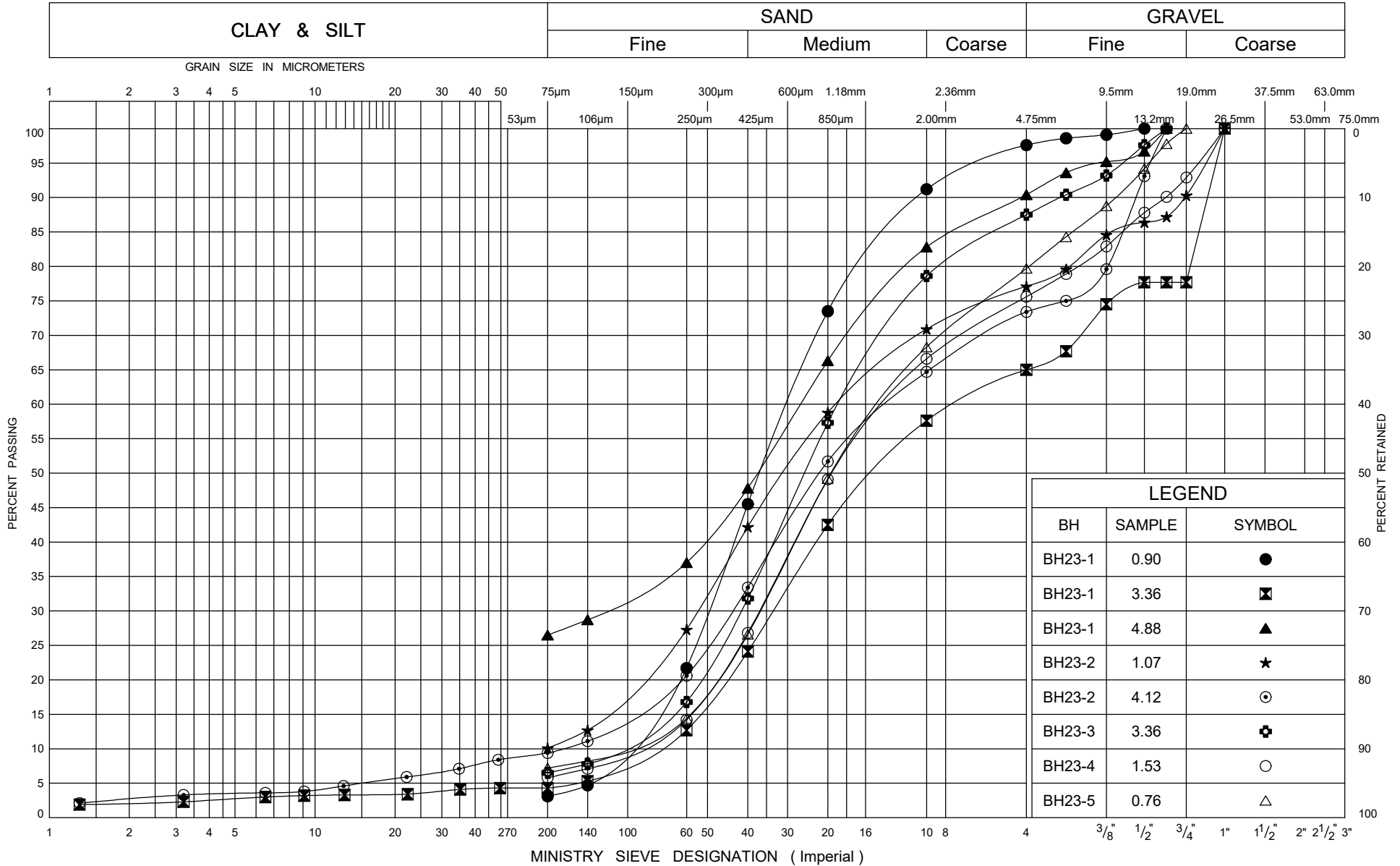


Photograph D5. Rock cores from BH23-6, Run 1 to Run 3 – 5.3 m to 7.2 m (October 2023)

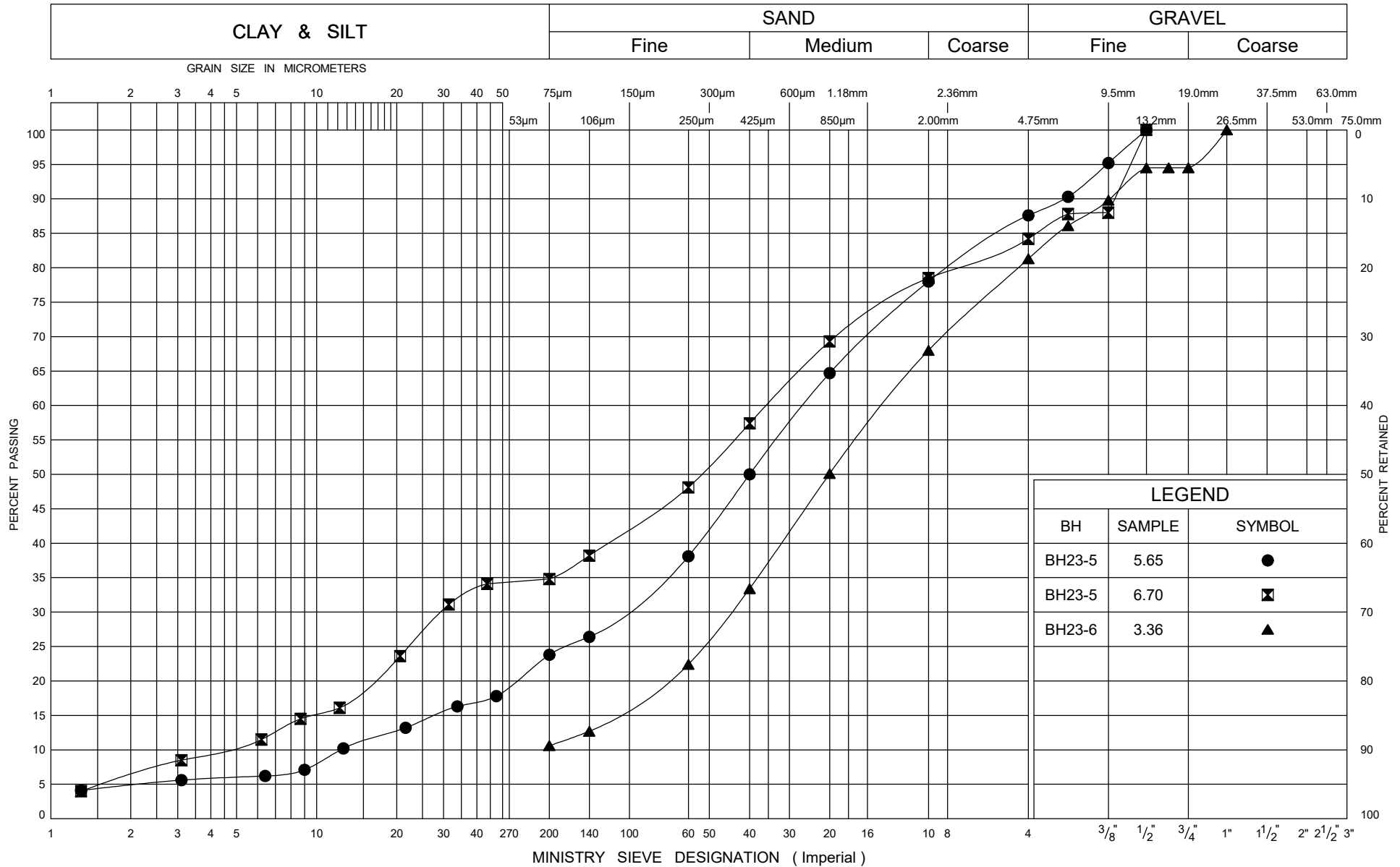


Photograph D6. Rock cores from BH23-6, Run 4 and Run 5 – 7.2 m to 9.4 m (October 2023)

UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM

Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

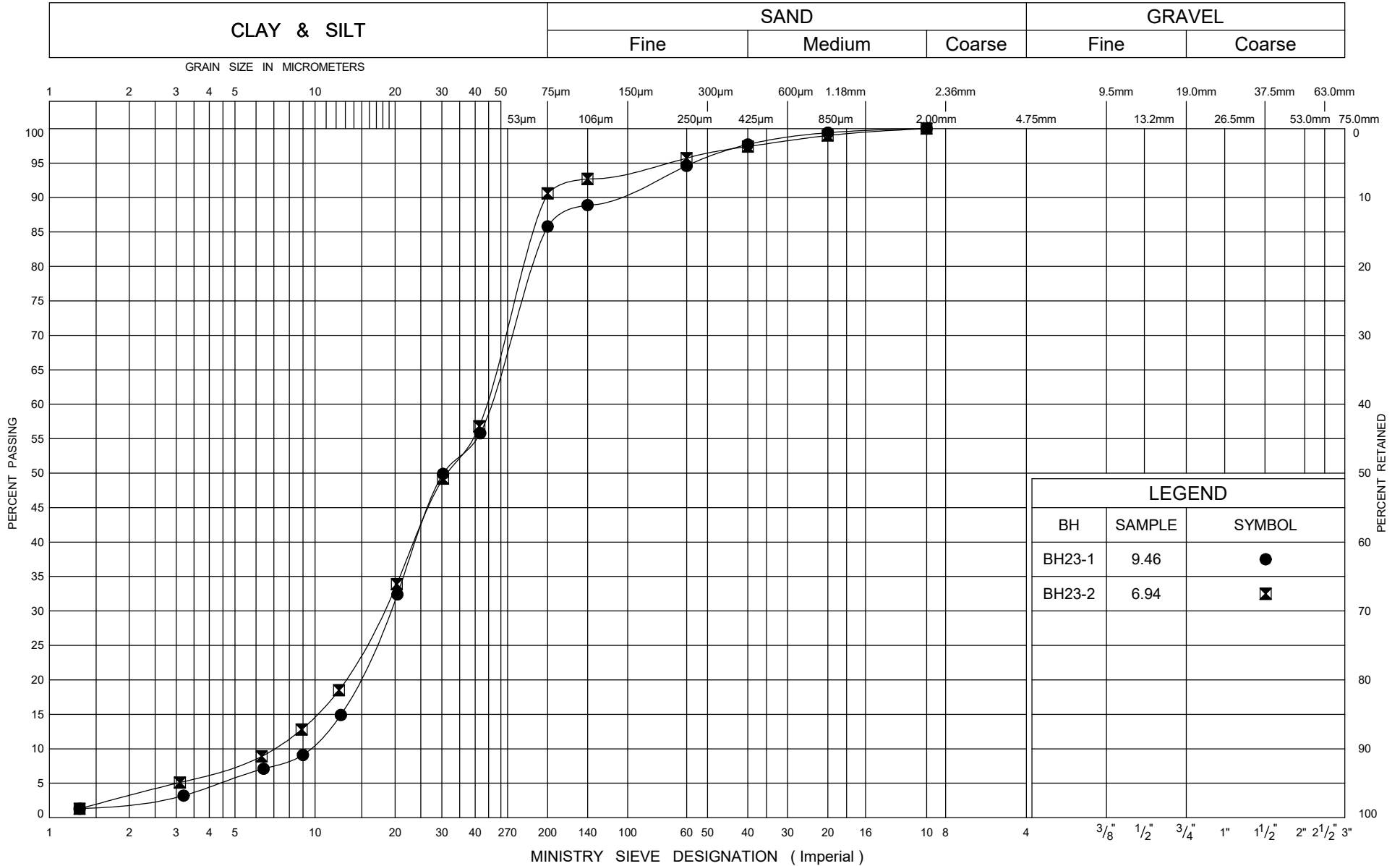
Sand (FILL)

FIG No 2

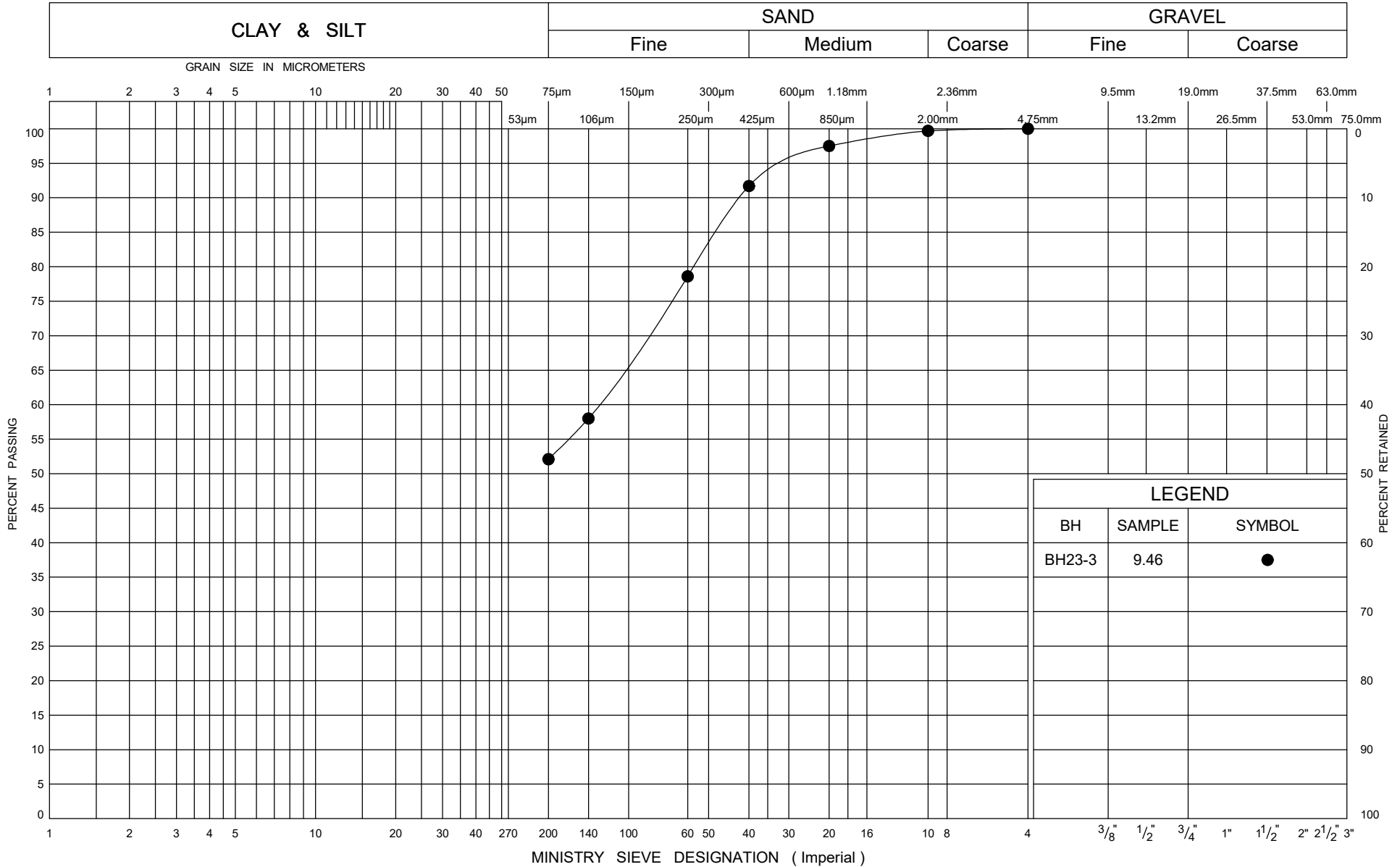
GWP No. 6109-17-00

6021-E-0019, Assignment 10

UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM

Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

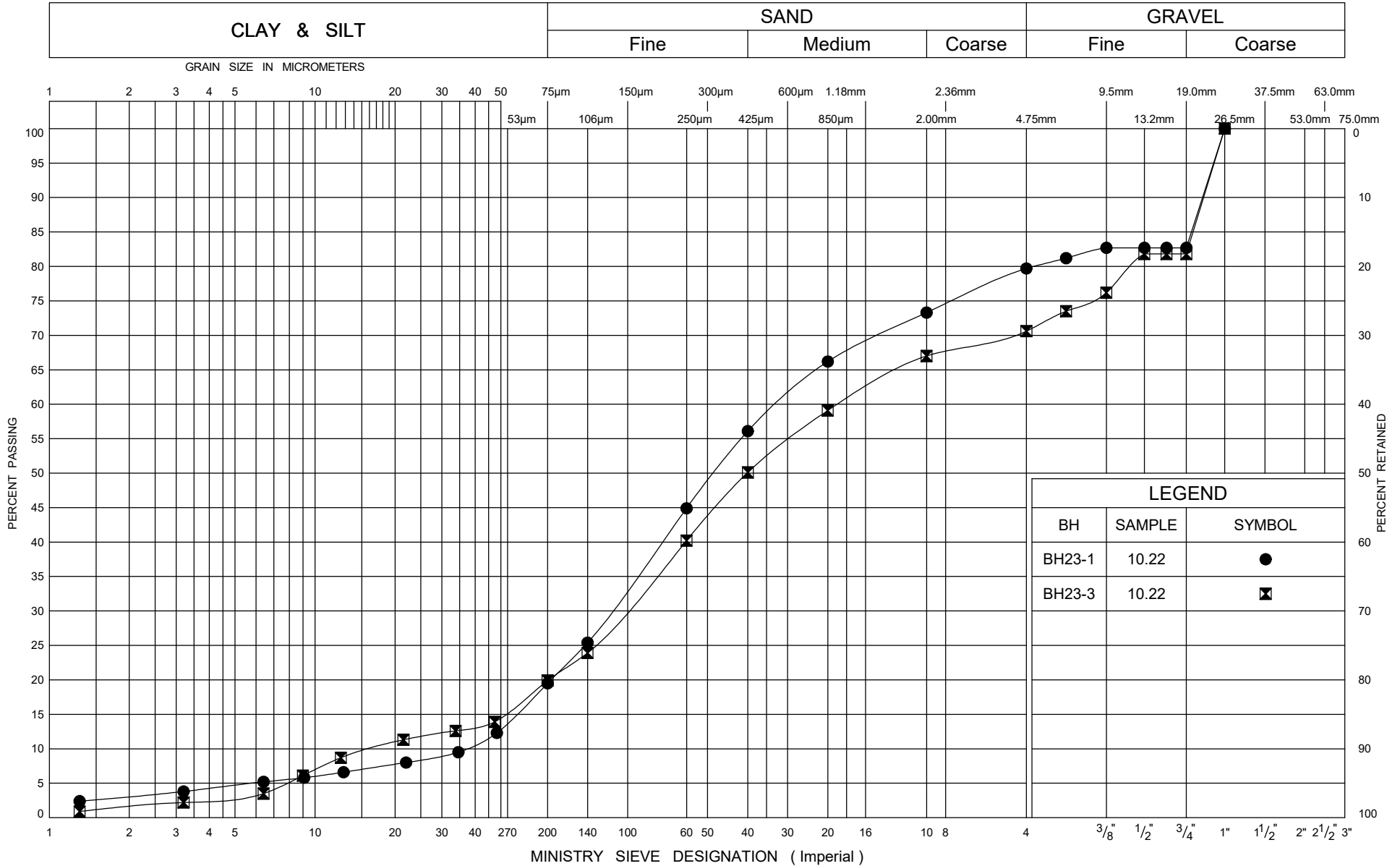
Silt and Sand

FIG No 4

GWP No. 6109-17-00

6021-E-0019, Assignment 10

UNIFIED SOIL CLASSIFICATION SYSTEM





BUREAU
VERITAS

Bureau Veritas Job #: C3W6139

Report Date: 2023/10/24

exp Services Inc

Client Project #: ADM-21019842-J0

Site Location: HWY 17 OVERPASS

Sampler Initials: KM

RESULTS OF ANALYSES OF SOIL

Bureau Veritas ID		XIO423		XIO424		
Sampling Date		2023/10/14 02:00		2023/10/15 03:00		
	UNITS	BH23-1 (S8)	RDL	BH23-3 (S4)	RDL	QC Batch
Calculated Parameters						
Resistivity	ohm-cm	560		360		8995535
Inorganics						
Soluble (20:1) Chloride (Cl-)	ug/g	1100	40	1600	100	8998694
Conductivity	mS/cm	1.8	0.002	2.8	0.002	8998914
Available (CaCl2) pH	pH	5.67		6.56		8999861
Soluble (20:1) Sulphate (SO4)	ug/g	75	20	58	20	8998703
RDL = Reportable Detection Limit						
QC Batch = Quality Control Batch						

Appendix E – Slope Stability Analyses

6021-E-0019 MTO Northwestern Region
 Assignment #20 - CPR at Hwy 17 Overpass Replacement at Martin, ON
 North Abutment w RSS Wall and 2H:1V Forward Slope
 Static/draind Condtions

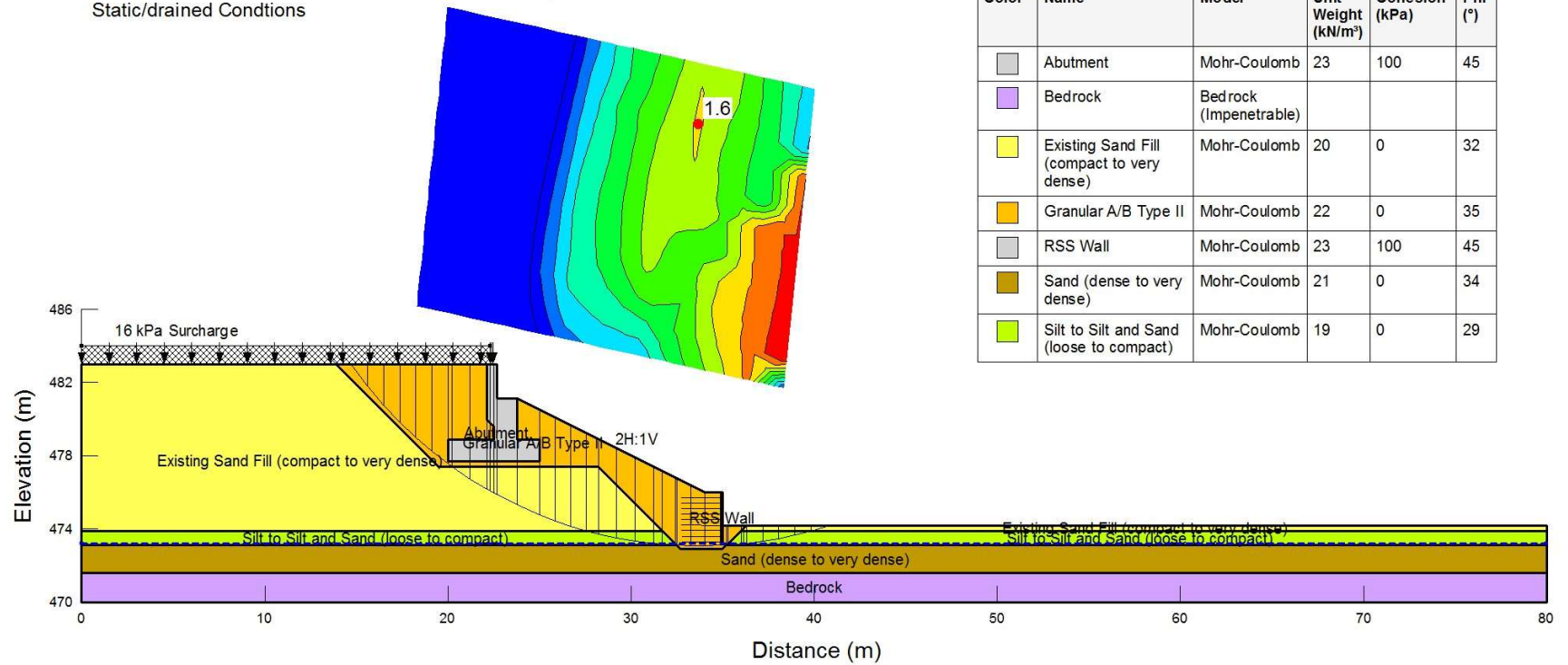


Figure E1. Slope stability Analysis at North Abutment Forward Slope – Drained Static Conditions

6021-E-0019 MTO Northwestern Region
 Assignment #20 - CPR at Hwy 17 Overpass Replacement at Martin, ON
 North Abutment w RSS Wall and 2H:1V Forward Slope
 Seismic/draind Conditions

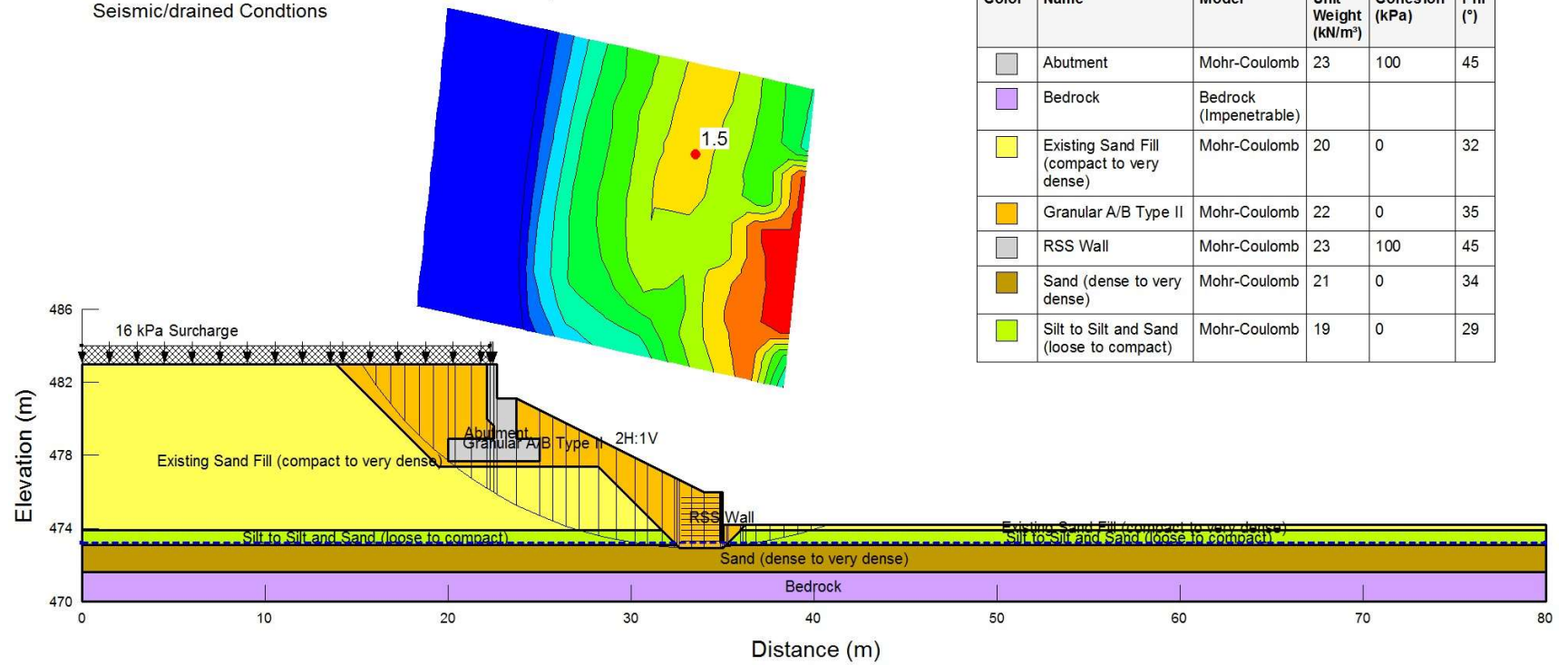


Figure E2. Slope stability Analysis at North Abutment Forward Slope – Drained Seismic Conditions

6021-E-0019 MTO Northwestern Region
 Assignment #20 - CPR at Hwy 17 Overpass Replacement at Martin, ON
 North Abutment w RSS Wall and 2H:1V Forward Slope (Failure in Front of Abut.)
 Static/draind Conditions

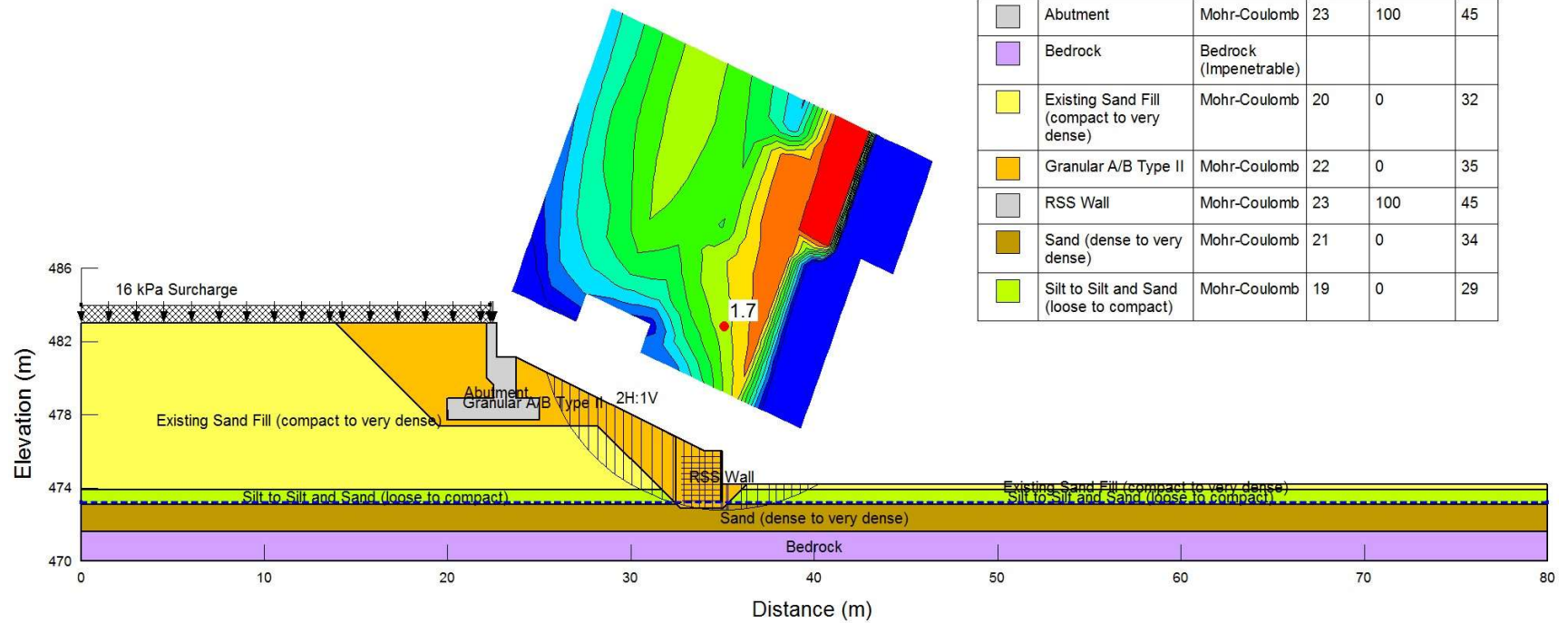


Figure E3. Slope stability Analysis for RSS Wall in Front of North Abutment Forward Slope – Drained Static Conditions

6021-E-0019 MTO Northwestern Region
 Assignment #20 - CPR at Hwy 17 Overpass Replacement at Martin, ON
 North Abutment w RSS Wall and 2H:1V Forward Slope (Failure in Front of Abut.)
 Seismic/draind Conditions

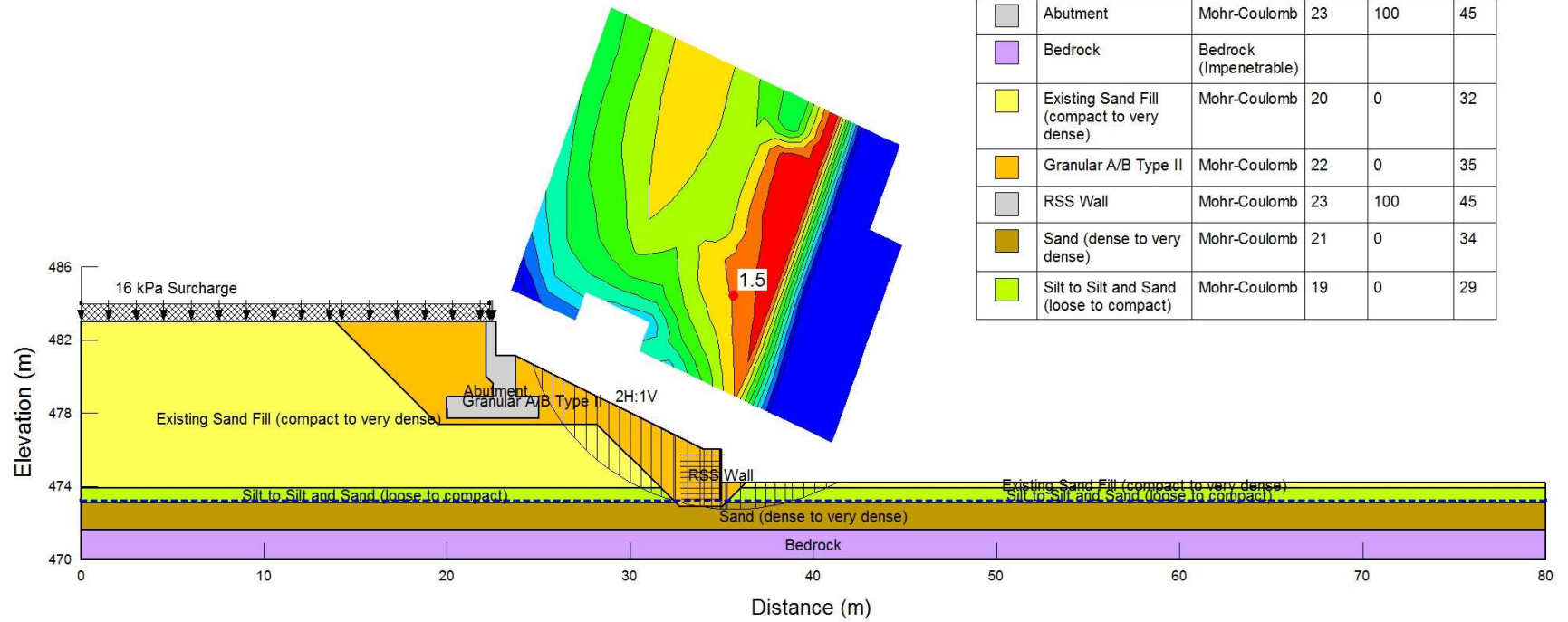


Figure E4. Slope stability Analysis for RSS Wall in Front of North Abutment Forward Slope – Drained Seismic Conditions

6021-E-0019 MTO Northwestern Region
 Assignment #20 - CPR at Hwy 17 Overpass Replacement at Martin, ON
 North Approach - New Embankment Fill at 2H:1V, West Side
 Static/draind Conditions

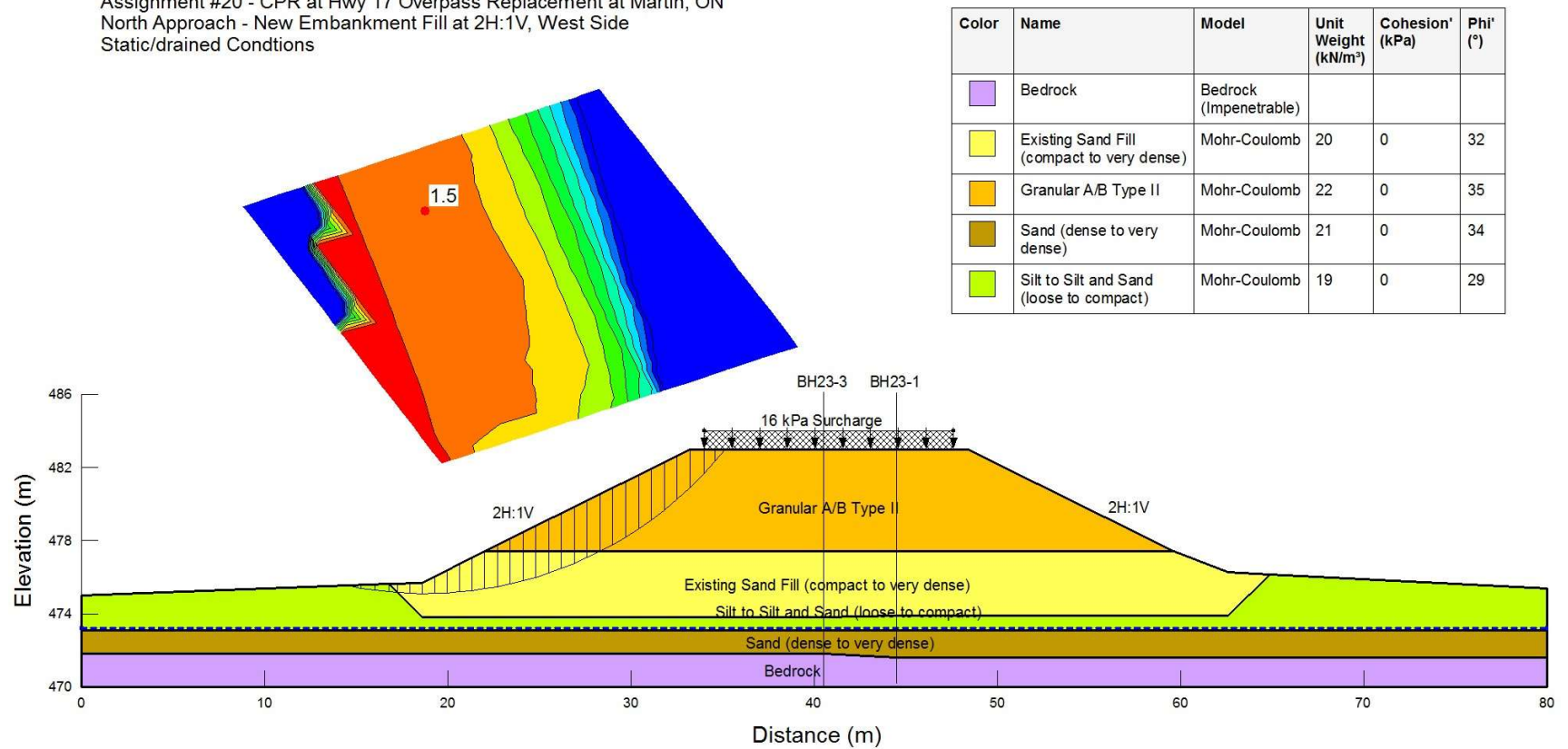


Figure E5. Slope stability Analysis for Approach Embankment Side Slope (West Side) – Drained Static Conditions

6021-E-0019 MTO Northwestern Region
 Assignment #20 - CPR at Hwy 17 Overpass Replacement at Martin, ON
 North Approach - New Embankment Fill at 2H:1V, West Side
 Seismic/draind Condtions

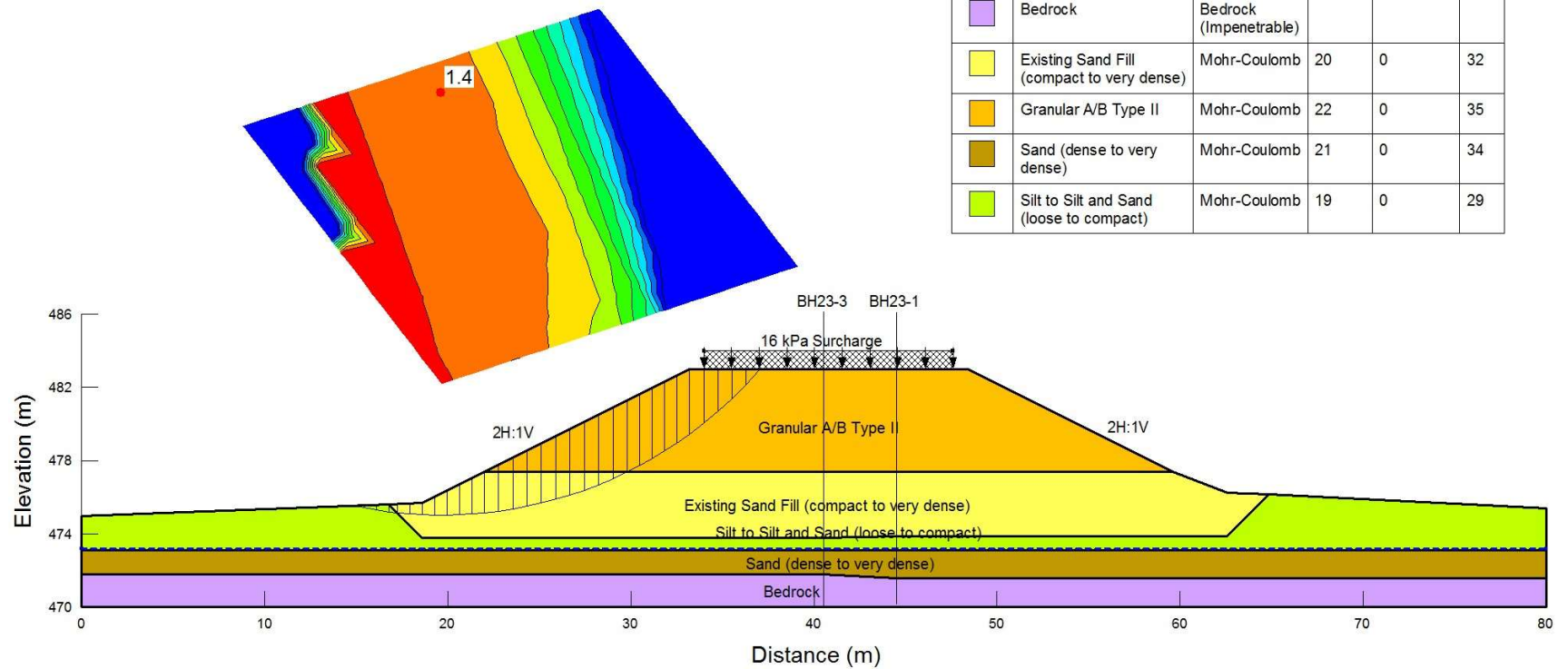


Figure E6. Slope stability Analysis for Approach Embankment Side Slope (West Side) – Drained Seismic Conditions

6021-E-0019 MTO Northwestern Region
 Assignment #20 - CPR at Hwy 17 Overpass Replacement at Martin, ON
 North Approach - New Embankment Fill at 2H:1V, East Side
 Static/draind Conditions

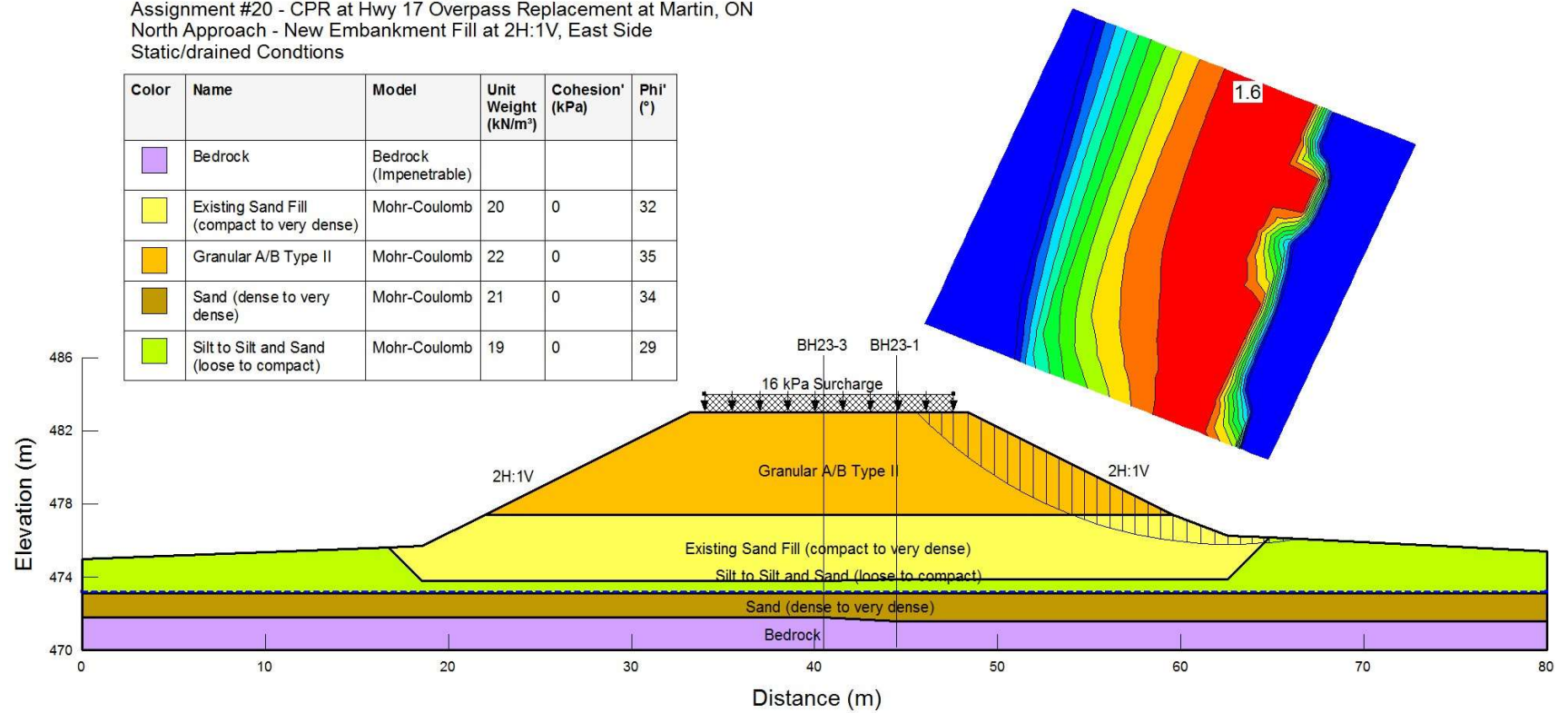


Figure E7. Slope stability Analysis for Approach Embankment Side Slope (East Side) – Drained Static Condition

6021-E-0019 MTO Northwestern Region
 Assignment #20 - CPR at Hwy 17 Overpass Replacement at Martin, ON
 North Approach - New Embankment Fill at 2H:1V, East Side
 Seismic/draind Condtions

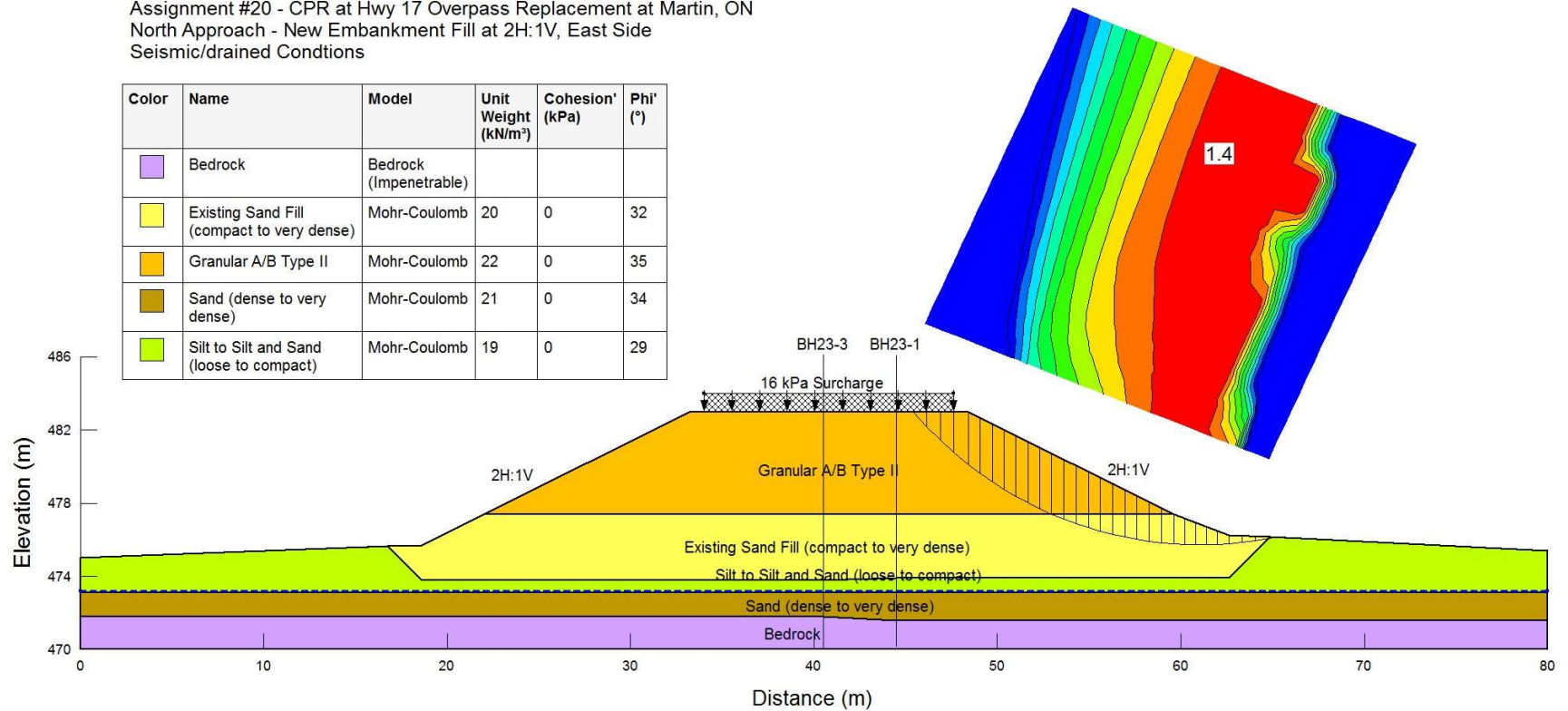


Figure E8. Slope stability Analysis for Approach Embankment Side Slope (East Side) – Drained Seismic Conditions

Appendix F – Seismic Hazard Calculation



Government
of Canada

Gouvernement
du Canada

Canada.ca › [Natural Resources Canada](#) › [Earthquakes Canada](#)

2020 National Building Code of Canada Seismic Hazard Tool

i This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_s	X_C
Latitude (°)	49.244
Longitude (°)	-91.073

Please select one of the tabs below.

NBC 2020

Additional Values

Plots

API

Background Information

The 5%-damped spectral acceleration ($S_a(T,X)$, where T is the period, in s, and X is the site designation) and peak ground acceleration ($PGA(X)$) values are given in units of acceleration due to gravity (g , 9.81 m/s^2). Peak

ground velocity. (PGV(X)) values are given in m/s. Probability is expressed in terms of percent exceedance in 50 years. Further information on the calculation of seismic hazard is provided under the *Background Information* tab.

The 2%-in-50-year seismic hazard values are provided in accordance with Article 4.1.8.4. of the NBC 2020. The 5%- and 10%-in-50-year values are provided for additional performance checks in accordance with Article 4.1.8.23. of the NBC 2020.

See the *Additional Values* tab for additional seismic hazard values, including values for other site designations, periods, and probabilities not defined in the NBC 2020.

NBC 2020 - 2%/50 years (0.000404 per annum) probability

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	PGA(X_C)	PGV(X_C)
0.109	0.0628	0.0288	0.0111	0.00222	0.000713	0.0558	0.0337

The log-log interpolated 2%/50 year $S_a(4.0, X_C)$ value is : **0.0033**

▼ Tables for 5% and 10% in 50 year values

NBC 2020 - 5%/50 years (0.001 per annum) probability

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	PGA(X_C)	PGV(X_C)
0.056	0.0323	0.0143	0.00521	0.000965	0.000306	0.0272	0.0162

The log-log interpolated 5%/50 year $S_a(4.0, X_C)$ value is : **0.0015**

NBC 2020 - 10%/50 years (0.0021 per annum) probability

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	PGA(X_C)	PGV(X_C)
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$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	PGA(X_C)	PGV(X_C)
0.0308	0.0179	0.00748	0.00255	0.000435	0.000132	0.0144	0.00837

The log-log interpolated 10%/50 year $S_a(4.0, X_C)$ value is : **0.0007**

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Date modified: 2021-04-06

Appendix G – NSSPs

NSSP FOR COBBLES AND/ BOULDERS OBSTRUCTIONS

Scope of Work

The Contractor shall be alerted to the potential presence of cobbles and boulders in fill and native soils as encountered in all boreholes advanced at the site. Therefore, appropriate equipment and procedures for piling for deep foundations/or for temporary shoring through these materials shall be selected.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision

Amendment to OPSS 902, November 2010

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means an electrical device that transfers power supply to a backup power source when there is an outage of the primary power source.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means a below grade wall that restricts groundwater flow and/or supports excavations, typically using soil-bentonite or cement-bentonite.

Design Storm Return Period means the average number of years based upon probability, between the occurrences of a storm event of a certain severity or greater.

Dewatering System means the components required to control water to permit construction work to proceed under specified conditions, and may include a groundwater control system, impermeable barriers, pumps, and/or equipment to carry out unwatering.

Groundwater Control System means sump pumps, oversized excavations with perimeter ditches, deep wells or well points or other systems used to lower the groundwater table.

Plug means an impervious, natural, or constructed drainage work that blocks water.

Sediment means soil particles detached from an earth surface by erosion.

Sediment Control Measure means a measure to remove sediment from water prior to discharge to the natural environment and sewer systems.

Temporary Flow Control means temporary flow control devices, channels, pipes, and other materials used to convey or divert water past an area under construction.

Unwatering means the removal of ponded or flowing surface water.

Vegetated Discharge Area means a sloped, open area of land with existing vegetation suitable to prevent erosion.

Waterbody means as any permanent or intermittent, natural or constructed body of water including lakes, ponds, wetlands and watercourses, but does not include sewage works as defined in the Ontario Water Resources Act.

Watercourse means a stream, creek, river, or channel including ditches, in which the flow of water is permanent, intermittent, or temporary.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

Subsections 902.04.01 and 902.04.02 of OPSS 902 are deleted in their entirety and replaced with the following:

902.04.01 Design Requirements

902.04.01.01 Dewatering

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work. The design of the system shall be sufficient to permit the work to be carried out as specified in the Contract Documents.

The design shall meet the requirements of the Contract Documents, and where a waterbody is present, shall include channel and inlet and outlet protection measures as required to protect the environment in the event of system failure or the design flow rate being exceeded.

The design shall not include the use of embankments and/or structures in public use, either existing or to be constructed as part of the Work, to control or stop water flow, unless approved by the Contract Administrator.

The design shall not result in displacement or damage to property, buildings, structures, utilities and other facilities adjacent to the Working Area, including from drawdown related settlement or other groundwater related effects.

The system shall be designed to prevent soil loss or erosion where water is removed, pumped, or discharged. The system shall be designed to prevent basal heave or instability.

Where the system involves the taking of water from a waterbody, the design shall maintain the flow of water and the natural functions of the waterbody upstream and downstream of the work area, and shall not interfere with other uses of the water.

When the system includes temporary flow control, the temporary flow control shall be designed, as a minimum, for a [* Designer Fill-In, See Notes to Designer] year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Temporary flow control shall include provision for fish passage during low flows.

902.04.02

Submission Requirements

902.04.02.01

Working Drawings

Three (3) sets of Working Drawings for the dewatering system shall be submitted to the Contract Administrator at least 7 Days prior to commencement of the dewatering system installation, for information purposes only. Prior to submission of Working Drawings, the seals and signatures of a design Engineer and a design-checking Engineer shall be affixed on the Working Drawings verifying that the drawings are consistent with the Contract Documents.

One person shall not perform both the design Engineer and design-checking Engineer roles for a system.

Where multi-discipline engineering work is depicted on the same Working Drawing and the design or design-checking Engineer or both are unable to seal and sign the Working Drawing for all aspects of the work, the drawing shall be sealed and signed by as many additional design and design-checking Engineers as necessary.

The following information and details shall be shown on the Working Drawings, where applicable:

a) Plans, Elevations, and Details

- i. Type of system(s).
- ii. Design calculations demonstrating adequacy of the system and equipment.
- iii. Design flow rate(s).
- iv. Plan location, description, and dimensions of system components, including dams, cofferdams, cut-off walls, temporary channels, pipes, culverts, sewers, groundwater control systems employing wells and/or well points, sedimentation basins, tanks, pumps, power supply, and standby equipment.
- v. Method of management of pumped water and plan location of all dewatering discharge points.
- vi. Profile drawings shall extend through and immediately beyond the limits of the system.
- vii. Water elevations upstream and downstream of the system at design flow rate.
- viii. Dam height or crest elevation, cofferdam depth and tip elevation, cutoff wall depth or base elevation, pipe invert elevations, depths of wells and wellpoints, pump intake elevation, and sedimentation basin depth or base elevation.
- ix. Plan location, elevation, and dimensions of environmental protection measures.
- x. Pipe type, size, and length, pump capacity, and tank capacity.
- xi. Material and construction standards to be used for the work.
- xii. Method for establishing and monitoring construction site groundwater levels.
- xiii. Criteria and method of removal of the system.

b) Procedures for the system construction, operation, and maintenance, including daily start-up sequence where applicable, and operation shut down.

c) Procedures for the removal of the system, including the removal sequence, and well decommissioning.

d) Stand-by power or pumping system requirements and the use of automatic transfer switching, when required to protect the environment and the Work.

e) A copy of the Permit to Take Water issued by the Ministry of the Environment and Climate Change or confirmation of registration of water taking for construction dewatering, if a permit or registration is required by provincial regulation.

f) When applicable, a copy of the water taking report and discharge plan required by provincial regulation.

- g) A copy of any necessary permits for the discharge of water to a sanitary sewer, or stormwater sewer system, stormwater pond, or other facility.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of [~~**~~ 100 m ~~or~~] metres from the groundwater control system. In addition, all ~~water wells used as a supply of drinking water~~ and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

The Quality Verification Engineer shall witness the following Interim Inspections of the work:

- a) Dewatering of excavation for structure.
- b) Completion of excavation for foundation.
- c) Excavation for backfill and frost tapers.
- d) Backfilling.

A copy of the written permission to proceed shall be submitted to the Contract Administrator prior to commencement of the successive operation.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation of temporary flow control, if applicable, shall be as specified in the Contract Documents.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When temporary flow control is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the temporary flow control during the seasonal shutdown period.

Temporary erosion and sediment control measures, including to control the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow control shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

Water from dewatering and unwatering operations shall be directed to a sediment control measure and/or a vegetated discharge area 30 m away from waterbodies or as far away as practicable from the top of the bank of any waterbody, prior to discharge to the natural environment.

Equipment and materials shall not be used or stored in vegetated discharge areas.

The discharge of water to the natural environment shall not be directed across pavements, sidewalks, curb and gutter or similar hard surfaces except through appurtenances as specified in the Contract Documents.

902.07.04.03 Monitoring

The Contract Administrator shall be notified of any complaints and any action taken or proposed to be taken in response to complaints.

Daily external visual monitoring of the surrounding area and property and structures on the preconstruction survey, if applicable, for impacts such as settlement and erosion shall be completed. Any observed impacts shall be immediately reported to the Contract Administrator. When public safety, the environment, or property is impacted or potentially impacted, the design Engineer shall, without delay, make a full assessment and direct changes to the system to eliminate impacts or potential impacts. Any changes shall be documented according to the System Amendments subsection.

When a groundwater control system is observed to negatively impact water supplies obtained from any adequate sources that were in use prior to groundwater control system operation, then water shall be supplied to the affected water users. The water shall be equivalent in quantity and quality to the normal water takings of the users. Supply shall continue until the negative impacts on the water supplies are removed, or until Contract Completion, whichever occurs first.

902.07.04.04 System Amendments

When displacement or damage to embankments and/or structures, or property adjacent to the Working Area, occurs due to the operation of the system, or soil loss or erosion occurs where water is removed, pumped, or discharged, the dewatering system or temporary flow control shall be amended to stop the displacement, damage, soil loss, or erosion.

Amendments shall be submitted to the Contract Administrator within two Business Days of the system being amended, on revised Working Drawings bearing the seal and signature of the design Engineer and design-checking Engineer.

902.07.04.05 Removal

Dewatering system and temporary flow control components shall be removed when no longer required. Removal of system components shall be according to the procedures specified on the Working Drawings, where applicable, and as specified in the Contract Documents.

Deactivation of temporary flow control shall be as specified in the Contract Documents.

Removal of temporary drainage work shall be according to OPSS 510.

Environmental protection measures and cut-off walls shall be removed, unless approved otherwise by the Contract Administrator.

Sedimentation basins and other excavations shall be backfilled with the original soil excavated, unless approved otherwise by the Contract Administrator. All disturbed areas shall be restored to an equivalent or better condition than existed prior to the commencement of construction.

NOTES TO DESIGNER:

Designer Fill-Ins

* Fill in the design storm return period according to MTO Drainage Design Standard TW-1.

** Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this item **only** on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.