

Final Foundation Investigation & Design Report (FIDR)

Highway 61 Culvert Replacement
Station 20+375, Township of Blake

Gannett Fleming
Ontario Ministry of Transportation (MTO)
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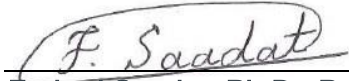
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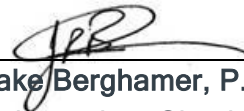


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1 Introduction

Englobe Corp. (Englobe) has been retained by Gannett Fleming (Client), on behalf of the Ministry of Transportation of Ontario (MTO, Owner), to carry out a foundation investigation and prepare Foundation Investigation (FIR) and Foundation Investigation and Design (FIDR) Reports for the proposed replacement of an existing culvert at approximate Station 20+375 on Highway No. 61 in the Township of Blake, Ontario (Site) shown on Drawing No. 1, Appendix A. This assignment was performed at the request of the Client as per the project Terms of Reference outlined in MTO Request for Quotation (RFQ) Version 3.2 under Assignment Number 6020-E-0021 (GEOCREs No. 52A00-264).

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2 Site Description

The existing 51.9 m long culvert structure is crossing Highway 61 at approximate Station 20+375, approximately 1230 m south of Moose Hill, in the Township of Blake. Highway 61 at this culvert crossing is a two-lane undivided highway with asphalt surface and partially paved shoulders on both sides running in an approximate north-south direction, as shown on Drawing No. 1 in Appendix A. Highway 61 is constructed on an embankment about 12.5 m wide (including shoulders) and up to approximately 9 m in height above the crown of the culvert, with the centreline of the roadway at an approximate elevation 236.9 m at the culvert location. The pavement surface is generally in good to fair condition with some longitudinal and transverse cracks across the asphalt surface. The topography of the surrounding area varies in the vicinity of the crossing. The sides of the roadway at the culvert crossing were observed to be heavily vegetated with bushes, shrubs, and mature trees. Minor bank and embankment erosion were also noted in GF Culvert Inspection Report. Existing overhead infrastructure observed at the culvert location included low hanging hydro line on both sides of the roadway.

The existing culvert consists of two sections of undermined length. The first section (upstream) is a 1.7 m wide and 1.2 m high Creosoted Timber Culvert (CTC) at the inlet, and the second section (downstream) is a 1.2 m diameter Corrugated Steel Pipe (CSP) at the outlet extending beyond the existing embankment slope, as shown on Drawing No. 2 in Appendix A and described in detail and shown on the sketches and Figures in Gannett Fleming (GF) Culvert Inspection Report in Appendix D. The existing CTC culvert at the inlet was observed to be deteriorated and deformed. The existing CSP culvert barrel at the outlet invert is corroded and its springline is very severely corroded and damaged. The channel dimensions were described by GF in general as 2 m wide channel upstream (US) with banks at ~3H:1V and water depth of 130 mm; and 5 m wide channel downstream (DS) with banks at 3H:1V, 400 mm of water. The top of the culvert elevations at the inlet and outlet are El. 227.3 and El. 226.9 m, respectively with clearance of 1030 mm and 822 mm, respectively. Flow through the culvert is from east/right (Rt) to the west/left (Lt) as shown on Drawing No. 2 in Appendix A.

2.1 Site Physiography and Surficial Geology

Based on published Northern Ontario Geology Terrain Study (NOEGTS) of the general area by J. F. Gartner, J.D. Mollard, and M.A. Road (1981), the Site is located within the Glaciolacustrine Plain with native overburden/sediments within the immediate project area consisting mainly of sand and silt deposits.



3 Investigation Procedures

3.1 Site Investigation

The purpose of the geotechnical investigation was to explore and record the subsurface conditions at both ends of the existing culvert and in the roadway embankment at the culvert crossing. The fieldwork was carried out between October 12 and November 12, 2021 and consisted of two boreholes on the roadway extending to a maximum depth of 22.3 m below existing ground/road surface (mbgs) and two boreholes off the roadway at the culvert inlet and outlet extending to a maximum depth of 6.7 mbgs.

The fieldwork included locating the boreholes, clearing the borehole locations of underground services, in-situ sampling and testing operations, logging of the boreholes, labeling and preparation of samples for transportation to the Englobe geotechnical laboratory in North Bay, plus overall drill supervision.

Englobe's staff visited the Site before the planned site investigation to mark out the selected borehole locations. Utility clearance was obtained from Ontario-1-Call. Public utility authorities were informed, and all utility clearance documents were obtained before the commencement of drilling work. A traffic control plan was prepared and implemented by Workforce Inc. of Sudbury, Ontario, according to Ontario Traffic Manual Book 7 during the fieldwork. The drilling rigs used for drilling were owned and operated by Maple Leaf Drilling Ltd. of Sunnyside, Manitoba. Boreholes were advanced using a CME 750 track mounted drill and a B20 portable drilling rig.

The fieldwork for this investigation included four (4) sampled boreholes (BH) were advanced. BH Nos. 1 and 2 were advanced in the roadway shoulders through the embankment. BH Nos. 3 and 4 were advanced at the inlet (Rt) and outlet (Lt) ends of the culvert, respectively. The locations of the boreholes are shown on Drawing No. 2 in Appendix A and are provided in the Table below.

Table 1 Borehole Locations

Borehole No.	Borehole Location (MTM Nad 83)		Borehole Location (Geographic)	
1	N 5345622.9	E 343135.5	Lat: 48.24823°	Long: - 89.483761°
2	N 5345622.4	E 343125.0	Lat: 48.24822°	Long: - 89.483902°
3	N 5345613.8	E 343160.2	Lat: 48.24815°	Long: - 89.484261°
4	N 5345623.1	E 343098.4	Lat: 48.24823°	Long: - 89.484260°

BH Nos. 1 and 2 were advanced using a hollow stem auger aided by track-mounted CME 750 drilling rig equipped with wash boring equipment, N-size casing, rock coring equipment (NQ size core) and

routine geotechnical sampling equipment. BH Nos. 3 and 4, which were drilled off the roadway near the inlet and outlet, were advanced using a B20 portable drilling rig equipped with hollow stem augers.

Soil samples were obtained at regular intervals of depth at the borehole locations using a standard 51 mm split spoon sampler advanced in accordance with the Standard Penetration Test (SPT) procedures ASTM D1586. All soil samples taken during this investigation were stored in labeled airtight containers for transport to the Englobe geotechnical laboratory in North Bay for visual examination and select laboratory testing. Bedrock was cored with NQ size coring equipment at BH No. 2.

Groundwater conditions in the open boreholes were observed during the advancement of the individual boreholes. Two 19 mm diameter standpipes were installed in Borehole Nos. 3 and 4 prior to backfilling to allow for follow-up monitoring of the stabilized groundwater levels. The remaining boreholes were backfilled upon completion of drilling in accordance with requirements of Ontario Regulation 903.

The location of the individual boreholes was determined in the field using highway chainage established by the Ministry of Transportation and offsets relative to highway centreline. The MTO co-ordinates, northing and easting, were then established for the boring locations using coordinates from MTM Zone 15, NAD 83 CSRS. Elevations contained in this report are referenced to an on-site geodetic datum. The borehole elevations are based on the GPS RTK survey carried out by Englobe.

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4 Laboratory Investigation

All soil and rock samples obtained during the investigation were transported to Englobe Laboratory in North Bay, Ontario. This laboratory is certified by the Ministry of Transportation Ontario (MTO) under RAQS program at Medium Complexity level for Soil and Rock Testing including Testing for Foundation Engineering. All retrieved samples were subjected to visual identification and tactile categorization to describe the soils. The laboratory tests to determine index properties were performed in accordance with the Ministry of Transportation Ontario (MTO) test procedures, which follow the American Society for Testing Materials (ASTM) test procedures. Laboratory testing consisted of grain size distribution; sieve and hydrometer analysis according to ASTM D422 and LS-702, Atterberg's Limits ASTM D4318 and LS-703/704, water content ASTM D2216 and LS-701. The results of the laboratory testing are presented on the individual Record of Borehole Sheets (Appendix B), with a summary of results presented on the laboratory sheets in Appendix C (Figures Nos. L-1 to L-8).

Chemical tests on one representative soil and one surface water sample to determine the soil and water corrosivity characteristics (pH, chloride, resistivity, sulphate) were carried out by an accredited independent laboratory (Bureau Veritas in Mississauga) to assess soil condition for buried structural steel and concrete elements.

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5 Subsurface Conditions

The subsurface conditions revealed by the investigation program are summarized in Table 2 below and on the stratigraphic profile presented on Drawing No. 2 (Appendix A) and on the detailed Records of Borehole Logs (Appendix B). It should be noted that the stratigraphic delineation presented on the borehole logs and soil strata plot is interpreted from the results of non-continuous sampling, response to drilling progress, recorded SPT 'N'-values, plus field observations. Typically, such boundaries represent transitions from one zone to another and are not an exact demarcation of specific geological units. Additional consideration should be given to the fact that subsurface conditions may vary markedly between adjacent boreholes and beyond any specific boring location and are shown on the drawings for illustration purposes only.

Table 2 Summary of Generalized Stratigraphy in Boreholes with Depth and Elevation (m)

Deposit/Layer Description	Depths/Elevations (m)			
	Borehole No. 1	Borehole No. 2	Borehole No. 3	Borehole No. 4
Asphalt/Topsoil	0.04 (El. 236.7 – 236.6)	0.04 (El. 237.1)	--	0 – 0.1 (El. 226.6 – 226.5)
Pavement Granular Base: Compact Sand and Gravel	0.04 – 0.8 (El. 236.6 – 235.9)	0.04 – 0.8 (El. 237.1 – 236.4)	--	--
Embankment Fill: Compact to Loose Sand/Silty Sand, some Gravel	0.8 – 4.6 (El. 235.9 – 232.1)	0.8 – 4.6 (El. 236.4 – 232.5)	--	--
Embankment Fill: Firm Silty Clay with traces of Sand and gravel	4.6 – 6.1 (El. 232.1 – 230.6)	4.6 – 6.9 (El. 232.5 – 230.3)	--	--
Embankment Fill: Compact to Loose Sand/Silty Sand, some Clay, trace Gravel, buried concrete and rock debris near bottom in Borehole No. 1	6.1 – 11.4 (El. 230.6 – 225.3)	6.9 – 10.7 (El. 230.3 – 226.4)	--	--
Native: Loose to Very Dense Native Clayey Silt /Silt	11.4 – 21.3 (El. 225.3 – 215.4)	10.7 – 12.2 (El. 226.4 – 224.9) and 18.3 – 20.9 (El. 218.8 – 216.2)	0 – 6.7 (El. 228.0 – 221.3)	0.1 – 5.3 (El. 226.5 – 221.3)
Native: Loose to Compact Sandy Silt/Silty Sand with trace Clay and Gravel	--	12.2 – 18.3 (El. 224.9 – 218.8)	-	5.3 – 6.7 (El. 221.3 – 219.9)
Bedrock or Inferred Bedrock	21.3 (El. 215.4)	20.9 – 22.1 (El. 216.2 – 215.0)	--	--

5.1 Asphalt and Pavement Structure

A thin layer of approximate 40 mm asphalt was observed in both BH Nos. 1 and 2 which were drilled on the shoulders through the embankment. The asphalt cover is underlain by a granular fill layer consisting mainly of brown sand and gravel, trace silt. The pavement structure/granular fill was observed to extend to approximately 0.8 m depth.

5.2 Topsoil

A thin topsoil layer (organic silt) of approximately 0.1 m thickness was observed in BH No. 4 only, located beyond the paved shoulder. No surficial topsoil was found in BH No. 3.

5.3 Embankment Fill

The composition of the embankment fill below the pavement structure at Borehole Nos. 1 and 2 varied with depth ranging from mainly loose sand/silty sand down to about 4.6 m depth to silty clay down to 6.9 m depth. Below the silty clay fill, another layer of silty sand fill was encountered extending down to maximum depth of 11.4 m (El. 225.3 m), that included buried concrete and rock debris in BH No. 1 down to El. 225.3 m

The embankment fill, immediately below the pavement structure, is mainly composed of brown sand/silty sand with variable portions of gravel and clay. This fill layer was approximately 4.3 m thick and extended down to approximate El. 232.1 m in Borehole No. 1. The moisture content of this upper fill layer varied from moist near top to wet (approximately 16%) near bottom. The results for grain size analyses of representative samples of two component soil types comprising the embankment fill are summarized in Table 3 and presented on Figure Nos. L-1 and L-2 in Appendix C.

Table 3 Particle Size Distribution Results of the Sand/Silt and Sand Fill

Sample Tested	Sample Depth / Elev. (m)	Grain Size Analysis (%)				Soil Classification
		Gravel	Sand	Silt	Clay	
BH No. 1 / SS-6	4.0 (232.7)	16	38	38	8	SM
BH No. 2 / SS-3	1.6 (235.5)	13	36	31	20	SC-SM

This sandy/silty fill layer was generally loose to compact, based on recorded SPT 'N' values ranging from 3 to 14 blows/300 mm.

The upper sand/silty sand fill was underlain by silty clay fill approximately 1.5 m to 2.3 m thick which extended down to approximately El. 230.3 m (lowest) in Borehole No. 2. The silty clay fill included minor portions of sand and gravel. It was reddish brown in colour with its in-situ moisture content measured at approximately 40%. A representative soil sample of this layer was subjected to grain size analysis and Atterberg limits tests (Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI)) and the results are summarized in Table 4 and presented on Figure Nos. L-3 and L-8, Appendix C. This silty clay fill layer can be classified as clay of high plasticity (CH).

Table 4 Particle Size Distribution and Atterberg Limit Results for Silty Clay Fill

Sample Tested	Sample Depth / Elev. (m)	Grain Size Analysis (%)				Atterberg Limits (%)			wc (%)	Soil Classification
		Gravel	Sand	Silt	Clay	LL	PL	PI		
BH No. 1/SS-8	5.6 (231.1)	1	2	39	58	55	29	26	40	CH

The consistency of silty clay fill varied from soft to firm with measured SPT 'N' values ranging from 3 to 8 blows/300 mm.

Below the silty clay fill layer, another brown sand/silty sand fill layer with different portions of clay and gravel was encountered, approximately 5.3 m and 3.9 m thick, and extending down to El. 225.3 m and 226.4 m in BH No. 1 and BH No. 2, respectively. This fill layer was damp near top to moist below with average moisture content of approximately 18%. Two representative soil samples underwent grain size analyses, and the results are summarized in Table 5 and presented on Figure Nos. L-2 and L-4 in Appendix C.

Table 5 Particle Size Distribution Results of the Sand/Silty Sand Fill

Sample Tested	Sample Depth / Elev. (m)	Grain Size Analysis (%)				Soil Classification
		Gravel	Sand	Silt	Clay	
BH No. 1 / SS-13	9.4 (227.3)	8	43	30	18	SC-SM
BH No. 2 / SS-11	7.8 (229.3)	17	70	13		SM

The sand/silty sand fill layer was observed to be loose to compact, based on measured SPT 'N' values ranging from 4 to 13 blows/300 mm.

Split spoon sampler and auger refusal were encountered in BH No. 1 at approximate El. 226.3 m on what was inferred to be buried concrete or rock debris. A piece of rock was retrieved in the split spoon sampler. Coring through the obstructions yielded no core recovery, and was followed by wash boring down to El. 225.3 m. Without core recovery, the frequency and size of these obstructions could not be determined. The buried concrete and/or rock debris, however, was not encountered in BH No. 2 and drilling with hollow stem augers and sampling continued through the embankment fill without any major obstructions encountered to approximately El. 226.4 m.

5.4 Native Clayey Silt/Silt (CL to ML)

Underlying the embankment fill in BH Nos. 1 and 2, and at surface in BH Nos. 3 and 4, a native fine-grained deposit of clayey silt/silt was encountered. The clayey silt/silt was also encountered in BH No.2 at approximate El. 218.8 m and extended down to El. 216.2 m, or top of the bedrock.

The native clayey silt/silt in BH No. 1 was approximately 9.9 m thick and was encountered at approximate El. 225.3 m and extended to auger and SPT spoon sampler refusal on inferred bedrock (El. 215.4 m).

The clayey silt/silt was encountered at the surface in BH No. 3 at approximate El. 228.0 m and extended to the maximum drilling depth (El. 221.3 m).

In BH No. 4, the clayey silt/silt was approximately 5.2 m thick and was encountered below the topsoil at approximate El. 226.5 m and extended to El. 221.3 m.

This deposit mainly consisted of low plasticity silt to clayey silt with varying portions of sand and gravel. Occasional boulders were encountered at a greater depth (below El. 218.0 m), suggesting probable till deposit close to the bedrock or inferred bedrock. The layer was observed to be brown to grey in general.

The natural moisture contents measured on samples recovered from the deposit ranged from 14 to 32%. Gradation analyses and Atterberg limits tests were carried out on four samples of this deposit, and the results are summarized in Table 6 and provided in Figure Nos. L-5, L-6, L-7, and L-8, Appendix C.

Table 6 Particle Size Distribution and Atterberg Limits Results of the Native Clayey Silt/Silty Sand

Sample Tested	Sample Depth / Elev. (m)	Grain Size Analysis (%)				Atterberg Limits (%)			Water Content (%)	Soil Classification
		Gravel	Sand	Silt	Clay	LL	PL	PI		
BH No.1/SS15	11.6 (225.1)	0	2	76	22	--	--	--	--	CL to ML
BH No.1/SS19	15.4 (221.3)	7	25	61	6	--	--	--	14	ML
BH No.2/SS16	11.6 (225.5)	12	25	46	17	27	17	10	16	CL
BH No. 2/SS22	18.5 (218.6)	1	11	81	7	--	--	--	18	ML
BH No. 3/SS2	0.9 (227.1)	--	--	--	--	28	18	10	--	CL
BH No. 3/SS4	2.6 (225.4)	4	11	69	15	--	--	--	32	ML
BH No. 3/SS-6	4.1 (223.9)	0	4	89	7	--	--	--	24	ML
BH No. 3/SS9	6.3 (221.7)	1	13	77	9	--	--	--	21	ML
BH No. 4/SS-3	1.7 (224.9)	7	19	45	29	35	22	13	27	CL
BH No. 4/SS-6	4.0 (222.6)	2	14	75	10	--	--	--	18	ML

The clayey silt/silt layer was observed to be loose to dense based on recorded SPT 'N' values ranging from 4 to 34 blows/300 mm. Higher SPT 'N' values (64 to 81 blows/300 mm) were encountered closer to the bedrock probably due to the presence of boulders and cobbles in the probable till.

5.5 Native Sandy Silt/Silty Sand (SM)

The sandy silt/silty sand was encountered in BH No. 2 between El. 224.9 m to El. 218.8 m. The layer was observed to be 6.1 m thick.

In BH No. 4, the sandy silt/silty sand was also encountered below El. 221.3 m and extended to the maximum depth drilling (El. 219.9 m).

This deposit mainly consisted of silt and sand with minor portions of clay and gravel. The layer was observed to be brown to grey in general. The natural moisture contents measured on samples recovered from the deposit ranged from 14 to 23%. Gradation analysis was carried out on one samples of this deposit, and the results are summarized in Table 7 and provided in Figure No.L-6, Appendix C.

Table 7 Particle Size Distribution and Atterberg Limits Results of the Native Sandy Silt/Silty Sand

Sample Tested	Sample Depth / Elev. (m)	Grain Size Analysis (%)				Atterberg Limits (%)			wc (%)	Soil Classification
		Gravel	Sand	Silt	Clay	LL	PL	PI		
BH No. 4/SS-8	5.6 (221.0)	2	58	41		--	--	--	23	SM

The sandy silt/silty sand layer was observed to be loose to compact based on recorded SPT 'N' values ranging from 7 to 14 blows/300 mm.

5.6 Bedrock/Inferred Bedrock

Auger and SPT spoon sampler refusals were encountered in two deep boreholes, namely BH Nos. 1 and 2. In BH No. 2, the refusal was encountered on confirmed bedrock at approximate depth of 20.9 m (El. 216.2 m) and the bedrock was cored and sampled from El. 216.2 m to 215.0 m using diamond core (NQ size) drilling. The cored bedrock was described as grey argillite-tuff Gunflint Formation. In

BH No. 1, the refusal was encountered at approximate depth of 21.3 m (El. 215.4 m) on suspected/possible bedrock, as no rock coring was undertaken.

Based on RQD values of 85 %, the bedrock was generally described as fair to excellent quality. Sampling in the bedrock was terminated at depths of 22.1 m (El. 215.0) in BH No. 2. Based on the elevation at which refusal on bedrock was encountered in BH's 1 and 2, the bedrock surface across the site is inferred to be relatively flat.

5.7 Groundwater Conditions

Groundwater and cave-in levels were measured in the open boreholes during the course of the fieldwork as summarized in Table 8. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix B).

Table 8 Groundwater Levels

BH No.	Drilling Date	Ground Surface Elve. (m)	Borehole Bottom		Monitoring Date	GW in Well		Monitoring Date	GW in Well	
			Depth (m)	Elve. (m)		Depth (m)	Elev. (m)		Depth (m)	Elev. (m)
BH No. 1	Oct 12-15, 2021	236.7	21.3	215.4	--	--	--	--	--	--
BH No. 2	Oct 15-18, 2021	237.1	22.1	215.0	--	--	--	--	--	--
BH No. 3	Nov 12, 2021	228.0	6.7	221.3	Nov 22, 2021	1.1	226.9	Dec 21, 2021	1.4	226.6
BH No. 4	Nov 10, 2021	226.6	6.7	219.9	Nov 22, 2021	1.8	224.8	Dec 21, 2021	2.6	224.0

The groundwater and surface water levels should be expected to fluctuate seasonally/yearly. It is noted that water was introduced into the borehole during drilling operations which may not have had sufficient time to stabilize. The stabilized groundwater level is anticipated to correspond with the creek water level. The lowest creek level is anticipated to be above the average invert elevation of the culvert at elevation 225.9 m.

5.8 Soil Corrosivity Testing

A representative soil sample collected from BH No. 2 and water sample collected from BH No.3 were subjected to corrosivity chemical tests by Bureau Veritas Laboratories in Thunder Bay to determine its potential corrosivity by measuring resistivity, pH, sulphate and chloride content of the sample within the estimated infrastructure depths. The results are presented in Table 9.

Table 9 Soil and Water Corrosivity Chemical Analysis Results

BH No.	Sample	Depth (Elev.) (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (Ohm-cm)
BH No. 2	SS-15	11 (226.7)	7.17	0.0310	0.0180	1500
BH No. 3	Water	--	7.22	0.0012	0.0068	--

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6 General Comments

The field investigation was carried out using track mounted CME 750 drilling rigs and a portable B20 drilling rig owned and operated by Maple Leaf Drilling Ltd. Laboratory testing of select soil samples was undertaken at the Englobe Laboratory in North Bay. The fieldwork for this site investigation was under the full-time supervision of Englobe technical staff. The report was written by Mr. Farbod Sadaat, Ph.D., P.Eng., and peer reviewed by Mr. Ala Abu Obeid, M.Sc., P.Eng., PMP. The report was also reviewed by the MTO Designated Contact Mike Tanos, P.Eng., with independent review by Jake Berghamer, P.Eng.

7 Foundation Design Recommendations

7.1 General

This part of the FIDR report presents recommendations for the design and installation of the new culvert at Station 20+375 as per MTO Guideline for Foundation Engineering Services (V2, October 2020). It is solely for the use of Gannett Fleming (GF) for the detail design of this specific project on behalf of the Ministry of Transportation and shall not be used for any other purposes or by any other parties including the construction Contractor.

Where comments are made on construction, they are provided solely to identify aspects that could affect the design of the project. Construction contractors should make their own assessment of the factual information provided in the FIR for their decisions related to construction including, but not limited to, equipment selection, proposed construction methods and scheduling.

7.2 Project Description

Based on the subsurface conditions described in Section 5, the existing paved roadway embankment is about 12.5 m wide (including shoulders) and approximately 9 m high above the crown of the existing culvert. The stratigraphy consisted of variable embankment fill over native silt/silty sand deposit underlain by bedrock. The embankment fill consisting of mainly sand and silt, some gravel with some clayey interlayers and buried rock and concrete debris near the bottom of embankment, as shown on Drawing No. 2 (Appendix A).

Based on the results of hydraulic analysis and sizing assessment carried out by Gannett Fleming, the proposed replacement culvert will be a new 2.4 m diameter CSP, to be installed at the same invert elevation as the existing culvert. As per the project Terms of Reference (ToR), it is assumed that the culvert will be replaced with either open-cut techniques along the same alignment or trenchless techniques adjacent to the existing culvert alignment. The new alignment can be on either side of the existing culvert

7.3 Evaluation of Alternatives

Based on the anticipated geotechnical conditions along the proposed replacement culvert at Highway 61 Station 20+375, the following potential construction methods can be considered.

Table 10 Construction Methodology Alternatives - Advantages and Disadvantages

Options	Description	Advantages	Disadvantages	Risks/Consequences	Relative Costs
1	Trenchless Techniques	<ul style="list-style-type: none"> • Minimum disruption to traffic • Avoids a large excavation through the existing 	<ul style="list-style-type: none"> • Requires entry and exit pits involving potential dewatering 	<ul style="list-style-type: none"> • If cobbles/boulders are encountered, they can stop penetration • Integrity of existing embankment may be affected due to induced settlement and vibrations 	<ul style="list-style-type: none"> • High

Options	Description	Advantages	Disadvantages	Risks/Consequences	Relative Costs
		highway embankment • Staged construction will not be required		from the trenchless operation • Loss of strength or liquefaction of the underlying silt with high water table that is susceptible to disturbance under dynamic loading during vibration inducing by trenchless operations • Deviations in specified alignment • Possible ground loss/heave in the soils above the crown and/or below finished pavement	
2	Open Cut with Full Road Closure and Temporary Detour	• Allows for an expedited construction schedule • Reduces costs associated with roadway protection and groundwater control comparing to option #3 and #4 but not comparing to option #1	• Necessary detour route may not be available/practical • Requirements for property acquisition and access • Disruption to traffic	• Destabilization of the highway embankment due to excavation through saturated cohesionless soil • May require water flow realignment • Detour construction- may need property acquisition	• low
3	Open Cut with Staged Temporary Widening	• May be less expensive than trenchless methods, (subject to site physical constraints) • Reduces costs associated with roadway protection and groundwater control comparing to option #4 but not comparing to option #1	• Longer schedule to build the widening • Requirements for property acquisition and access • Settlement under the footprint of the embankment widening as well as the existing embankment. • Disruption to traffic	• Destabilization of the highway embankment due to excavation through saturated cohesionless soil • May require roadway protection • May require water flow realignment • Widening construction- may need property acquisition	• Medium
4	Open Cut with Staged Replacement and Temporary Protection System	• May be less expensive than trenchless methods	• Longer schedule due to staged construction • Will require extra cost for roadway protection • Single lane of traffic flow	• Destabilization of the highway embankment due to excavation through saturated cohesionless soil • May require water flow realignment • longer construction time	• Medium

Based on the existing subsurface conditions and the list of advantages and disadvantages of all options considered for this culvert replacement as indicated in the above table, the open cut methods will involve extensive construction activities for excavation, shoring, and dewatering. The major

advantages of the open cut methods are possibility to assess the foundation soil below the new culvert location and to remove the existing culvert. The major disadvantages of the open cut methods are disruption of traffic, large excavation (>11 m high), extensive dewatering, requirements for property acquisition and access, longer schedule due to staged construction, and construction risk associated with saturated cohesionless soil encountered at this site. On the other hand, the option of trenchless techniques will allow the installation of culvert in a less disruptive manner for this site. However, the major disadvantages for trenchless techniques at this site are the potential for induced settlement and vibrations from the trenchless operation which may cause instability for the existing embankment.

Construction methodology that minimizes dewatering and excavation would be preferable from a foundation engineering perspective. Option selection will also depend on the construction staging and traffic interruption constraints, the hydraulic capacity and size of the existing and proposed culvert and other considerations.

It is understood that the design team is currently evaluating the feasibility of different culvert replacement methods. However, considering the significant height of the existing embankment (in excess of 11 m relative to culvert invert elevation) the trenchless installation method is preferred subject to budget restriction.

From a foundations engineering perspective, the trenchless techniques (Alternative #1) are feasible for this site and generally have the advantage of minimum disruption to traffic, would minimize potential dewatering and would avoid a large excavation through the existing highway embankment. However, the potential presence of occasional buried rock and concrete remains in the fill near the proposed tunnel alignment and/or potential presence of saturated cohesionless soils at this site near the invert elevation of the culvert may impose some potential challenges for the installation of culvert via trenchless techniques. Trenchless techniques would generally be favored provided that the integrity of existing embankment will be maintained during trenchless operation, and the size of the buried rock and concrete debris near the bottom of the embankment be less than 200 mm (limited to cobble size obstructions of rock and fragments of concrete debris). To confirm the size of the rock and concrete debris obstructions in deep fill material additional auger probes and/or excavations of the foreslopes of the embankment on both sides in the vicinity of the culvert are recommended in advance of trenchless installation to facilitate the proposed installation. In addition, the trenchless operations will mainly occur within and in the vicinity of the existing culvert bedding and cover, which could potentially minimize the likelihood and the risk of encountering buried rock and concrete debris obstructions.

The alternative to replace the existing culvert with a precast segmental closed box culvert using open cut techniques with staged construction (Alternative #4) can be also considered as a feasible culvert replacement option from a foundation engineering perspective. The closed box culvert will be less susceptible to differential settlements and will be most appropriate for the relatively weak foundation soil at this site. However, this option will require roadway protection installed along the embankment centerline to maintain a single lane of traffic flow along the current highway alignment.

7.3.1 Trenchless Techniques Option

7.3.1.1 Tunnelman's Ground Classification System

A framework for describing soil behaviour in a tunnel heading (face) has been developed over the years. Initially developed by Terzaghi in 1950, the system was modified by Heuer in 1974. Known as the Tunnelman's Ground Classification System, it provides an understanding of how different soils may behave in an unsupported tunnel face under atmospheric conditions. This system is also a powerful tool for evaluation of the soft ground conditions applicable to this project. Table 11 below provides a general description of various ground performances.

Table 11 Tunnelman's Ground Classification for Soils

Classification		Performance	Typical Soil Types
Firm		Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.	Loess above the water table; hard clay, marl, cemented sand, and gravel when not highly overstressed.
Raveling	Slow Raveling	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed. This is caused by loosening or overstress and "brittle" fracture (ground separates or breaks along distinct surfaces, as opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes; otherwise, the ground is slow raveling.	Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon the degree of overstress.
	Fast Raveling		
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow are caused by overstress.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination with raveling at excavation surface and squeezing at depth behind surface.
Running	Cohesive-Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose /from ± 30 to 35 degrees). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry granular materials. Apparent cohesion in moist sand or weak cementation in any granular soil may allow the material to stand for a brief period of raveling before it breaks down and runs, such behavior is cohesive-running.
	Running		
Flowing		A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls and can flow for great distances, completely filling the tunnel in some cases.	Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly preconsolidated clay with a plasticity index in excess of about 30, generally containing significant percentages of montmorillonite clay.

Considering the proposed longitudinal cross section of the new culvert, the tunneling horizon will be between elevation 225.7 m and elevation 228.1 m. Therefore, it is expected that the vast majority of tunneling excavation will be performed through a mixed face of variable loose to compact silty sand/sand fill with occasional inclusions of rock and concrete debris (embankment fill) and native loose to compact, moist to wet silt to silty sand deposit. Therefore, the tunnel horizon is generally expected to be located within saturated soils.

Based on Tunnelman's Ground Classification System, most of the silty sand fill encountered at the tunneling face will have a tendency to flow. Therefore, tunnel face instability caused by flowing soils would be generally expected. Any excavation within unsupported tunnel face shall be done in a manner to control groundwater inflow to the excavation and to prevent loss of ground into the excavation.

7.3.1.2 Alternative Trenchless/Tunnelling Techniques

The boreholes through the embankment indicates that the embankment fills consisted primarily of sand and silt, some gravel with some clayey interlayers and buried rock and concrete debris near the

bottom encountered within the existing embankment fill in Borehole No. 1. The major impediment to trenchless techniques at this site could be potential presence of rock pieces and concrete debris within the fill that may cause issues during trenchless operation. Therefore, to accommodate such possibility a Non-Standard Special Provision (NSSP) regarding potential obstructions in the subsurface soils has been provided in Appendix E. In addition, the potential presence of water bearing silty sand near the invert elevation of existing culvert must be taken into consideration for potential installation via trenchless methods.

A significant challenge for any trenchless technology technique is the potential for presence of obstructions along the proposed culvert alignment around the elevations of the pipe invert and/or springline. Should significant obstructions be present around the existing culvert, it would be very difficult or challenging to maintain the tunnel vertical alignment when advancing the tunnel. As such, it would be prudent to investigate the potential presence and frequency of such potential obstructions along the full length of the culvert using geophysical techniques or closely spaced obstruction probes to ensure that the culvert can be installed at the correct elevation and not diverted by the obstructions. Depending on the culvert elevation and alignment, the installation technique shall consider the potential difference in soil conditions at the interfaces along the proposed alignment and the Contractor should be prepared to advance culverts through the multiple fill and native soil types and/or potential obstructions.

A trenchless construction approach for the proposed approximately 52 m long and 2.4 m diameter culvert replacement, if selected, would eliminate the need for open cuts and/or roadway protection systems, and associated traffic delays on the existing road during construction.

The following table summarizes the general advantages and disadvantages of the different trenchless techniques for consideration from the foundation engineering perspective at this project.

Table 12 Trenchless/Tunnelling Techniques- Advantages and Disadvantages

Method	Advantages	Disadvantages
Jack and Bore involving Hydraulic Jacking of Casing Controlled by Laser Beam and Navigation System at Surface Directing Machine Cutter Head	<ul style="list-style-type: none"> • Good contractor availability • Good for shorter tunnel lengths (less than 100 m) • Good gradient control • Although new pipe size of 2.4 m exceeds the commonly used size by local contractors, the proposed larger boring diameter could potentially allow removal of existing culvert and occasional obstructions such as cobbles/boulders/concrete debris if they could be swallowed • Would permit personal entry, if needed for >1.2 m pipe size 	<ul style="list-style-type: none"> • Requires construction entry and exit pits with groundwater control/dewatering • Elevated potential for ground subsidence • Not well suited for use in rock fills or if there is potential for high concentration of large obstructions • Not generally recommended where subsurface conditions indicate that saturated gravel, sand and silt soils may be encountered at pipe level or within one pipe diameter above or below outside pipe dimensions
Pipe Ramming using Compressed Air Pneumatic Rammers Attached to the Rear of Steel Casing	<ul style="list-style-type: none"> • Minimal groundwater control required along the installation route (unless required to remove obstruction/old pipe) • Casing advancing with repeated percussive blows swallowing face cutting into the casing facilitated by cutting shoe to minimize ground displacements • Can penetrate soils containing cobbles/boulders if obstruction less than casing diameter • Can accommodate pipe diameter up to 3.6 m 	<ul style="list-style-type: none"> • Installation problems may occur in soils with large cobble/boulders or other potential obstructions without an active navigation system • Requires staging construction pits • Groundwater control may be required at construction entry and exit pits • Possible ground displacement/heaving in the soils above the crown, so this method may not be recommended for shallow cover • A non-steerable system used could make it difficult to control line and grade of the installation

Method	Advantages	Disadvantages
	<ul style="list-style-type: none"> • Would permit personal entry, if needed for >1.2 m pipe size 	
Down-the-Hole (DTH) percussive hammer	<ul style="list-style-type: none"> • Can drill through soil and rock at the same time. • Requires a small amount of water to facilitate the drilling. • Cause less ground disturbance. • More cost-effective comparing with other trenchless methods, particularly when rock or rock fill is anticipated. • Low torque is needed which in turn means less time steering to keep the hammer on course (i.e. the hammer tends to stay on course and drill a straight hole). 	<ul style="list-style-type: none"> • Local Contractor availability could be an issue • Maximum pipe diameter for this method is only 48" (1.2 m) which is smaller than the required size of replacement culvert of 2.4 m
Horizontal Directional Drilling (HDD) or Pipe reaming (modified back reaming)	<ul style="list-style-type: none"> • Can be used in most ground conditions • Does not generally require deep staging pits therefore minimal ground water control required • Alignment may be adjusted to avoid minor obstructions 	<ul style="list-style-type: none"> • Proposed 2.4 m pipe diameter exceeds the commonly used upper size limit for HDD • Larger drilling equipment is expected to be required • Requires drilling fluid to maintain the bore, which could result in heave and may warrant potential environmental impact assessment • The required laydown area for pullback may extend beyond the highway right-of-way
Microtunneling and Conventional Tunneling	<ul style="list-style-type: none"> • Precise, guided remote-controlled method of jacking pipe behind a Microtunnel boring machine (MTBM) • Earth is continuously supported at the face (earth pressure balance) • Pipes of all sizes, typically in long runs • The existing culvert can be used to maintain the surface water flow during the construction • Minimum surface disruption 	<ul style="list-style-type: none"> • Excavation required for Launching and Receiving Pits. • Elevated cost for small jobs. • No access during tunneling

It is important to note that “the selection of trenchless installation method is the responsibility of the Contractor”, as per Section 6.3.1 of MTO 2020 Guideline for Foundation Engineering Services. In general, compared to the other trenchless methods, pipe ramming is deemed to be the most feasible trenchless installation method at this site from a foundation engineering perspective. Other trenchless methods such as Jack and Bore with or without Motorized Small Boring Units (SBU-M) have higher risk levels due to the high potential for ground loss and the difficulties and challenges associated with potential obstructions. Micro-tunnelling and tunnelling would also face potential difficulties and challenges associated with potential obstructions. The proposed 2.4 m pipe diameter exceeds the commonly used upper size limit for Down-the-Hole (DTH) percussive hammer and HDD methods.

Pipe ramming is feasible in a wider variety of soils including cobbles and boulders, stable (nonflowing) and unstable (flowing) ground conditions. Boulders and obstacles of considerable size can be traversed, as long as their diameter is smaller than that of the pipe culvert or they are broken up by the ramming action of the pipe. When using open faced ramming, the spoil moves steadily into the cavity of the pipe, reducing potential for damage, deviations in alignment, creation of void, and surface disruptions. In addition, pipe ramming restricts the pipe material to steel due to the installation forces. However, the advantages of using steel tend to outweigh the disadvantage of the need for high strength material. The use of steel minimizes the damage to the leading edge of the culvert. A

reduction in the damage to joints is also achieved through the use of steel. If boulders or cobbles are encountered, directional changes are less likely than with other materials. Steel joints are more rigid and allow even distributions of thrust forces from one section to another. In addition, the service life of steel is greater than other materials such as reinforced concrete. Furthermore, pipe ramming can accommodate up to 3.6 m pipe diameter and length up to 122 m which are generally considered very favorable for this site.

With pipe ramming, a rigid high strength steel casing is expected to be used to replace the existing culvert along a new alignment that is adjacent to the existing culvert. The pipe ramming potential adverse impacts on the stability of the existing embankment is a risk that should be considered, but it can be properly addressed/mitigated by careful selection of appropriate ramming energy. In addition, it is highly recommended that a contingency mitigation plan should be prepared for the trenchless approach at this site which identifies and implements actions to deal with any potential trenchless installation stoppage and/or potential construction obstruction issues. In addition, the water at the inlet and outlet and around the proposed entry and exit pits for trenchless installation would require cofferdams, as discussed in Section 7.9 below. Diversion of the upstream and downstream channels will be provided later during development of 60% design.

7.3.1.3 Trenchless Installation and Typical Costs

Given the subsurface conditions and the anticipated challenges associated with various trenchless installation methods in general from the foundation engineering perspective (discussed above), selection of an experienced Tunnelling Contractor with proven trenchless installation track record during tender is of paramount importance. Table 13 below presents rough estimates of anticipated tunnelling costs based on similar MTO trenchless installation projects in the past in North-Western Ontario to assist the project management team (MTO and their prime consultant) in the tendering process and evaluation of the bids from pre-qualified Tunnelling Contractors/proponents.

Table 13 Estimated Costs of Trenchless/Tunnelling Installation Methods

INSTALLATION METHOD	TYPICAL RANGES OF TUNNELLING COST
Pipe Ramming with a percussive hammer head to break the cobble sized rock pieces and concrete debris	Estimated Cost for conventional pipe ramming with diameters less than 2.5 m can ranged between \$10,000/m to \$20,000/m
Microtunneling with Cutting head which can cut cobble sized rock pieces and concrete debris	High cost- linear unit cost varies and can ranged between \$ 25,000/m to \$40,000/m

Pipe ramming will include driving an open-ended steel casing with a percussive hammer. After the pipe has been driven, the soil in the pipe will be removed by using water pressure, air pressure, or scrapers, depending on accessibility. A heavier wall steel casing should be utilized for the pipe ramming (more robust with increased wall thickness), making obstructions such as occasional inclusions of rock/concrete debris in fill more tolerable.

The pipe ramming hammer shall be capable of driving the pipe casing from the entry pit to the exit pit through the existing subsurface conditions at the site without removal of soil from within the casing until the lead end of the pipe is outside the zone of influence for any overlying infrastructure. A slurry of water and bentonite may be applied to the leading edge to help reduce friction.

Due to the presence of rock and concrete debris material being buried near the bottom of the embankment fill, the new steel pipe may be driven through some abrasive soils. One means of minimizing the resistance of abrasive soils along the tunnel drive is to attach a driving shoe to the pipe before driving. In addition, it would be prudent to use a high-grade steel and increased the wall thickness/section for the pipe to ensure that the casing is not deformed during driving. The casing shoe will reduce frictional drag around the pipe, reinforce the leading edge to assist in the breakup of debris, resist the hoop stress created from uneven stress distribution due to any boulders, direct spoil into the pipe and promote better directional control for pipe ramming. Casing shoes can be also designed to incorporate the release of the lubrication/slurry. If partial removal of soil within the casing is required to break bigger cobbles or boulders or other obstacles (such as wooden piers/ties), then it must be completed without creating an open tunnelling face condition.

From a geotechnical perspective, the new culvert pipe should be at least 1.2 m diameter to provide proper access for obstruction removal and to facilitate personal entry (if needed) during installation. It should be noted that pipe ramming is a non-steerable system and it may be difficult to control line and grade of the installation, however the existing old culvert pipe can be used as a pilot hole to direct the ramming operation. In addition, pipe ramming may produce high vibrations considering the percussive hammer head which may cause deformation at the ground surface and/or may cause flow liquefaction for the embankment slopes. The potential vibrations effects should be assessed by the Contractor depending on the used hammer type and energy prior to driving the pipe. Vibration level that causes damage/liquefaction depend upon the Peak Particle Velocity (PPV) and the frequency at which it occurs. Damage is likely to occur where PPV is high when its frequency is low. To prevent any damage levels of PPV must be kept below 10 mm/s. Good tunnelling practices should be implemented to mitigate risk of ground loss and vibrations effects. It is also generally preferred to tunnel up-gradient for drainage control.

Using casing shoe would create an overcut around the pipe liner slightly larger than the pipe diameter, therefore it is highly recommended to complete post tunnelling grouting around the pipe liner to minimize ground surface settlement. The amount of spoil removed during tunneling should be monitored during tunnelling to confirm whether over-excavation has occurred or not and the gaps/voids should be filled with grout in a timely manner.

Details of the pipe ramming equipment and measures to protect the tunneling interface to prevent soil loss into the pipe shall be determined and submitted by contractor for approval prior to proceeding with the work.

A Non-Standard Special Provision (NSSP) regarding the installation of pipe by a selected trenchless method has been provided in Appendix F.

7.3.1.4 Launching and Receiving Shafts/Pits

Excavations for the launching and receiving shafts/pits must be carried out in compliance with O.Reg. 213/91 under the Occupational Health and Safety Act. The predominant soils encountered will be classified as Type 3 soils (O.Reg. 213/91, s. 226(4)) and temporary side must be cut at an inclination of 1 Horizontal (H) to 1 Vertical (V) or less from the base of the excavation. If saturated deposits are exposed, the cut slopes will have to be sloped back to 3H to 1V. Steeper cut slopes can be employed if shoring is used to protect workers. The protection system can be designed using the lateral earth pressure parameters as outlined in Section 7.8 and recommendations provided in Section 7.6. The design of temporary support must include control of hydrostatic forces and maintenance of basal stability.

In order to provide the required excavation geometry for the drilling (e.g. vertical front face for tunnel entry and a vertical rear face with a ballast system to act as a reaction force), the sides of the excavation will have to be shored. Shoring recommendations are provided in Section 7.6

Groundwater infiltration is expected within and near the bottom of the entry and exist pits. This groundwater may be handled using recommendations provided in Section 7.9. Pumping of groundwater seepage should be anticipated.

All fill, organic material and loose/disturbed native subsoils shall be removed down to a minimum 0.5 m depth or the native subgrade at the launching and receiving shafts/pits locations. A 0.5 m thick on-grade granular pad may be used to support the trenchless equipment. Fill required to build the pad should comprise approved imported sand and gravel materials similar to OPSS.PROV 1010 Granular B Type 1 placed in 150 mm thick lifts with each lift compacted to 100% Standard Proctor Maximum Dry Density (SPMDD).

To achieve a reasonable level of performance from a granular pad, it is essential to have a relatively uniform subgrade. Uniformity in subgrade material, moisture content, and density would be desired. This level of uniformity would require the same type of fill at a similar moisture content and density placed on the entire subgrade.

After completing the trenchless work, all pit excavations should be backfilled with OPSS.PROV 1010 Granular B Type 1 placed in maximum 150 mm thick lifts and compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD). Where subgrade is found to consist of fine-grained potentially erodible subsoils near static groundwater level, the subgrade shall be covered and protected with a rip-rap (R-50 size as per OPSS.PROV 1004).

7.3.1.5 Geotechnical Instrumentation and Monitoring Program

The potential impact of trenchless construction and/or their temporary protection system should be monitored following installation of the necessary geotechnical instrumentation, as described below and/or outlined in the associated NSSP in Appendix F. The settlement monitoring program should include a pre-condition survey of the existing highway, installing Surface Monitoring Points (SMP) and In-Ground Monitoring Points (IGMP) within the zone of influence of the trenchless alignment, collection of settlement monitoring data, and assessment of the settlement monitoring data with comparison to prescribed trigger levels. The primary purpose for monitoring ground movements is to detect them while they are still small and to modify construction procedures before the movements grow large enough to constitute a real problem.

The settlement monitoring program should comply with “Settlement Monitoring Guidelines - Tunneling” included in the MTO Guidelines for Foundation Engineering - Tunneling Specialty for Corridor Encroachment Permit Application- Details of instrumentation design; and Section 7.07 of the Non-Standard Special Provision (NSSP) “Pipe Installation by Trenchless Method” provided in Appendix F. Review Level and Alert Level and amount of displacement/distortion that necessitate response for each level should be provided/included in the Contract Documents. All monitoring points located in the unpaved portion of the right-of-way are to be founded below the frost penetration depth, which is typically 2.2 m in this area.

A suggested settlement monitoring program plan is presented below providing measures to monitor the settlement associated with the proposed trenchless activities:

- Fixed stationary reference points (i.e. Surface Monitoring Points (SMP) and In-Ground Monitoring Points (IGMP)) should be established along the pipe alignment (at a maximum spacing of 5 m) to evaluate potential soil settlement effects during trenchless works.
- The fixed points should be established on permanent structures and should be located at various distances within the trenchless zone of influence.
- The fixed points should be surveyed and recorded at least three times prior to the commencement of tunneling (at least one week prior to construction) to establish baseline elevations and should be identified and marked to serve as control (reference) points for all subsequent monitoring.
- The fixed points should be surveyed once per shift or once daily during tunnelling operations period whichever results in the more frequent reading intervals.
- The fixed points should be surveyed weekly after completion of the work for one month, or until such time at which all parties agree that further movement has stopped.
- Monitoring points should be surveyed to an accuracy of ± 2 mm.
- Review Level of 10 mm of surface ground settlement will be considered as the value of settlement reading at which it would provide necessity of altering method, rate or sequence of construction.
- Alert Level of 15 mm of surface ground settlement will be considered as the value of settlement readings at which the trenchless operations should be ceased, to allow for taking the necessary measures to mitigate unacceptable movements, lower groundwater levels or pressures, and assure safety of work and public.

The proposed settlement monitoring program plan described above are intended to monitor the construction area during trenchless operation assuming “good workmanship” and proper control of groundwater. The program of instrumentation in the tunnel could include surveying targets/prisms.

In addition, vibration monitoring points should be established along at least three cross sections (i.e. each cross section with seven monitoring points) transverse to the centerline axis of the advancing pipe. At each cross section, one monitoring point will be located directly above the pipe, with three monitoring points on right hand and left-hand side of the centerline at offsets of D , $3D$ and $5D$, where D equals the diameter of the pipe. The ground vibration should be measured using a seismograph and the Peak Particle Velocity (PPV) must be kept below 10 mm/s. Furthermore, installation of pneumatic vibrating wire piezometers along the culvert alignment (at the base of the embankment) should be considered to alert contractor about development of excess porewater pressure during the ramming operation.

Ultimately the Contractor is responsible for settlement and vibration monitoring and shall provide the collected data for the project records. The Contractor's Instrumentation personnel shall include a specialist geo-engineering consulting firm specialized in installation, monitoring and maintenance of tunnelling instruments with at least 5 years experience of similar projects.

7.3.1.6 Scour Protection

Scour protection must be provided for this culvert after the trenchless installation. Based on the information provided by GF, the creek base is at approximately elevations 226.1 and 225.70 m at the inlet and outlet, respectively. Bedrock was encountered at about 7.5 m depth below the existing culvert invert elevation. Therefore, scour protection with installation of sheet piling, or cut off wall may be attainable. Alternatively, the scour protection can be provided with equivalent horizontal cut-off walls that can reduce the hydraulic gradient and provide protection against scour. The inlet and outlet stream bed shall be also protected with a rip-rap (R-50 size as per OPSS.PROV 1004) apron. The rip-rap apron should be at least 5 m in length, a minimum 500 mm thick and extend across the stream bed to 5 m beyond the outside edges of the culvert. The rip rap placement should be in accordance with OPSP 810.010.

7.3.2 Staged Open Cut Option

Considering the subsurface conditions described in Section 5 of FIR, the use of open cut techniques with Temporary Protection System (TPS) in conjunction with staged culvert replacement is also a feasible construction option from a geotechnical perspective. The general arrangement and construction staging for staged open culvert installation option is shown on Sheet 10 of 30% Design Package by GF.

The following Table 14 summarizes evaluations of the culvert types that can be considered for open cut option, their advantages and disadvantages as well as their risks/consequences and relative costs.

Table 14 Evaluation of Culvert Type Alternatives for Open Cut Option - Advantages and Disadvantages

Options	Type	Advantages	Disadvantages	Risks/Consequences	Relative Costs
1	Precast Concrete Box Culvert	<ul style="list-style-type: none"> • Ease of installation. • Less time required for construction. • Less complex dewatering and potential to utilize partial dewatering with installation in the wet. • More tolerant to settlement than CIP options. 	<ul style="list-style-type: none"> • Transportation of culvert segments. • Limitation of width and height of culvert sections in comparison to other options. 	<ul style="list-style-type: none"> • Construction in-the-wet, if adopted, carries some risk along with advantages. 	<ul style="list-style-type: none"> • Less costly construction due to shorter construction time, but cost of transportation of segments has to be considered.
2	Cast-in-Place (CIP) Concrete Box Culvert	<ul style="list-style-type: none"> • More flexibility in sizing than precast option. • Less transportation cost for materials than precast option. 	<ul style="list-style-type: none"> • More dewatering required than precast concrete box culvert. • Longer culvert construction schedule than precast option. • Less tolerant to settlement than precast option. 	<ul style="list-style-type: none"> • Differential settlement could cause cracking of concrete in the culvert base and walls. 	<ul style="list-style-type: none"> • More costly than precast concrete box culvert due to longer construction time. • May require excavation below water level with risk of flooding into excavation. • Higher cost for dewatering than for concrete precast box culverts due to requirements for construction in the dry.
3	Cast-in-Place (CIP) Open Footing Concrete Culvert	<ul style="list-style-type: none"> • More flexibility in sizing. • Less transportation cost for materials than precast option. 	<ul style="list-style-type: none"> • Longer culvert construction schedule than precast option. • Requires footing depth to provide frost protection • More complex dewatering required than precast concrete box culvert for footing construction below water table. • Less tolerant to settlement than CIP concrete box culvert. 	<ul style="list-style-type: none"> • Due to deeper footings, increased risk of flooding of excavation and undermining existing culvert that remains in place during construction. 	<ul style="list-style-type: none"> • Higher cost for dewatering than for concrete precast box culverts due to requirements for construction in the dry.
4	Closed Pipes (Concrete, HDPE, Steel)	<ul style="list-style-type: none"> • Smaller magnitude of settlement than open footing culvert due to lower bearing stress on subgrade • Relatively expedient installation 	<ul style="list-style-type: none"> • Transportation of culvert segments. • Multiple pipes may be required to provide the same hydraulic properties as the existing culvert. 	<ul style="list-style-type: none"> • Requires water flow realignment or installation of a temporary bypass culvert to maintain existing water flow alignment 	<ul style="list-style-type: none"> • Least costly than other alternatives

Based on the evaluation in Table 14, Option 1 or 4 are considered the most feasible option for this site. Given the size of the culvert, a precast concrete box culvert option is the preferred option from a foundation engineering perspective.

The Design Team has indicated that there are no head walls or wing walls, or alternative geometry currently contemplated in their detailed design.

Culvert should be designed with built-in camber in the longitudinal direction in order to prevent ponding in the culvert in low water conditions. That is, construct the upstream half of the culvert level at the invert inlet elevation, then from that mid-point, slope the downstream half to the outlet invert elevation. In addition, the stability of the existing and proposed upstream and downstream channels during and following culvert replacement shall be addressed once additional details such topographic maps and cross-sections of the upstream and downstream channels and surrounding areas are made available to Englobe during detailed design (if the staged open cut installation method are pursued by the Design Team). Additional information such as any potential changes to the existing channel geometry and anticipated changes to the replacement culvert gradient are also expected to be provided by Design Team before Englobe can proceed with further recommendation and comments for detailed design of the staged open cut culvert replacement option.

Review of the geometry of existing embankment has raised some concerns with regards to long-term stability of the existing embankment. Therefore, a slope stability evaluation of existing embankment was conducted as part of this report and the results are presented in Section 6.11.1 below.

Finally, high level construction cost estimate for this option, provided as part of 30% Design Package to MTO, suggested estimated cost of \$2,070 per meter for culvert replacement. This estimate appears to be too low and is expected to be upgraded and revised once the preliminary design is completed during 60% design stage.

7.3.2.1 Precast Concrete Box Culvert -Foundation Recommendation

Based on the results of the boreholes advanced through the embankment, the native, undisturbed silt below Elevation 225.3 m is considered capable of supporting a culvert and a conventional highway embankment of the height anticipated at this site.

Any organic soil encountered at the founding elevation should be sub-excavated and replaced with OPSS.PROV 1010 Granular B Type 2. Availability of adequate bearing resistance should not be a major issue provided the natural bearing surface is not disturbed during construction and groundwater is controlled throughout construction, as discussed in Section 7.9.

Based on the characteristics of the native subgrade present below the culvert and the response of the existing embankment, a factored bearing resistance at Ultimate Limit State (ULS) of 300 kPa can be used for the design of the precast concrete box culvert at a founding elevation of approximately 225.3 m or below. In consideration of the width of the culvert, depth of overburden, and response of the existing embankment, a geotechnical resistance at Serviceability Limit State (SLS) of 200 kPa, founded on the granular bedding over native silt (free of organic matters) can be used for the design of the precast concrete box culvert.

A geotechnical resistance factor of 0.5 was used for obtaining bearing resistances assuming vertical loading as per Table 6.2 of CHBDC.

The total and differential settlements of the proposed closed bottom culvert subjected to the above maximum Serviceability Limit State (SLS) pressure, are estimated to not exceed 25 and 19 mm, respectively.

Sliding resistance of culvert foundation can be calculated using a coefficient of sliding resistance of 0.30 for concrete against the native soils and 0.45 for concrete against well graded granular engineered fills.

The footing areas must be inspected by a qualified geotechnical engineer/technician at the time of construction to confirm soil conditions encountered.

7.3.2.2 Culvert Design, Bedding, and Embedment

In general, culvert installation shall be carried out in accordance with the applicable OPSS Standards, i.e., OPSS.PROV 902 (Construction Specification for Excavating and Backfilling - Structures), OPSS 421 (for flexible pipe culvert installation in open cut) and OPSS 422 (for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut).

If a precast concrete box culvert is considered, bedding material shall consist of Granular A with a thickness of 300 mm. The bedding under the middle third of the box unit base should be loosely placed and uncompacted. The upper 75 mm portion of the Granular A bedding should be uncompacted throughout the length/width of the box and incorporated as the top levelling course in conformance with OPSS 422. Alternatively, specifically if construction is carried out under wet conditions, a bedding and levelling course consisting of 19 mm clear stone as per OPSS.PROV 1004 should be used, which would aid in dewatering applications (wrapped with geotextiles). During backfilling the embankment fill should be placed in a balanced manner on the outer sides of the box unit. The elevation difference of the backfill on either side of the box unit must be limited to a maximum of 300 mm. Backfilling and construction of precast concrete box culverts shall be in accordance with OPSS 422. Cover material for concrete box culverts can consist of compacted OPSS Granular B Type 1, placed to minimum 300 mm around the box culvert.

The joints between precast box units should be covered with a strip of Non-Woven Class II Geotextile (per OPSS 1860) 600 mm in width, centred over the joint, covering the top of the culvert and extending down the sides of the culvert to prevent the infiltration of fines.

Vertical cut-off walls, 1.2 m deep, shall be added to the ends of the box culvert in accordance with the MTO Concrete Culvert Design Manual. Alternatively, horizontal cut-offs wall (apron) may be also used; the advantage of horizontal cut-offs is that they minimize dewatering requirements, which are a critical issue at this site.

Scour protection must be provided at the ends of the culvert and should consist of a layer of rip-rap. The inlet and outlet stream bed shall be protected with a rip-rap (R-50 size as per OPSS.PROV 1004) apron. The rip-rap should serve as a rock protection layer and can be machine placed. The rip-rap apron should be at least 5 m in length, a minimum 400 mm thick and extend across the stream bed to 5 m beyond the outside edges of the culvert. The rip rap placement should be in accordance with OPSD 810.010.

Scour recommendations provided above are a minimum requirement from a foundations engineering perspective. CHBDC requires the hydrologist to do the detailing of the scour protection and then for the foundation engineer to check if that design meets these minimum requirements. Clay seals will be required at this culvert location; the clay seal should be at least 500 mm thick and should be extended 300 mm above the high-water level covering the width of the granular material. The clay seal seepage barrier should extend the full width of the trench to contact the existing soils on either side (surrounding the culvert bedding). The minimum length of the clay seal should be twice the culvert rise/diameter (2D). The clay seal barriers should consist of relatively dry and compactable inorganic clay meeting OPSS.PROV 1205 placed in maximum 200 mm thick loose lifts and compacted to at least 95% of Standard Proctor Maximum Dry Density (SPMDD). The clay seal should be protected by a layer of rip-rap. The clay seal material requirements should be in accordance with OPSS.PROV 1205. Alternatively, a geosynthetic clay liner can be utilized as a clay seal.

Woven geotextiles Class II are recommended below Granular A bedding for separation (separating the native subgrade from the aggregate layer). However, the geotextiles should not be placed at the clay seal sections (i.e. the geotextile is not continuous).

The Contractor should provide silt fences and erosion control blankets as per OPSS 805 throughout the duration of construction to prevent transport of silt/sediment.

7.3.2.3 Staged Construction Using Shoring

Installing of temporary shoring down the centre of the highway would be needed to allow staged construction with one lane of traffic during construction. The currently proposed staging plan for a continuous open cut excavation while maintaining one lane of traffic is as follows:

- Install Temporary Protection System (TPS) down the centre of the highway;
- Divert the traffic to the right side of the embankment;
- Excavate to the left of centreline, and install half of new culvert;
- Reconstruct the embankment on the left;
- Divert the traffic to the left side of the embankment;
- Excavate to the right of centreline and install the other half of new culvert;
- Reconstruct the embankment on the right;
- As the removal of the shoring walls may create big voids in the soil behind the walls, it is recommended that the TPS should be left in place and cut it off to the required depth; and
- Once traffic can be returned to the existing alignment, remove the culvert extensions.

7.3.2.4 Settlement of Reinstated Embankment

According to MTO Guideline for Foundation Engineering Services, Version 2.0 (October 2020), Appendix C - Embankment Settlement Criteria for Design, Figure 1, New Embankment, 100 mm of post-construction settlement is considered to be acceptable for embankments constructed on non-compressible soils. Considering the response of the existing embankment and given that the culvert replacement will not involve a grade raise, settlement of the final embankment is not considered to be a significant concern using the recommendations and placement techniques presented in this report.

A total settlement equal to 1.0% of the embankment height is expected for granular embankments constructed over a non-compressible subgrade. Given that the height of the embankment will be approximately 9 m above the culvert, a total of primary and post-construction (secondary) settlement would be 90 mm which does not exceed the 100 mm threshold.

7.4 Design Code Consideration

The foundation recommendations presented in this report have been prepared in accordance with CSA S6:19 Canadian Highway Design Bridge Code (CHBDC), Ontario Provincial Standard Specifications (OPSS) and MTO's policies.

The Site Consequence and Understanding classifications at this site are "Typical" in accordance with Section 6.5 of the CHBDC. The geotechnical resistance factors presented in Table 6.2 of CHBDC were used to calculate the factored foundation resistances. A geotechnical resistance factor of 0.5 was applied for the ultimate geotechnical resistance of shallow foundation to resist loads in bearing.

7.5 Frost Penetration

Generally, culverts within the depth of frost penetration below the pavement structure are included in the pavement structure frost treatment (see OPSS 803.010 and OPSS 803.030). In general, closed culverts under road embankments are not designed in consideration of frost penetration below the

culvert, whereas culverts with footings, (i.e., open culverts, culvert retaining walls, etc.) require the footings to be designed for frost penetration.

The estimated frost penetration depth below exposed asphalt at the culvert site is about 2.2 m, as per OPSD 3090.100 (Frost Penetration Depths for Northern Ontario). It should be noted that the closed culvert option currently being contemplated at this location will not require frost treatments.

7.6 Temporary Protection System

Consideration should be given by the Contractor to the provision of a Temporary Protection System (TPS) along the approximate location of launching and receiving shafts/pits, and where necessary considering the stability of the existing embankment during construction. The TPS should be designed and constructed by an experienced Contractor and must comply with OPSS.PROV 539 and SP 105S09. The design and installation of the shoring and protection system is the responsibility of the Contractor and must comply with O.Reg. 213/91, and must be endorsed by a professional engineer with at least 5 years experience with similar work and installations.

In addition, the Contractor must also be prepared during construction to handle wet weak subgrade conditions with high moisture content and low strength and the presence of obstructions such as rock and concrete debris within embankment fill material and control the groundwater as the excavation progresses without compromising the stability of the existing embankment. A NSSP regarding potential obstructions in the subsurface soils has been included in Appendix E

The following aspects regarding shoring systems are provided for general guidance only. The temporary shoring design must include factors of safety as per Table 6.2 of the CHBDC for Retaining systems, and should address any possible surcharge loading (such as traffic loading and any stockpiles placed within a distance from the supporting face of the TPS equal to its effective height).

Special attention should be paid to the presence of occasional cobble sized rock pieces and remnant of concrete debris in the embankment fill (identified for example in Borehole No. 1), and the fine-grained nature of underlying native subsoils and presence of bedrock (at about 9.5 m below the invert of the existing culvert in Borehole No.2). The Contractor should be made aware of site-specific site conditions and potential presence of similar inclusions/obstructions within the embankment fill between the exploratory boreholes.

For temporary shoring support a wall consisting of soldier piles and lagging is typically considered along with the required dewatering system. The soldier piles would be H-piles advanced through the embankment fill and extending to sufficient depth (or, into bedrock), followed by sequential excavation and installation of lagging. To advance the H-piles through the buried rock and concrete debris near the bottom of the embankment fills, removal of the obstructions using a core barrel and other effective means of overcoming obstructions are expected to be required. Alternative support system such as reinforced shotcrete, or shoring support such as caisson wall may also be considered where appropriate. However, these systems are generally expected to be more costly.

The shoring protection system can be designed using the lateral earth pressure parameters as outlined in Section 6.8 below. If tiebacks are required, the resistance (P_{ar}) for grouted anchors located outside the active failure wedge in cohesionless soils can be estimated from the following equation as supplied in Section 26.12.4.1 of the Canadian Foundation Engineering Manual (CFEM) (4th Edition):

$$P_{ar} = \sigma'_z A_s L_s \alpha_g$$

where:

σ'_z = effective vertical stress at the mid-point of the load carrying length (Figure 26.15 of CFEM)

A_s = effective unit surface area of the anchor bond zone

L_s = effective length of the anchor bond zone (limited to about 8 m)

α_g = anchor coefficient dependent on the soil type and conditions as per Table 26.5 of CFEM

Unless the pull-out resistance (capacity) of the anchor is proven with a load test program, the factored ULS anchor load is commonly obtained by multiplying the computed capacity of the anchor by 0.4 resistance factor (as per CHBDC).

Considering the composition of the embankment fills and the soil below the embankment fills which consisted of a thick layer of compact native silty sand/silty sand, a rectangular apparent pressure distribution over the height of the cut would be appropriate for design of the TPS. The width of the apparent rectangular pressure distribution should be calculated based on Section 26.10 of the CFEM, assuming layered strata (loose to compact cohesionless silty sand/sandy silt).

7.7 Earthquake Considerations

7.7.1 Site Classification for Seismic Site Response

The CSA S6.1:19 Commentary on CHBDC describes the site classification depending on the seismic site response. Site classification is determined by the type of soil encountered during fieldwork.

A Site Classification E should be used for earthquake load and effects in accordance with Table 4.1.8.4.A. of the National Building Code of Canada (NBC 2015) and Table 4.1 of the CSA S6:19 CHBDC (utilizing the standard penetration resistance “soft soil ground”, $N_{60} < 15$ average properties for the overburden thickness above bedrock). The bedrock depth should not be considered in the evaluation of the Site Class unless Shear Wave Velocity (V_s) is measured for the site.

7.7.2 Uniform Hazard Spectrum

The CHBDC refers to the NBC 2015 to define the Uniform Hazard Spectrum, UHS. The NBC 2015 gives the spectral acceleration values for different periods, the peak ground acceleration (PGA), as well as the peak ground velocity (PGV) values for different cities or municipalities in Canada. In the study area, the spectral acceleration and peak ground acceleration data for a return period of 2% for 50 years are shown in Table 15.

Table 15 Uniform Hazard Spectrum

Site Localisation	Seismic Values					
	Sa (0.2)	Sa (0.5)	Sa (1.0)	Sa (2.0)	PGA (g)	PGV (m/s)
Latitude: 48.248199 Longitude: -89.483835	0.060	0.035	0.018	0.007	0.035	0.024

Reference : Tool of the National Building Code of Canada seismic hazard values - NBC 2015 :
http://www.seismescanada.mcan.gc.ca/hazard/interpolator/index_2015-fra.php

7.7.3 Seismic Liquefaction

Based on the PGA value of 0.035 g, and an earthquake magnitude, M_w , of 6.86, the subsurface conditions encountered at the drilled locations at this site and using the Simplified Boulanger and Idriss (2014) Method for liquefaction assessment, the foundation soils are considered to be not susceptible to liquefaction during a seismic event.

7.8 Lateral Earth Pressures

Lateral earth pressures in entry and exist pits can be computed using the recommended soil parameters in Table 16 below.

Table 16 Recommended Soil Parameters for Geotechnical Design

Material	Unit Weight (kN/m ³)	Effective Angle of Friction (ϕ)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure at Rest (K_o)
Granular A	22.8	35	0.27	3.69	0.43
Granular B Type I	21.2	33	0.29	3.39	0.46
Granular B Type II	21.0 to 22.0	37	0.25	4.00	0.40
Embankment Fill (Loose to compact)	20.0	28	0.36	2.77	0.53
Sand to Silty Sand (Loose to compact)	18.0	30	0.33	3.00	0.50

For shoring systems where limited lateral deflection up to a maximum 25 mm and/or rotation up to 0.004 of the wall height is permitted, the “active” condition (K_a) applies. For rigid shoring systems with tight deflection limits, where no significant movement is permitted “at-rest” condition (K_o) applies. The “passive” condition (K_p) applies when movement is allowed and when the wall is in compression (in a direction opposite to the wall loading) such as development of embedment passive resistance.

Table 17 Typical Wall Movements to Activate K_a and K_p

Type of Backfill	Value of Δ/H	
Level of Compaction	Active	Passive
Loose Sand/Silt	0.004	0.04
Compact Sand/Silt	0.002	0.02

where:

Δ = the movement of top of wall required to reach minimum active or maximum passive pressure, by tilting or lateral translation, and

H = height of wall.

7.9 Excavation, Dewatering, Channel Diversion and Cofferdams

All temporary excavations greater than 1.2 m in depth must, at a minimum, be sloped or shored in accordance with the Occupational Health and Safety Act Regulations for Construction Projects. The embankment material, above the water table, is considered a Type 3 soil as defined in the Occupational Health and Safety Act and Regulations for Construction Projects. Temporary open excavations above the groundwater table, could be cut back at an angle of 1H:1V, provided they are monitored continuously. However, below the groundwater table, the side slopes in fill and/or native materials may need to be sloped to angles as flat as 3H:1V or possibly flatter, dependent upon the Contractors’ chosen method of controlling the groundwater. No bedrock is expected along the culvert alignment since bedrock was only encountered at about 7.5 m depth below the existing culvert invert elevation.

At the time of investigation, the groundwater was encountered as described in Section 5.7 at elevations ranging from 224.8 to 226.9 m. The lowest creek level is anticipated at elevation 225.9 m. Seasonal fluctuation in the groundwater level typically up to 1 m should also be expected and considered for the purpose of detailed design and construction. Based on short-term observations made during the borehole drilling, no major groundwater seepage is expected within the embankment, but groundwater may be encountered in the native subsoils. As such, dewatering is expected to be required during excavation and culvert installation. In general, the groundwater level should be kept at

least 0.5 m below the bottom of the proposed excavations, or more as required by the Contractor (considering their choice of their equipment). The proposed dewatering should be effective considering the stability of the sides and bases of the excavation and allow the construction activities to proceed in dry conditions.

In general, The Contractor shall be responsible for employing appropriate excavation and dewatering methods to maintain the stability of the sides and base of excavation. Nevertheless, at a minimum Installation of filtered sumps and pumping from the base of the excavation is expected to be required during construction to maintain the excavation in a dewatered condition during culvert installation. The effectiveness of this method of groundwater control would be limited to conditions where excavations of less than 1 m below the prevailing groundwater table are anticipated, and the soil is such that the groundwater can be drawn down a minimum of 500 mm below the working surface. If the excavation must penetrate to a greater depth below the prevailing groundwater table, a more effective groundwater control method, should be considered by the Contractor to maintain a stable excavation base. The Contractor's dewatering method must be designed to prevent piping.

Registering with the Environmental Activity and Sector Registry (EASR) for the construction activity may be required. Consideration must be given to the anticipated water levels at the culvert site throughout the construction and the potential need for a Permit to take Water (PTTW), which will also depend upon the Contractor's proposed methodology and schedule. An EASR or PTTW is generally required by the Ministry of Environment and Climate Change in the event that the daily taking of groundwater exceeds 50,000 L or 400,000 L per day, respectively.

A cofferdam constructed of earth fill, sandbags, or water-filled bag (i.e., aqua dam) can be considered at this site to temporarily divert the creek water during the construction period. Steel sheet piles may also be considered for controlling stream flow. Sheet piles should extend to adequate depth below base of the proposed excavation. By-pass pumping can be carried out to divert the stream flow at the time of construction.

Ultimately, the method of excavation, shoring, dewatering, and stream flow diversion shall be selected by the Contractor responsible for effectiveness of the works. However, the importance of maintaining the excavations in a dewatered stable condition cannot be over-stressed enough.

7.10 Slope Stability Evaluation

7.10.1 Slope Stability for Open Cut option (Embankment Reconstruction)

It is noted that part of the embankment slopes at the existing culvert are with inclinations of 1.75H:1.0V. A two-dimensional (2-D) limit equilibrium stability analysis, using the GEO-SLOPE computer program, Slope/W (GeoStudio 2020, version 10.02, Geo-Slope International Ltd.), was carried out using the Morgenstern-Price method for a roadway cross section located at the middle of the existing culvert with slopes established at an angle of 1.75H:1.0V. For the purpose of these analyses, the materials were modeled using the soil parameters for embankment fill and foundation subsoils, as noted in Table 18 below.

Table 18 Recommended Soil Parameters for Slope Stability

Material	Unit Weight (kN/m ³)	Effective Angle of Friction (°)	Effective Cohesion (kPa)
Embankment Fill-Sand some Gravel	20.0	35	0
Embankment Fill- Clay	18.0	28	2.5
Embankment Fill-Silt and Sand	19.0	28	0
Embankment Fill-Silty Sand	19.0	30	0
New Embankment Fill (Granular B Type 2)	21.0	37	0

Material	Unit Weight (kN/m ³)	Effective Angle of Friction (ϕ)	Effective Cohesion (kPa)
Silty Sand	19.0	30	0
Sand	19.0	30	0
Silty Clay	18.0	28	2.5
Silt	18.0	28	1
Bedrock	Impenetrable		

The analysis indicated that the factor of safety was in the order of 1.12 for the existing embankment slopes, which is below the acceptable factor of safety of 1.3 for embankment slopes, see Figure No. S1 (Slope Stability for Existing Condition-Static), Appendix G.

In addition, the stability of embankment slopes has been analyzed by Englobe for seismic loading condition (using Pseudo-Static method of seismic response) to determine the seismic factor of safety. For this site, the following design parameters were considered to develop the seismic condition:

- Peak Ground Acceleration (PGA)= 0.035 g (as per National Building Code of Canada seismic hazard values and based on site Latitude and Longitude).
- Seismic Horizontal Acceleration Coefficient, $k_h = 0.5(\text{PGA}/g) = 0.0175$ as indicated in Section 6.14.9.1 of CSA S6:19 Canadian Highway Design Bridge Code (CHBDC).
- Seismic Vertical Acceleration Coefficient, $k_v = 2/3 k_h = 0.0117$.

The seismic evaluation indicated that the factor of safety was in the order of 1.07 which is below the acceptable factor of safety of 1.1, see Figure No. S2 (Slope Stability for Existing Condition- Seismic), Appendix G.

To improve the stability for the reconstructed embankment, the embankment slopes should be constructed with OPSS.PROV 1010 Granular B Type 2 (placed in maximum 150 mm thick lifts and compacted to at least 100% of SPMDD) and established at an angle of 2H:1V to provide additional rotational resistance. A factor of safety above 1.53 for static loading condition and 1.4 for seismic loading condition are achievable for the reconstructed embankment slope with suggested improvements, see Figure Nos. S3 (Slope Stability for Modified 2H:1V slope-Static), and S4 (Slope Stability for Modified 2H:1V slope- Seismic), Appendix G.

It should be noted that as the final slopes will be an inclination of 2H:1V, localized shallow sloughing (near-surface) may develop. Preventing surface erosion and accelerating the establishment of a vegetation cover play a significant role in preventing shallow slip surfaces.

Lower factors of safety will occur in the short-term during excavation and backfilling. However, short-term stability should not be an issue if the construction is carried out following appropriate geotechnical practices, as described in Section 7.9.

7.10.2 Potential Impacts of Pipe Ramming on Stability of Existing Embankment

Under each dynamic application of the force by the pipe ramming equipment, the pipe vibrates and the generated vibrations are transferred from the pipe to the surrounding soil particles. The degree of vibration depends on the hammer type and hammer energy, the dynamic properties of the soil, and the distance between the source of energy and location of interest.

Considering the fine-grained nature of the existing embankment subsoils at and near the location of the existing culvert crossing and the condition of existing embankment slopes at about 1.75H:1V, potential impacts of the proposed pipe ramming with high energy rammer on the stability of the embankment slopes should be considered by the Contractor.

As a general guidance, Englobe has completed a Pseudo-Static analysis of seismic response using Slope/W to mimic the pipe ramming vibrations effect on the embankment slopes assuming the following parameters:

- Maximum Horizontal Acceleration Coefficient, k_h that will be generated by pipe ramming impact hammer is 0.0075.
- Maximum Vertical Acceleration Coefficient, k_v that will be generated by pipe ramming impact hammer is 0.0050.

If the pipe ramming operation is planned and carried out in such a way not to exceed the acceleration coefficients specified above, a factor of safety of 1.1 for the existing embankment slopes would be maintained, see Figure No. S5 (Slope Stability for Assumed Acceleration for Pipe Ramming- Seismic), Appendix G. Successful completion of pipe ramming largely depends on appropriate selection of equipment and methods and the skills and experience of the Contractor. The Contractor proposed procedure should ensure that no flow liquefaction (significant strength loss in soils) will occur to the slope during the pipe ramming operation. A pipe ramming driveability analysis and report needs to be provided by the selected Contractor for review and approval in advance of the construction.

Prior commencing the pipe ramming operation, however, consideration should be given to construct a one third height berm at the toe of the embankment (10 m long on both sides) to increase the stability of the embankment slopes and achieve a minimum FS of 1.3 under static conditions, see Figure Nos. S6 (Slope Stability for One Third Height Berm-Static), Appendix G. In addition, the PPV level of the pipe ramming must be kept below 10 mm/s. Furthermore, installation of pneumatic vibrating wire piezometers along the culvert alignment (at the base of the embankment) should be considered to alert contractor about development of excess porewater pressure during the ramming operation.

7.11 Potential Corrosivity of Subsoil and Groundwater

Representative soil and water corrosivity test results was presented in Section 5.8 and compared with the applicable Canadian Standards Association (CSA) Standards in Table 19 below.

Table 19 Sulphate content and exposure classes¹

Class of Exposer	Degree of Exposure	Water-Soluble SO ₄ in Soil Sample (%)	Cementing Material to be Used
S-1	Very Severe	> 2.0	HS or HSb
S-2	Severe	0.20 - 2.0	HS or HSb
S-3	Moderate	0.10 - 0.20	MS, MSb, LH, HS, or HSb

¹ Information from Table 3 of CSA Standards A23.1-04

The sulphate content analyses for the tested soil sample resulted in a sulphate percentage of 0.031%. The result was compared with the Canadian Standards Association (CSA) Standards A23.1 for sulphate attack potential on concrete structures, which indicates the Site soils possess a “negligible” risk for sulphate attack on concrete material. Nevertheless, based on the observation that the existing CSP culvert barrel at the site is corroded, it is recommended that a “moderate” (Class S-3) classification be adopted. Accordingly, Type MSb (Moderate Sulphate Resistant) or Type HS or HSb (Highly Sulphate Resistant) cement may be used in the construction of any proposed concrete elements.

Based on the test results and using the guidelines and 10-point scoring method provided by the American Water Works Association (AWWA) publication ANSI/AWWA C105/A21.5-10, the tested soil sample from BH No.2 received a corrosivity score above 10. A score of 10 or greater indicates that soil is corrosive to ductile-iron pipe and protection is needed. Thus, based on the results from BH No. 2, buried ductile iron components of the subsurface structures are expected to require corrosion protection measures.

8 Statement of Limitations

The design recommendations given in this geotechnical report are applicable only to the project described in the text and only if constructed substantially in accordance with details of alignment and elevations stated in the report. Since all details of the design may not be known, in our analysis certain assumptions had to be made. The actual conditions, however, may vary from those assumed, in which case changes and modifications may be required to our geotechnical recommendations.

The comments in this report are intended solely for the guidance of the design engineer and address the geotechnical conditions only. The number of boreholes required to determine the localized conditions between boreholes directly affecting construction costs, equipment, scheduling, etc. would in fact be greater than what has been carried out for design purposes. Therefore, contractors bidding on this project or undertaking this work should make their own interpretations of the factual borehole results and carry out further work as they deem necessary to assess the scope of the project.

Foundation Design of this report is intended solely for the use of the client and the design team for the detail design of this specific project on behalf of the Ministry of Transportation and is not intended to be included in the tender documents; and shall not be used for any other purposes or by any other parties including the construction Contractor.

Appendix A

Drawings

Drawing No. 1 - Site Location Plan & Key Map

Drawing No. 2 - Borehole Location Plan & Embankment Profile

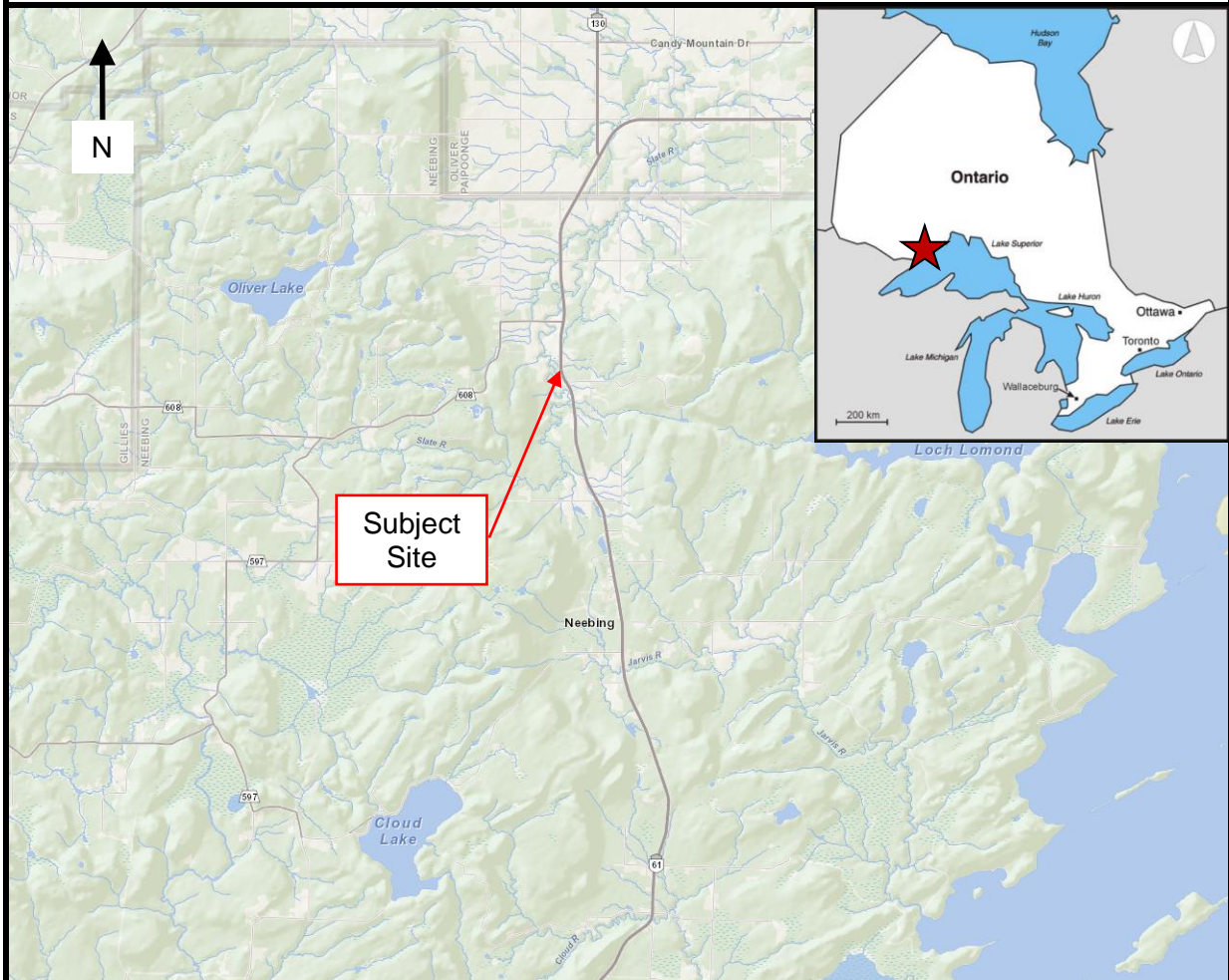


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KEY PLAN

Drawing No. 1

NOT TO SCALE



FINAL FOUNDATION INVESTIGATION REPORT

Station 20+375 Culvert
Culvert Replacement
Highway No. 61, Twp. of Blake
Assignment Number 6020-E-0021
GWP 6176-15-00

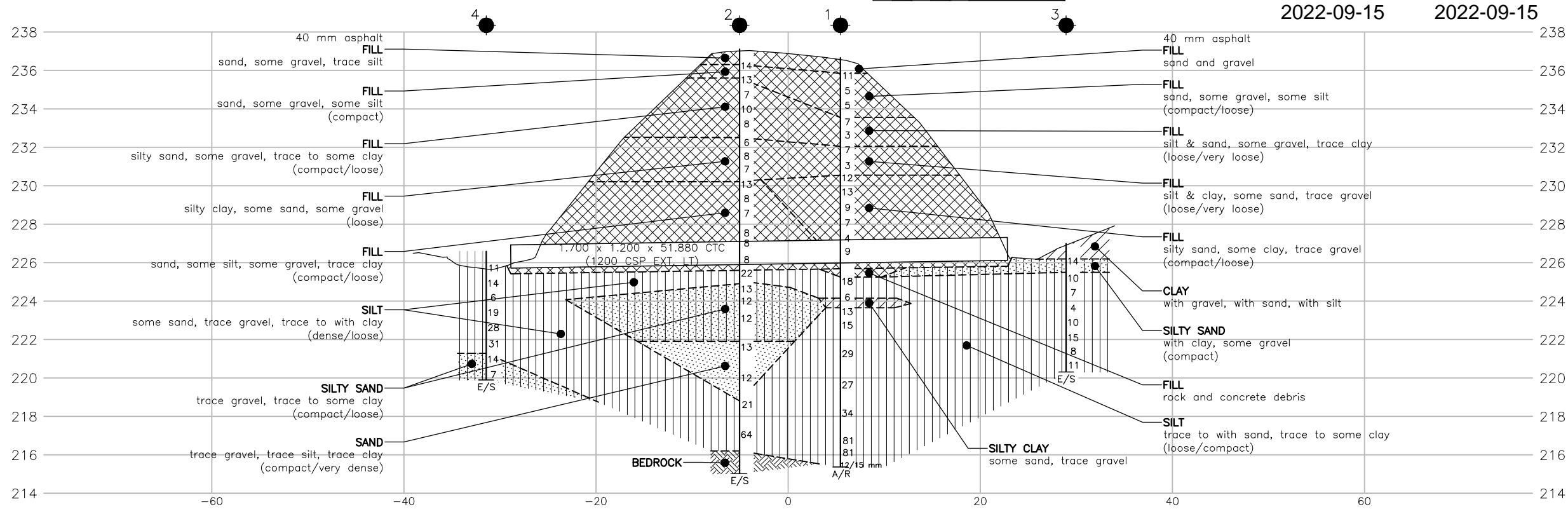
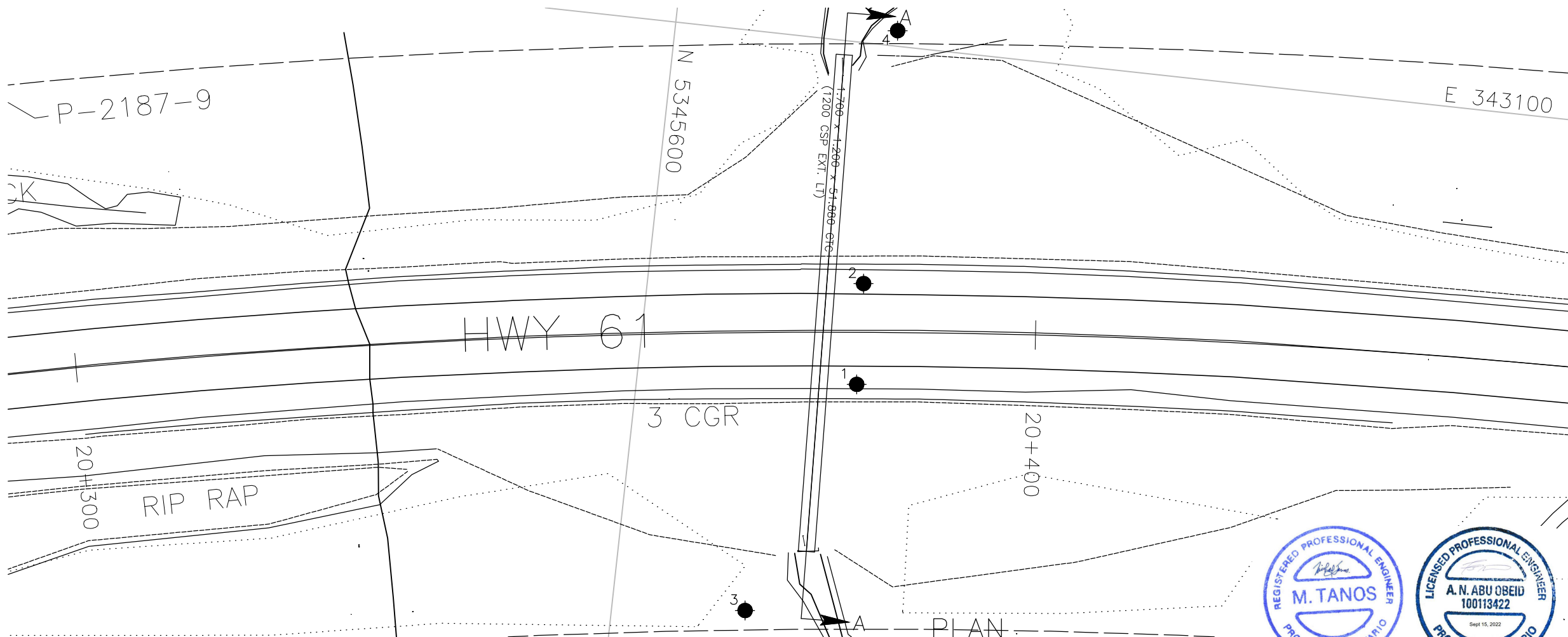
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September 2022



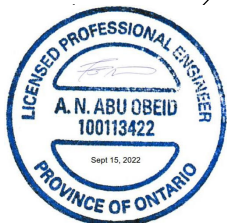
CAD FILE LOCATION AND NAME: E:\152\Projects For Other Offices\02109931 - EDN, HYD & DSS - Hwy 61, 6012-E-0021 (05T)\Drawings\02109931 - 20+040, 20+200, and 20+375, Brooks.dwg
MODIFIED: 9/6/2022 1:51:45 AM BY: MITOU
DATE PLOTTED: 9/6/2022 2:04:43 AM BY:

PR-D-707 BR-03
MINISTRY OF TRANSPORTATION, ONTARIO



This drawing is for subsurface information only. Surface details and features are for conceptual illustration. The proposed structure location is shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

SECTION A-A, STATION 20+375



DISTRICT
CONT. No.
GWP No. 6176-15-00



REHABILITATION OF HWY 61
CULVERT REPLACEMENT
STATION ±20+375
BOREHOLE LOCATIONS
AND SOIL STRATIGRAPHY

SHEET
2



KEY PLAN
N.T.S.

	Borehole
N	Blows/0.3 m (Std Pen Test, 475 J/blow)
DCPT	Blows/0.3 m (60° Cone, 475 J/blow)
W	Water Level at Time of Investigation
A/R	Auger Refusal at Elevation
E/S	End of Sampling
P	Piezometer

BOREHOLE No.	ELEVATION	O/S	NORTHING	EASTING
1	236.7	5.4 m Rt	5345622.8	343135.5
2	237.1	5.1 m Lt	5345622.4	343125.0
3	228.0	28.9 m Rt	5345613.8	343160.2
4	226.6	31.4 m Lt	5345623.1	343098.4

NOTES:
The boundaries between soil strata have been established at the borehole locations only. The boundaries illustrated and stratigraphy between boreholes on this drawing are assumed based on borehole data and may vary. They are intended for design only.

Base plan and alignment provided in digital format by Aecom on July 27, 2021

Coordinates based on MTM Zone 15 NAD83 CSRS

GEOCRES No. 52A00-264

REVISIONS	DEC/21	DM	DRAFT				
	MAY/22	DM	REVISED DRAFT				
	SEP/22	DM	FINAL				
DESCRIPTION							
DESIGN		CHK		CODE	LOAD	DATE	SEP/2
DRAWN	DM	CHK	FS	SITE	STRUCT	SCHEME	DWG 2

Appendix B

Subsurface Data

Enclosure No. 1 List of Abbreviations and Symbols
Enclosure Nos. 2 to 7 Record of Borehole Sheets



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LIST OF SYMBOLS AND DEFINITIONS FOR GEOTECHNICAL SAMPLING AND COMMON LITHOLOGIES

The following is a reference sheet for commonly used symbols and definitions within this report and in any figures or appendices, including borehole logs and test results. Symbols and definitions conform to the standard proposed by the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) wherever possible. Discrepancies may exist when comparing to third-party results using the Unified Soil Classification System (USCS).

PART A – SOILS

Standard Penetration Test (SPT) 'N'

The number of blows required to drive a 50-mm (2 in) split barrel sampler 300 mm (12 in). The standard hammer has a mass of 63.5 kg (140 lbs) and is dropped vertically from a height of 760 mm (30 in). Additional information can be found in ASTM D1586-11 and in §4.5.2 of the CFEM 4th Ed.

For penetration less than 300 mm, 'N' is recorded with the penetration that was achieved.

Non-Cohesive Soils

The relative density of non-cohesive soils relates empirically to SPT 'N' as follows:

Relative Density	'N'
Very Loose	0 – 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	> 50

Cohesive Soils

The consistency and undrained shear strength of cohesive soils relates empirically to SPT 'N' as follows:

Consistency	Undrained Shear Strength (kPa)	'N'
Very Soft	< 12	0 – 2
Soft	12 – 25	2 – 4
Firm	25 – 50	4 – 8
Stiff	50 – 100	8 – 15
Very Stiff	100 – 200	15 – 30
Hard	> 200	> 30

PART B – ROCK

The following parameters are used to describe core recovery and to infer the quality of a rockmass.

Total Core Recovery, TCR (%)

The total length of solid drill core recovered, regardless of the quality or length of the pieces, taken as a percentage of the length of the core run.

Solid Core Recovery, SCR (%)

The total length of solid, full-diameter drill core recovered, taken as a percentage of the length of the core run.

Rock Quality Designation, RQD (%)

The sum of the lengths of solid drill core greater than 100 mm long, taken as a percentage of the length of the core run. RQD is commonly used to infer the quality of the rockmass, as follows:

Rockmass Quality	RQD (%)
Very Poor	< 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	> 90

Weathering

The terminology used to describe the degree of weathering for recovered rock core is defined as follows, as suggested by the *Geological Society of London*:

Completely weathered: All rock material is decomposed and/or disintegrated to soil. The original mass structure is largely intact.

Highly weathered: More than half the rock material is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as a discontinuous framework or as core stone.

Moderately weathered: Less than half the rock material is decomposed and/or disintegrates to soil. Fresh or discolored rock is present either as a continuous framework or as core stone.

Slightly weathered: Discoloration indicates weathering of rock material and discontinuity of surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than its fresh condition.

Fresh: No visible signs of weathering.

PART C – SAMPLING SYMBOLS

Symbol	Description
SS	Split spoon sample
TW	Thin-walled (Shelby Tube) sample
PH	Sampler advanced by hydraulic pressure
WH	Sampler advanced by static weight
SC	Soil core

PART D – IN-SITU AND LAB TESTING

SOIL NAMING CONVENTIONS

Particle sizes are described as follows:

Particle Size Descriptor	Size (mm)
Boulder	> 300
Cobble	75 – 300
Gravel	Coarse 19 – 75 Fine 4.75 – 19
Sand	Coarse 2.0 – 4.75 Medium 0.425 – 2.0 Fine 0.075 – 0.425
Silt	0.002 – 0.075
Clay	< 0.002

The principle constituent of a soil is written in uppercase. The minor constituents of a soil are written according to the following convention:

Descriptive Term	Proportion of Soil (%)
Trace	1 – 10
Some	10 – 20
(ey) or (y)	20 – 35
And	35 – 50

Eg.: A soil comprising 65% Silt, 21% Sand and 14% Clay would be described as a: Sandy SILT, Some Clay

RECORD OF BOREHOLE No. 1

1 OF 2

METRIC

W.P. GWP 6176-15-00 LOCATION 20+381, 5.6 Rt, Blake Twp. ORIGINATED BY RT
 DIST Thunder Bay HWY 61 BOREHOLE TYPE Hollow Stem Auger COMPILED BY DM
 DATUM Geodetic DATE 2021.10.12 - 2021.10.15 MTM Zone 15 343135.477 E 5345622.839 N
 LATITUDE 48.248226 LONGITUDE -89.483761 CHECKED BY FS

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 (%)		
								20	40	60	80	100						○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL
236.7	40 mm asphalt		1	AS			236													
235.9	FILL - SAND & GRAVEL - trace silt, brown		2	SS	11															
0.8	FILL - SAND - some silt, gravel, brown, compact to loose		3	SS	5		235													
			4	SS	5		234													
233.6																				
3.1	FILL - SILT & SAND - some gravel, trace clay, brown, loose		5	SS	7		233													
			6	SS	3															
232.1	FILL - SILT & CLAY - some sand, trace gravel, brown/red, firm		7	SS	7		232													
4.6	- trace sand, gravel, high plasticity		8	SS	3		231													
230.6							230													
6.1	FILL - SILTY SAND - some clay, trace gravel, brown, compact to loose		9	SS	12															
			10	SS	13		229													
	- damp		11	SS	9															
	- some gravel, clay, brown/red, moist, loose		12	SS	7		228													
			13	SS	4		227													
	- some gravel, brown, moist to wet, loose		14	SS	9															
226.3					10/ 0 mm															
10.4	FILL - ROCK & CONCRETE - debris		RC				226													
			NR																	
225.3																				
11.4	SILT - some clay, trace sand, brown, moist to wet, compact		15	SS	18		225													
			16A	SS	6															
224.2			16B				224													
12.5	SILTY CLAY - some sand, trace gravel, grey, wet																			
223.7							223													
13.0	SILT - with sand, trace clay, grey, wet, compact to very dense		17	SS	13															
			18	SS	15															
	- occasional cobbles						222													

Continued Next Page

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

ONTARIO MTO GWP 6176-15-00 - HIGHWAY 61 - CULVERT 20+375.GPJ ONTARIO MTO.GDT 22-4-6

RECORD OF BOREHOLE No. 1

2 OF 2

METRIC

W.P. GWP 6176-15-00 LOCATION 20+381, 5.6 Rt, Blake Twp. ORIGINATED BY RT
 DIST Thunder Bay HWY 61 BOREHOLE TYPE Hollow Stem Auger COMPILED BY DM
 DATUM Geodetic DATE 2021.10.12 - 2021.10.15 MTM Zone 15 343135.477 E 5345622.839 N
 LATITUDE 48.248226 LONGITUDE -89.483761 CHECKED BY FS

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100	W _p	W	W _L		
							SHEAR STRENGTH kPa					WATER CONTENT (%)			kN/m ³	GR SA SI CL
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
							40	80	120	160	200	20	40	60		
	- trace gravel, occasional cobbles		19	SS	29							o				7 25 61 6
						221										
						220										
			20	SS	27											
						219										
	- trace sand, gravel		21	SS	34											
						218										
						217										
			22	SS	81											
						216										
	- some sand, gravel		23	SS	81											
215.4					12/15 mm											
21.3	Refusal on suspected bedrock/boulders at 21.3 m bgs															Spoon Bouncing, Auger Refusal

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

RECORD OF BOREHOLE No. 2

1 OF 2

METRIC

W.P. GWP 6176-15-00 LOCATION 20+381, 4.9 Lt, Blake Twp. ORIGINATED BY RT
 DIST Thunder Bay HWY 61 BOREHOLE TYPE Hollow Stem Auger COMPILED BY DM
 DATUM Geodetic DATE 2021.10.15 - 2021.10.18 MTM Zone 15 343124.967 E 5345622.402 N
 LATITUDE 48.248223 LONGITUDE -89.483902 CHECKED BY FS

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		20	40	60	80	100					
237.1	40 mm asphalt		1	AS		237										
236.4	FILL - SAND - some gravel, trace silt, brown															
0.8	FILL - SAND - some gravel, silt, brown, compact		2	SS	14											
235.6	FILL - SILTY SAND - some gravel, trace to some clay, brown, moist to wet, compact to loose		3	SS	13											
1.5			4	SS	7											
			5	SS	10											
			6	SS	8											
232.5	FILL - SILTY CLAY - some sand, gravel, brown/red, moist, firm		7	SS	6											
4.6	- silt layer, some clay, sand, gravel, grey, moist		8	SS	8											
	- sandy, some gravel, brown/red, moist		9	SS	7											
230.3	FILL - SAND - some silt, gravel, trace clay, brown, dry, compact to loose		10	SS	13											
6.9	- damp		11	SS	8											
			12	SS	7											
			13	SS	8											
226.4	SILT - some gravel, with sand, some clay, grey, damp, loose		15	SS	8											
10.7	- some gravel, clay, brown, damp, compact, low plasticity		16	SS	22											
224.9	SILTY SAND - trace gravel, clay, grey, moist to wet, compact		17	SS	13											
12.2			18	SS	12											
			19	SS	12											

Continued Next Page

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

ONTARIO MTO GWP 6176-15-00 - HIGHWAY 61 - CULVERT 20+375.GPJ ONTARIO MTO.GDT 22-4-6

RECORD OF BOREHOLE No. 2

2 OF 2

METRIC

W.P. GWP 6176-15-00 LOCATION 20+381, 4.9 Lt, Blake Twp. ORIGINATED BY RT
DIST Thunder Bay HWY 61 BOREHOLE TYPE Hollow Stem Auger COMPILED BY DM
DATUM Geodetic DATE 2021.10.15 - 2021.10.18 MTM Zone 15 343124.967 E 5345622.402 N
LATITUDE 48.248223 LONGITUDE -89.483902 CHECKED BY FS

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		× LAB VANE									
						20	40	60	80	100									
221.9																			
15.2	SAND - trace silt, gravel, clay, grey, wet, compact		20	SS	13										Start Wash Boring				
			21	NR	12														
218.8															1 11 81 7				
18.3	SILT - some sand, trace gravel, clay, grey, wet, compact to very dense		22	SS	21														
	- some gravel		23	SS	64														
216.2															Auger Refusal, Start Coring				
20.9	BEDROCK - Argillite-tuff Gunflint Formation RQD = 85.0 %		1	RC															
215.0																			
22.1	End of Borhole at 22.1 m bgs																		
















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

RECORD OF BOREHOLE No. 3

1 OF 1

METRIC

W.P. GWP 6176-15-00 LOCATION 20+370, 25.0 m Rt, Blake Twp. ORIGINATED BY MQ
 DIST Thunder Bay HWY 61 BOREHOLE TYPE Solid Stem Auger COMPILED BY DM
 DATUM Geodetic DATE 2021.11.12 - 2021.11.12 MTM Zone 15 343160.15 E 5345613.841 N
 LATITUDE 48.248148 LONGITUDE -89.484261 CHECKED BY FS

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	× LAB VANE
228.0								20	40	60	80	100								
0.0	CLAYEY SILT - with sand, gravel, brown, wet		1	AS																
227.2																				
0.8	SILT - some sand, clay, trace gravel, brown, wet, loose to compact		2	SS	14		227										Groundwater Level 1.1 m bgs (Elevation 226.9 m) Nov 22, 2021			
			3	SS	10		226										Groundwater Level 1.4 m bgs (Elevation 226.6 m) Dec 21, 2021			
			4	SS	7		225										4 11 69 15			
			5	SS	4		225													
	- trace sand, clay		6	SS	10		224										0 4 89 7			
			7	SS	15		223													
	- some sand, trace clay, gravel, grey, wet, loose to compact		8	SS	8		223													
			9	SS	11		222										1 13 77 9			
221.3																				
6.7	End of Borehole at 6.7 m bgs																			

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

RECORD OF BOREHOLE No. 4

1 OF 1

METRIC

W.P. GWP 6176-15-00 LOCATION 20+385, 30.0 m Lt, Blake ORIGINATED BY MQ
 DIST Thunder Bay HWY 61 BOREHOLE TYPE Solid Stem Auger COMPILED BY DM
 DATUM Geodetic DATE 2021.11.10 - 2021.11.10 MTM Zone 15 343098.389 E 5345623.065 N
 LATITUDE 48.248231 LONGITUDE -89.48426 CHECKED BY FS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				W _p	W	W _L		GR	SA	SI	CL
226.6 0.0	100 mm organic soil SILT - with clay, some sand, trace gravel, brown, damp, loose to dense - low plasticity - with clay, brown/grey, wet - trace sand, grey - some sand, clay, trace gravel - wet		1	AS															
			2	SS	11														
			3	SS	14														
			4	SS	6														
			5	SS	19														
			6	SS	28														
			7	SS	31														
221.3 5.3	SILTY SAND - some clay, trace gravel, grey, wet, compact to loose		8	SS	14														
			9	SS	7														
219.9 6.7	End of Borhole at 6.7 m bgs																		

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

Appendix C

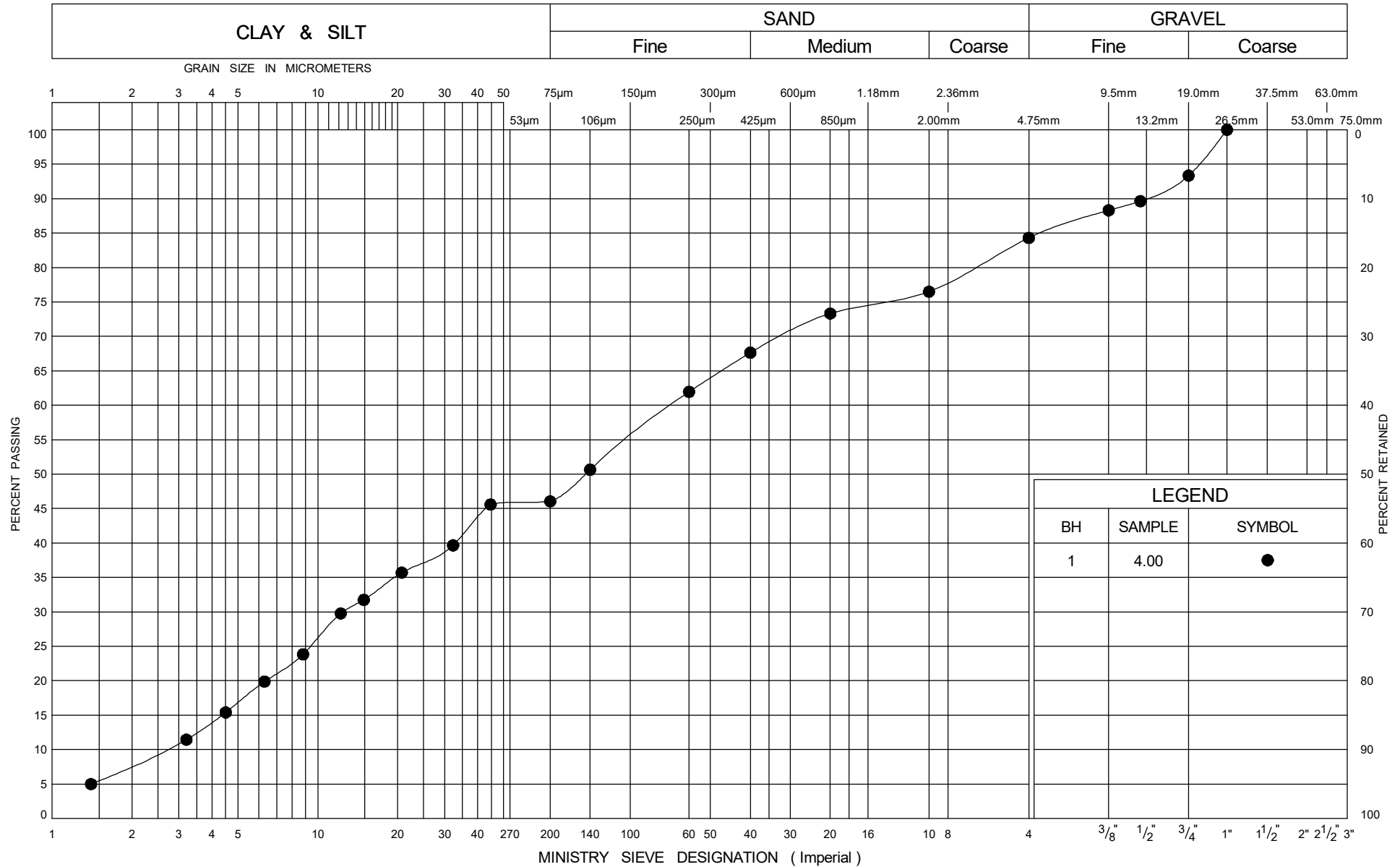
Borehole Plan and Laboratory Data

Figure No. L-1: Fill: Silt and Sand Grain Size Distribution Curve
Figure No. L-2: Fill: Silty Sand Grain Size Distribution Curve
Figure No. L-3: Fill: Silt and Clay Grain Size Distribution Curve
Figure No. L-4: Fill: Sand Grain Size Distribution Curve
Figure No. L-5: Silt Grain Size Distribution Curve
Figure No. L-6: Silt Grain Size Distribution Curve
Figure No. L-7: Sandy Silt Grain Size Distribution Curve
Figure No. L-8: Atterberg Limits Summary

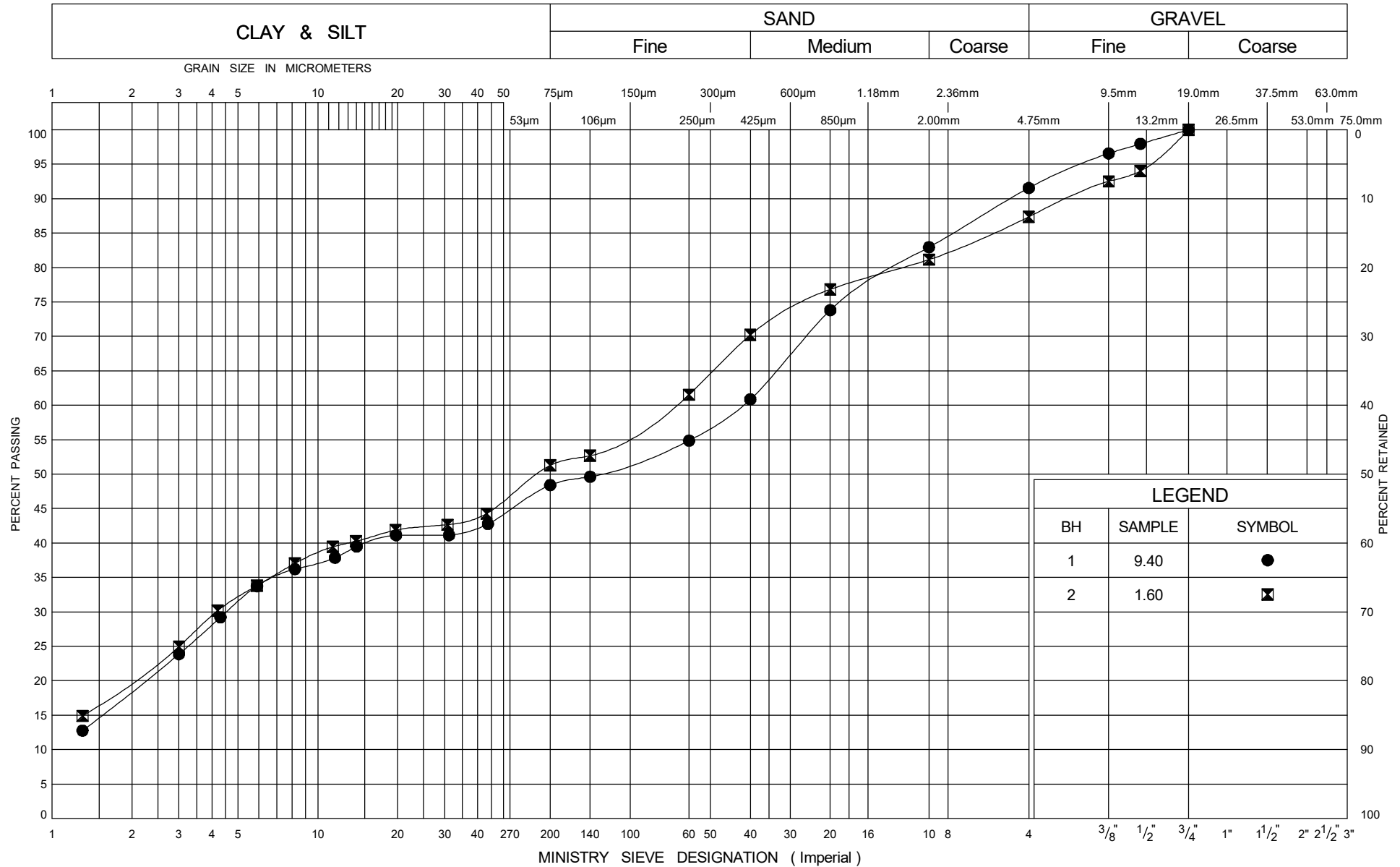


eNGLOBE

UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

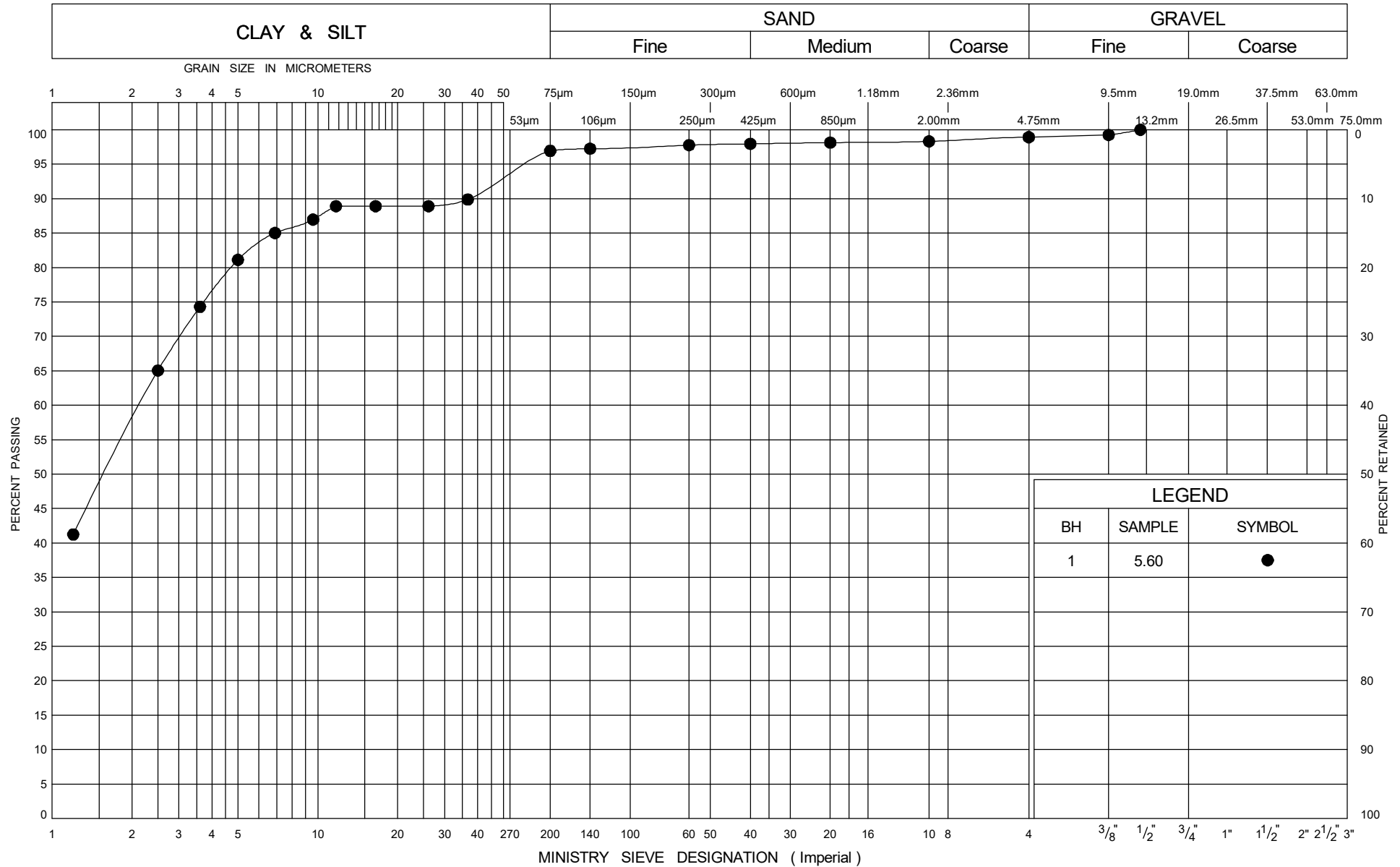
FILL - SILTY SAND

Figure L-2

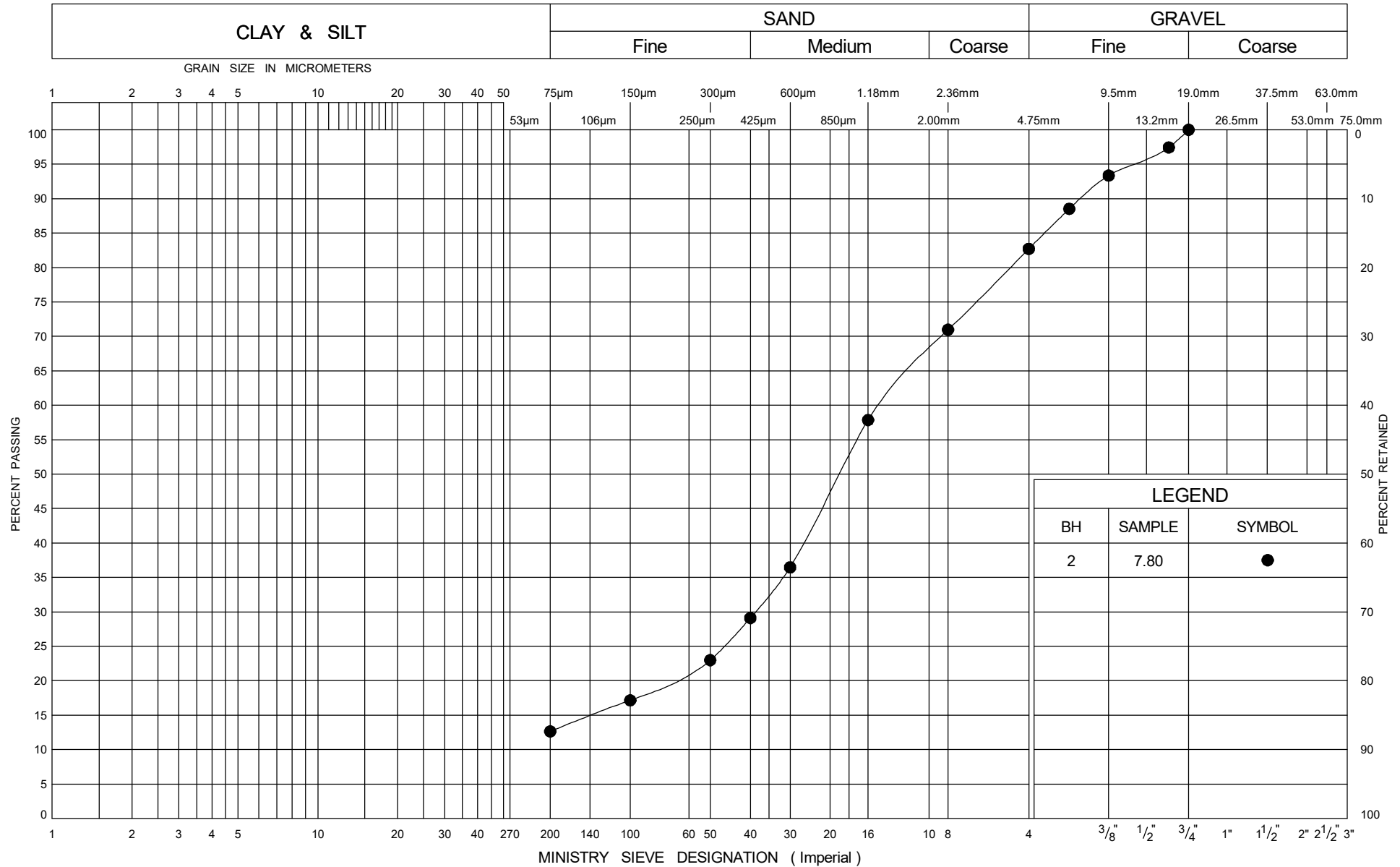
GWP 6176-15-00

Highway 61, NWR

UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

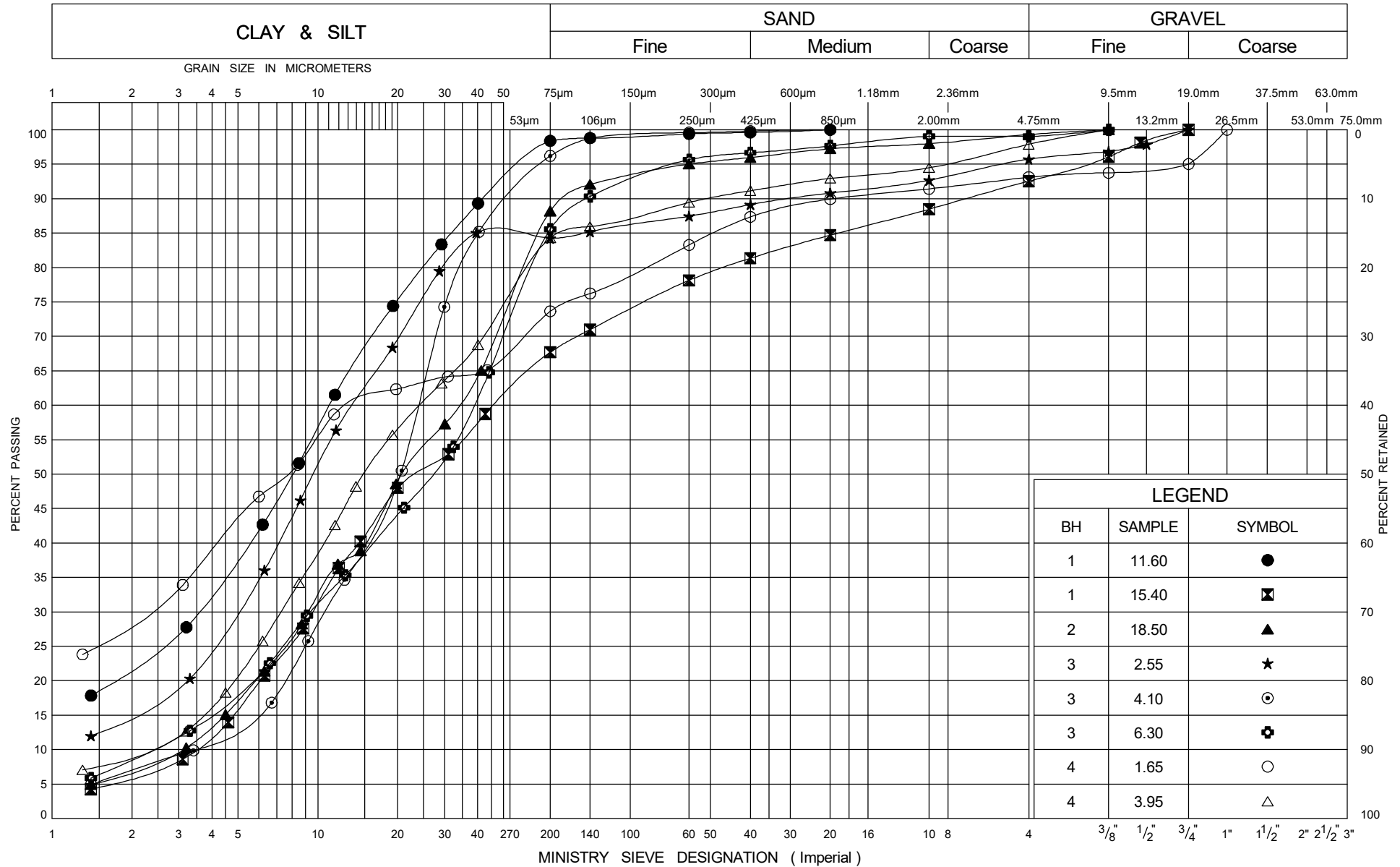
FILL - SAND

Figure L-4

GWP 6176-15-00

Highway 61, NWR

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

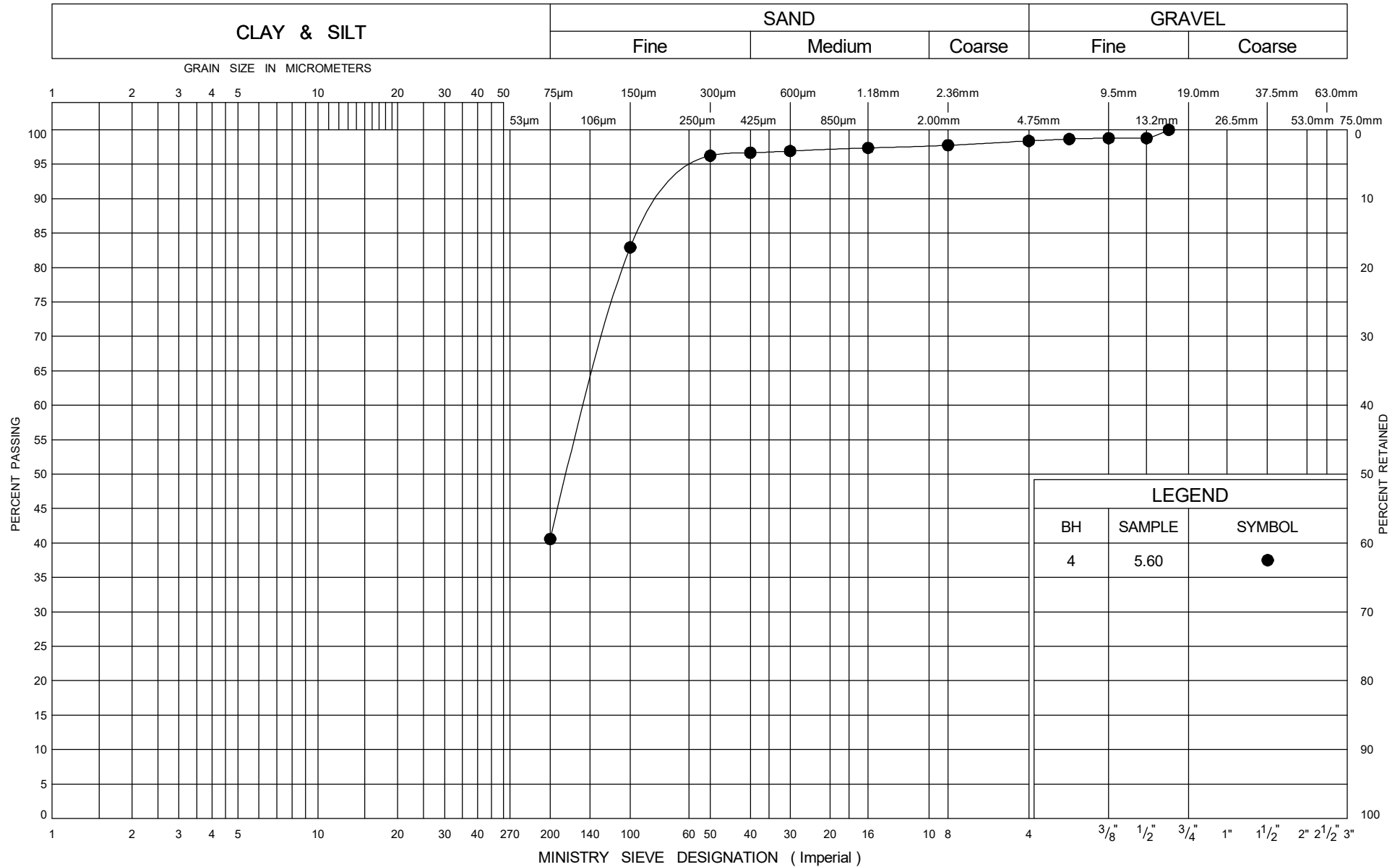
SILT

Figure L-5

GWP 6176-15-00

Highway 61, NWR

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

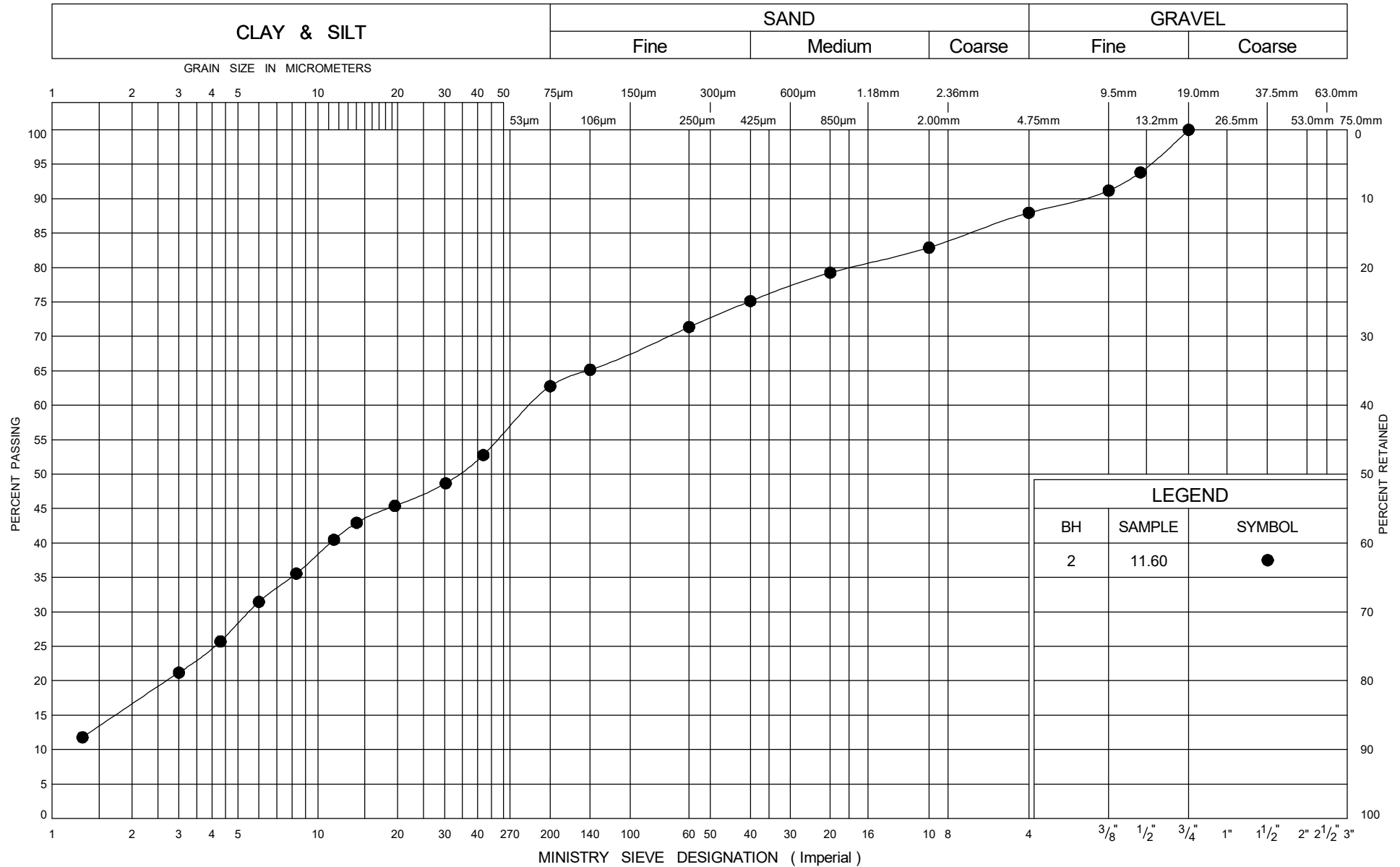
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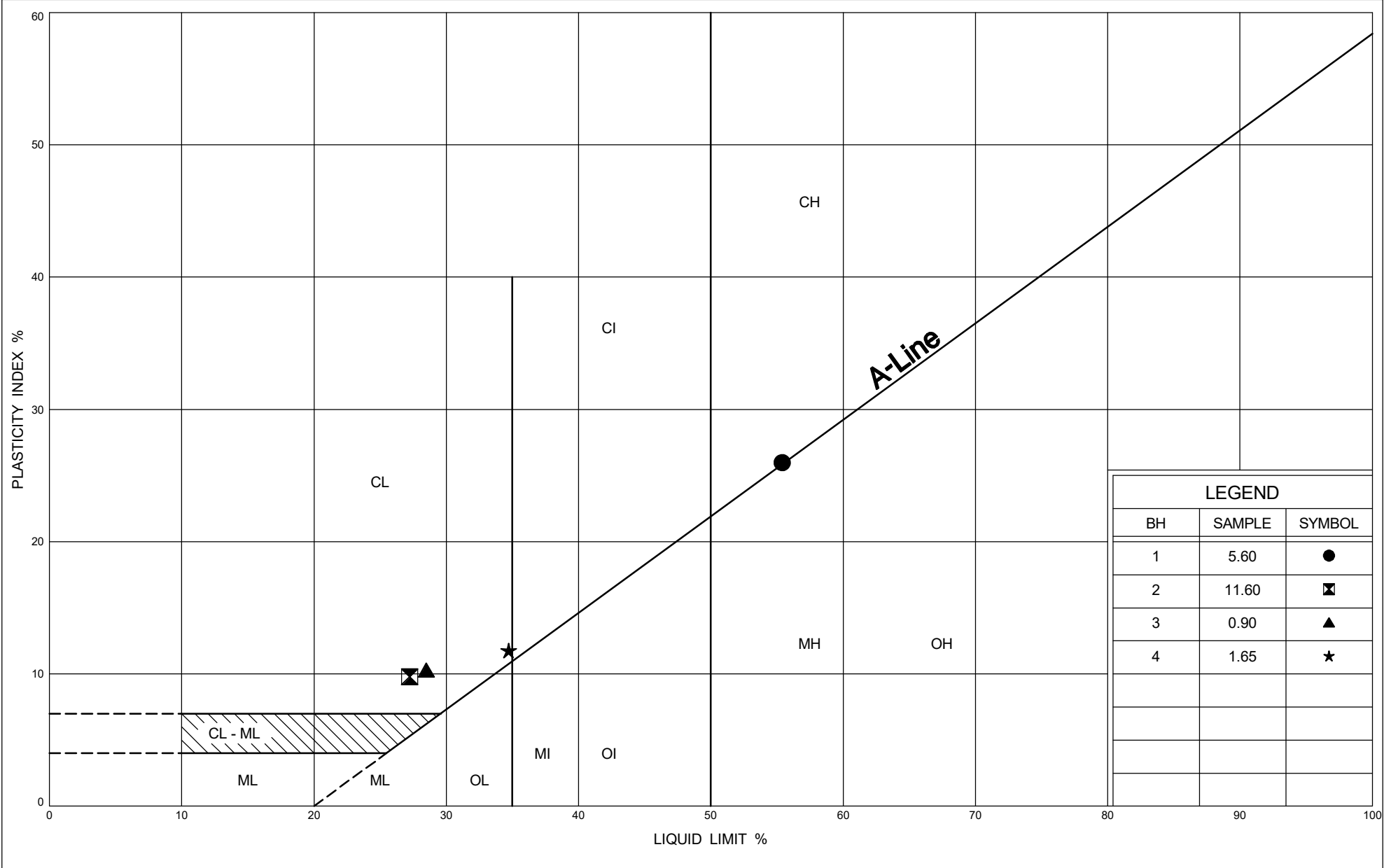
Figure L-6

GWP 6176-15-00

Highway 61, NWR

UNIFIED SOIL CLASSIFICATION SYSTEM



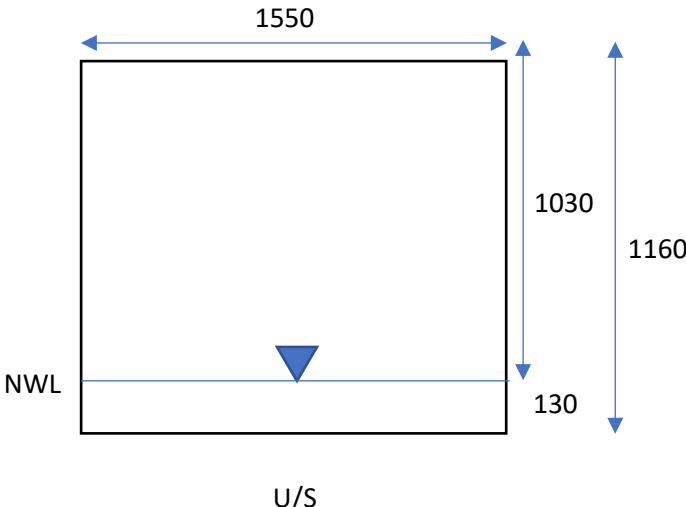
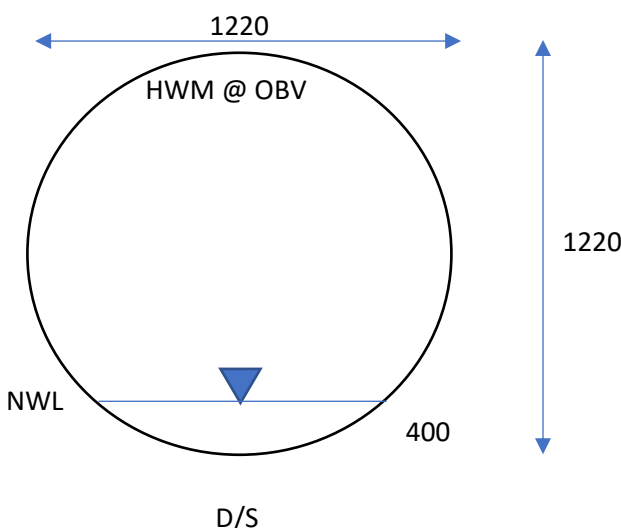


Appendix D
Culvert Inspection Report
(as provided by Gannett Fleming)



eNGLOBE

FIELD INSPECTION FORM

A. GENERAL INFORMATION			
Project #	6176-15-00 - Highway 61	Project Description	From 0.5km north of Jarvis Bay Road to 0.4km South of Hwy 130
Date	October 5, 2021	Weather Conditions	Sunny
Inspector 1	David Jackson	Inspector 2 /Reviewer	-
B. CULVERT ID / LOCATION			
Culvert ID	C7	Chainage	20+375
UTM Easting	343132.7421	UTM Northing	5345617.6244
Description	South of the Highway 608 & Highway 61 intersection		
C. STRUCTURE DETAILS			
Material – CTC / CSP			
Dimensions – 1550 x 1160 US / 1220 x 1220 DS			
Clearance (soffit to normal water level) – 1030 mm / 822mm			
High Water Mark (on structure) – N/A			
Structures (U/S / D/S of Crossing) – N/A			
Debris – Gravel/debris within the barrel			
D. ENVIRONMENTAL CONDITIONS			
Watercourse Type and Creek Material – Stones and pebbles			
Bank Conditions (stability) – Minor bank and embankment erosion			
Channel Dimensions (width and depth) – 2m, 3:1, 130mm US / 5m, 3:1, 400mm of water DS			
Observed Flow Conditions (ephemeral/permanent) – Permanent			
E. SITE CONDITIONS			
Road Condition (sag, settlement, etc.) – OK			
Physical Culvert Condition (rust, damage, etc.) – OK			
Culvert Appearance (general comments) – Replace			
Site Sketch – <div style="display: flex; justify-content: space-around; align-items: flex-end; margin-top: 20px;"> <div style="text-align: center;">  <p>U/S</p> </div> <div style="text-align: center;">  <p>D/S</p> </div> </div>			

Creosote Timber Culvert and Corrugated Steel Pipe (Culvert #7) @ 20+375

C7 - #1 – Upstream Channel Conditions



C7 - #2 – Upstream Face of the Culvert

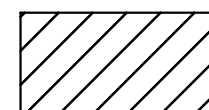
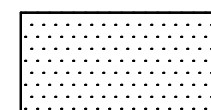
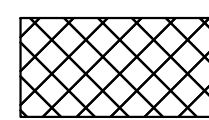
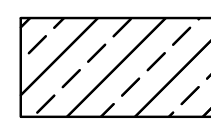


C7 - #3 – Downstream Channel Conditions



C7 - #4 – Downstream Face of the Culvert





METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

PLATE	No	----
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CONT	-----
WP	6176-15-00

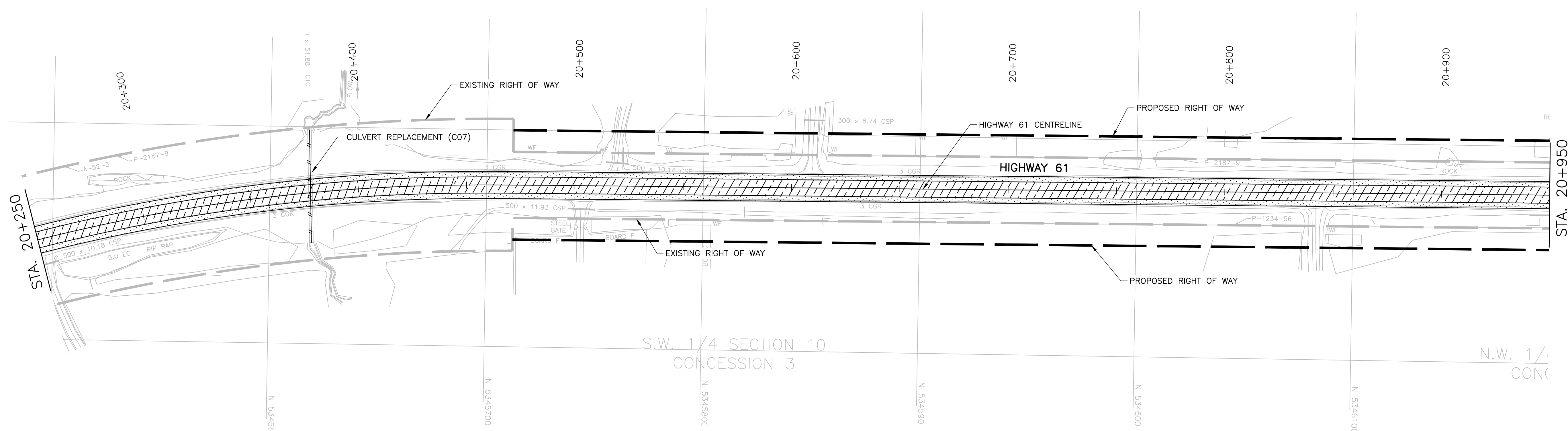
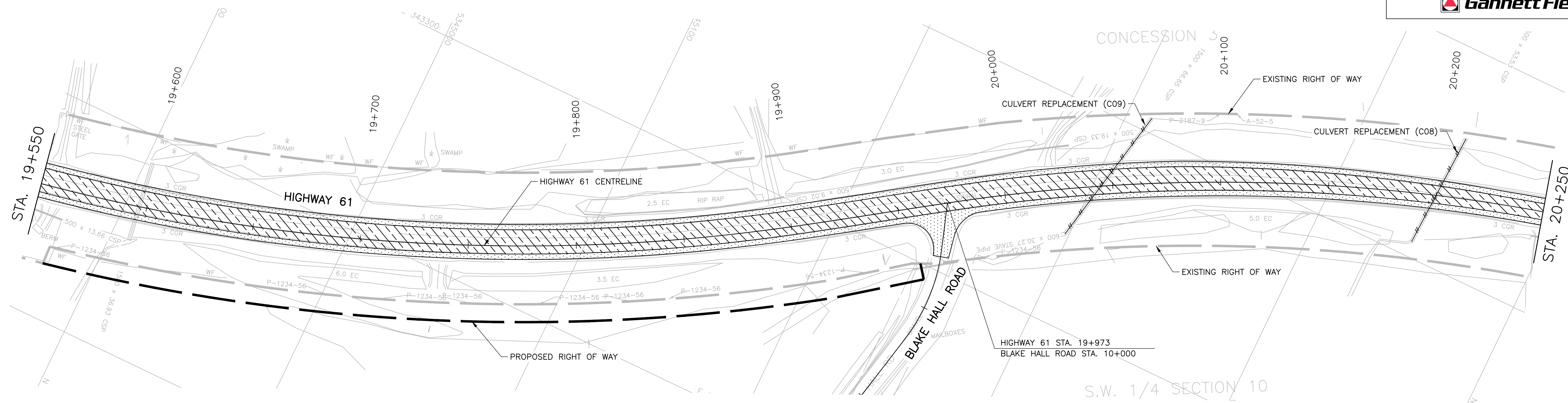
REMOVALS

STA 19+550 TO STA 20+950

Survey	Revised
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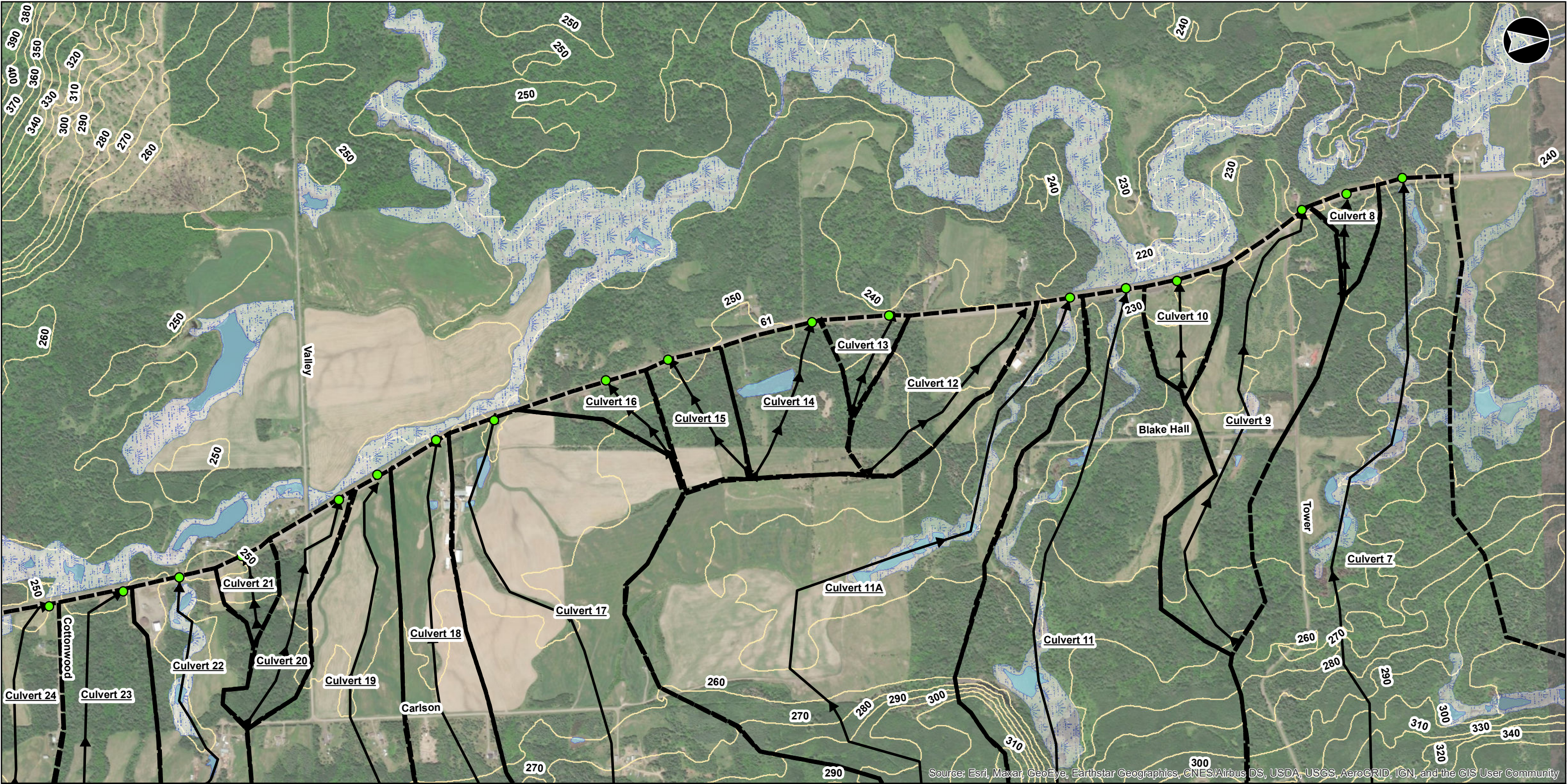
SHEET

20

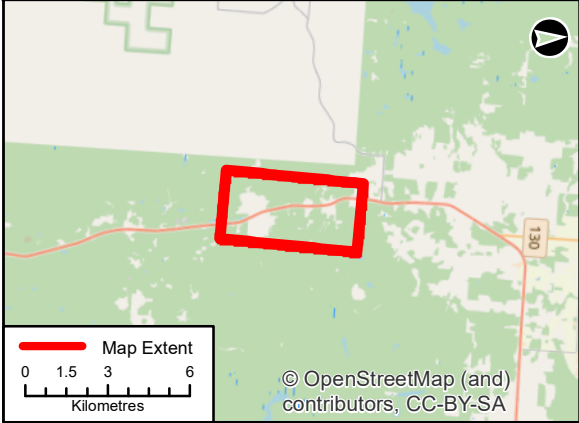


SCALE
10m 0 20m

Appendix B: Catchment Plans




Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

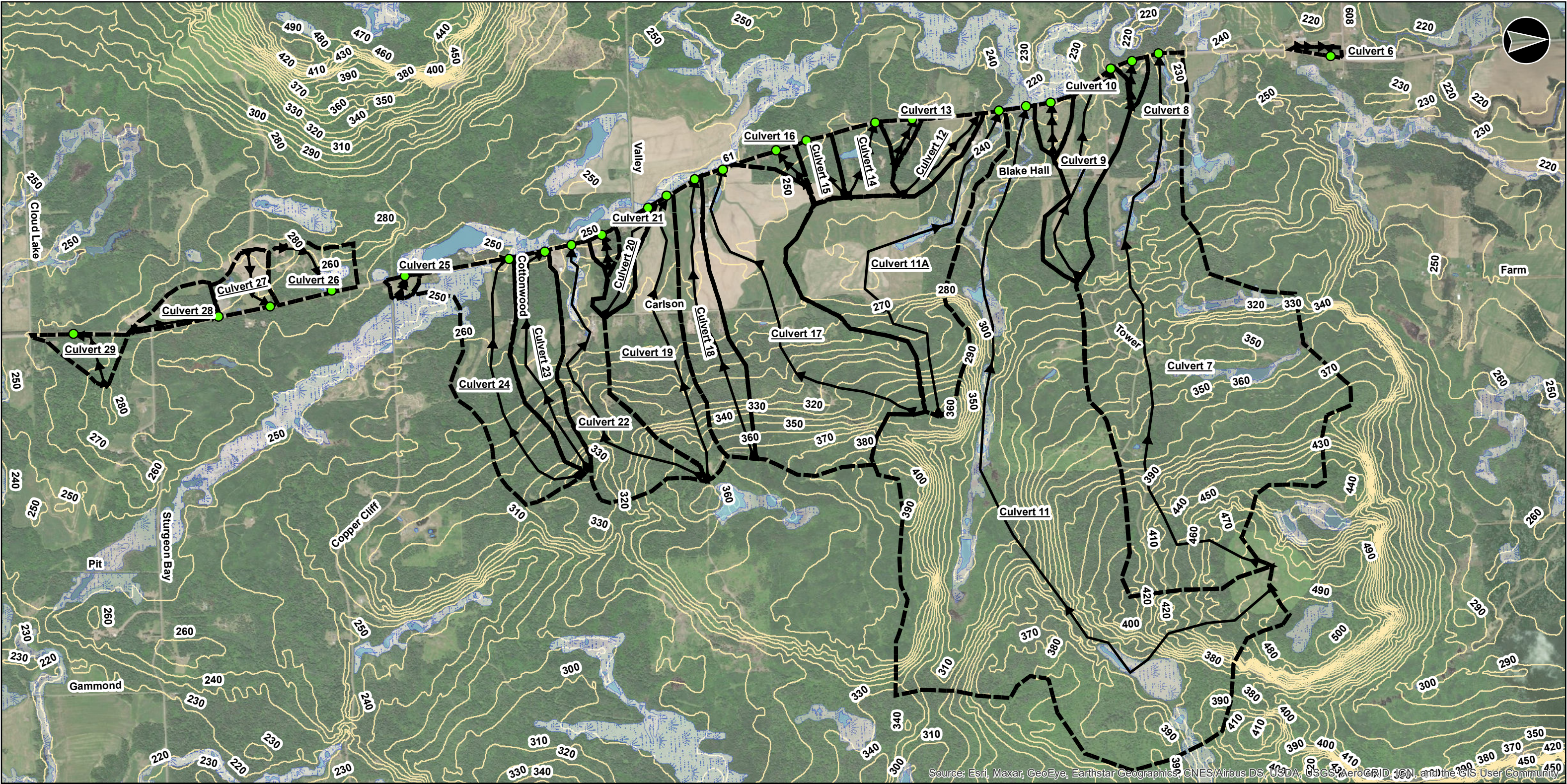


Legend

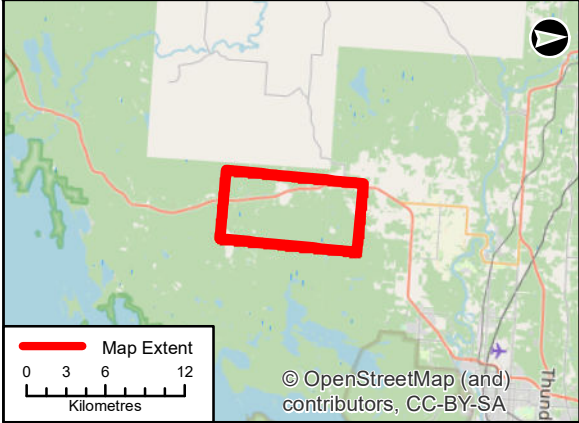
- Culvert Crossings
- Flow Path
- Contour
- Catchments
- Wetlands
- Waterbodies

Highway 61 Rehabilitation GWP# 6176-15-00, NWR Catchment Area Plan			
<div><div><div></div><div></div><div></div><div></div><div></div></div><div>0150300</div><div>Metres</div></div> <div>Datum: NAD 1983 UTM Zone 16N</div>	Figure 2		Ontario 
Data Sources:	Dec, 2021	1:12,000	
	.	Rev Draft	

Highway 61 Rehabilitation Catchment Area Plan
Date Saved: 12/21/2021 2:43:46 PM User Name: dijagason



Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community



Legend

- Culvert Crossings
- Flow Path
- Contour
- Catchments
- Wetlands
- Waterbodies

Highway 61 Rehabilitation GWP# 6176-15-00, NWR Catchment Area Plan

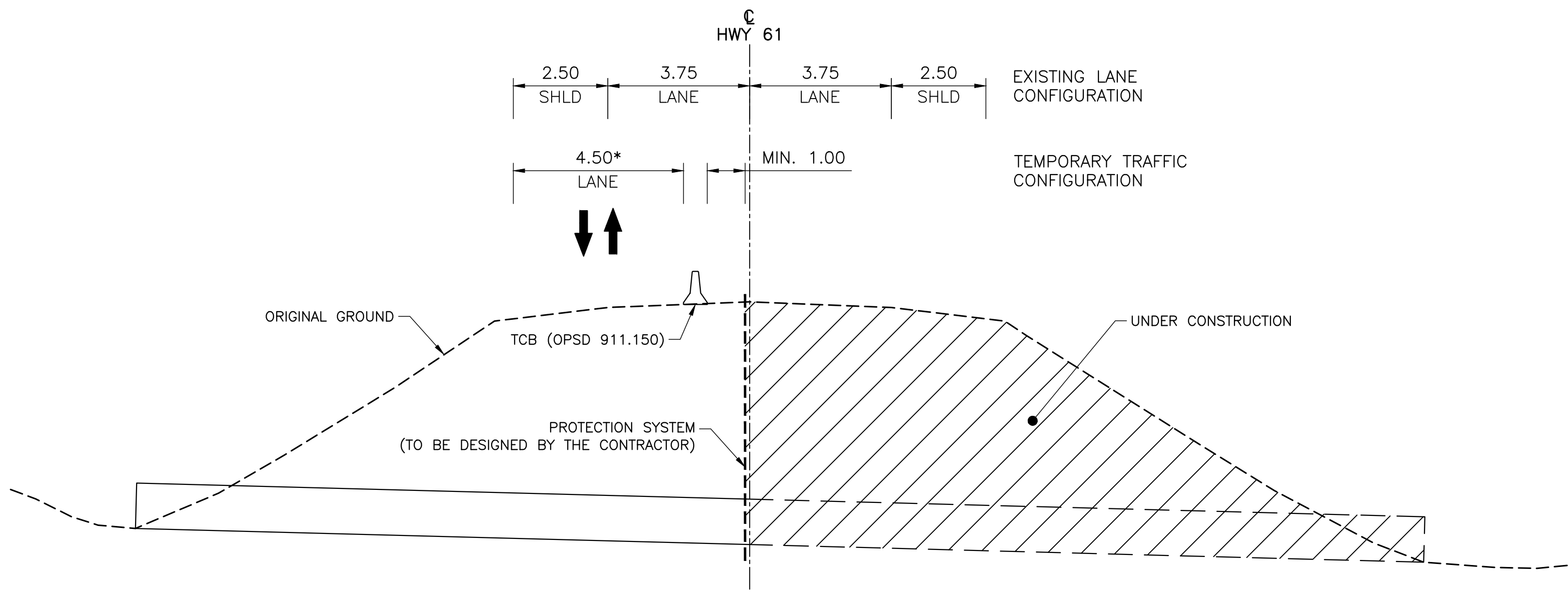
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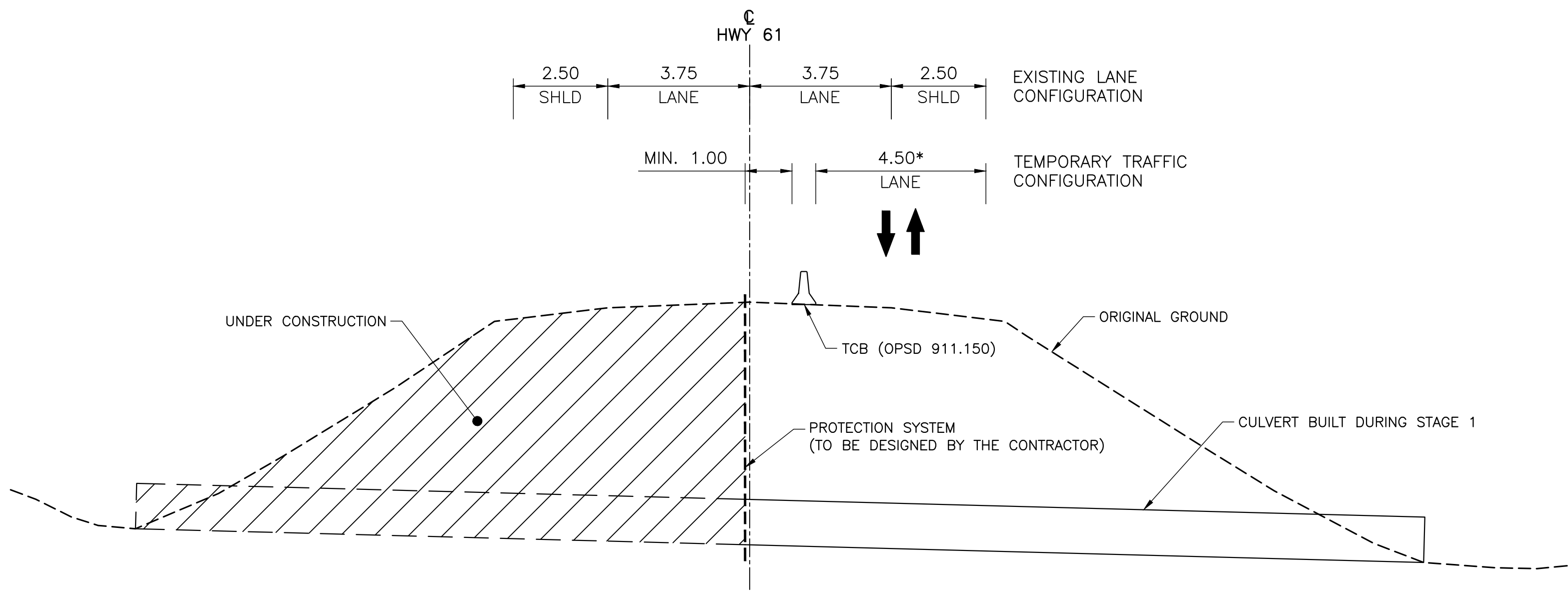
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MINISTRY OF TRANSPORTATION, ONTARIO
PR-D-707
88-05

HIGHWAY 61 CULVERT REPLACEMENT STAGING (WITH ROADWAY PROTECTION)



STAGE 1 – TYPICAL SECTION
* 4.50m INCLUDES 3.50m TRAVEL LANE AND 2 X 0.5m PAVED SHOULDERS



STAGE 1 – TYPICAL SECTION
* 4.50m INCLUDES 3.50m TRAVEL LANE AND 2 X 0.5m PAVED SHOULDERS

N.T.S.

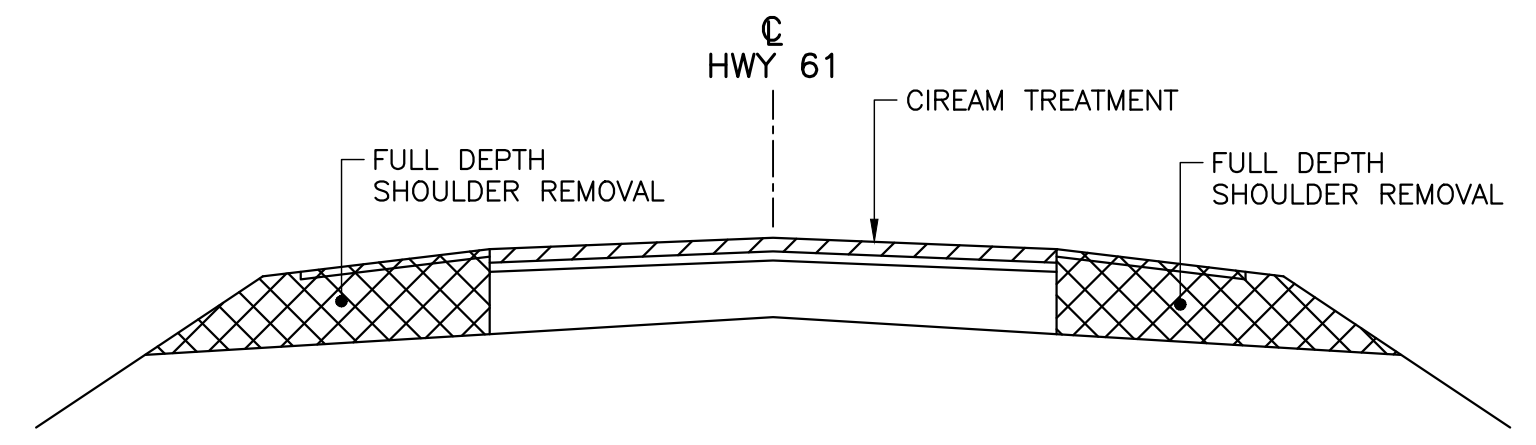
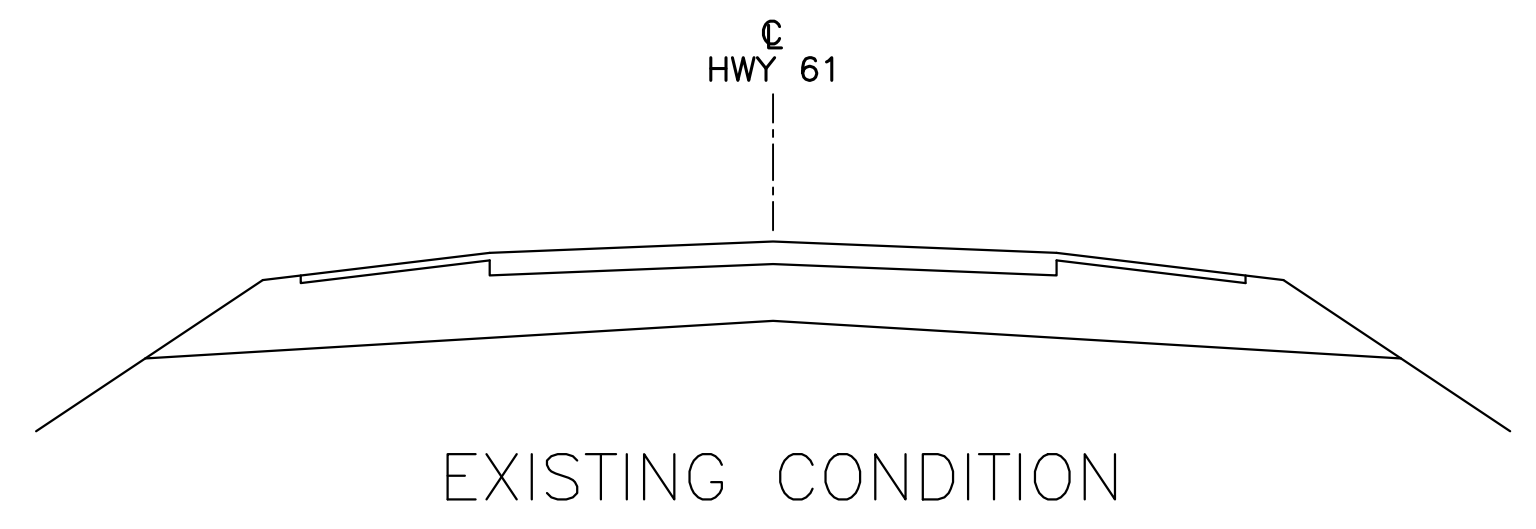
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AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

PLATE No	----
CONT WP	6176-15-00
CONSTRUCTION STAGING TYPICALS	
Survey	Revised

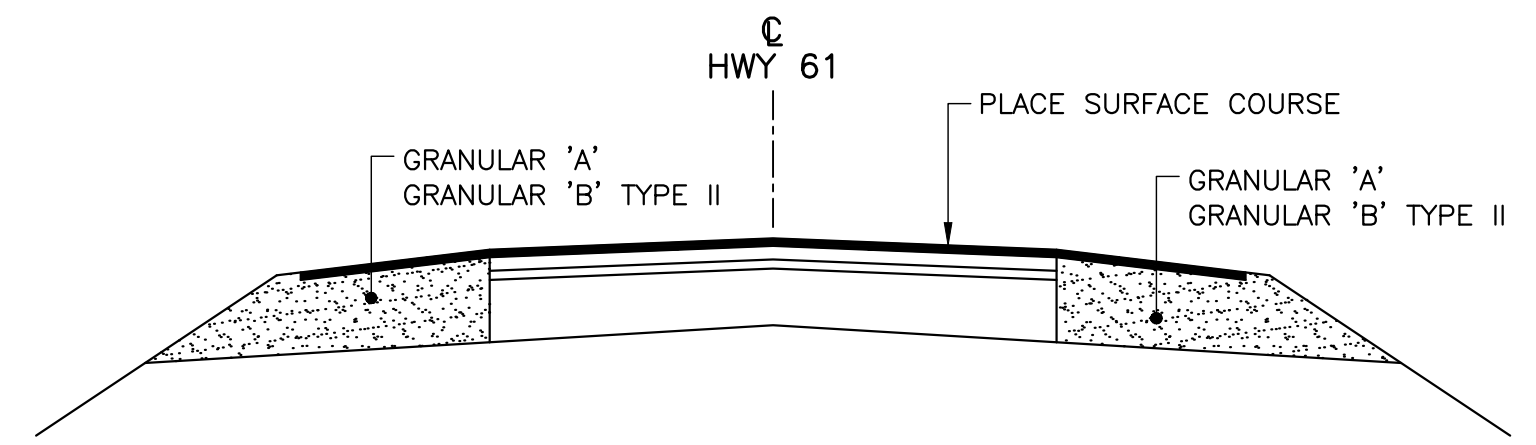
Gannett Fleming

SHEET 10

HIGHWAY 61 PAVEMENT REHABILITATION SEQUENCING



SEQUENCE 1
REMOVAL WORKS INCLUDING CIREAM TREATMENT AND FULL DEPTH SHOULDER REMOVAL. GUIDE RAIL REMOVAL AND STRIPPING WHERE PLATFORM WIDENING IS REQUIRED.



SEQUENCE 2
PAVE TOP LIFT ASPHALT (50mm GRAD RAISE). APPLY GRANULAR SEALING AND INSTALL NEW STEEL BEAM GUIDE RAILS WHERE REQUIRED.

NOTE:
TRAFFIC CONTROL AS PER OTM BOOK 7

N.T.S.

Appendix E

Non-Standard Special Provision (NSSP) - Potential Obstructions in Subsurface Soils



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NSSP – Potential Obstructions in Subsurface Soils

Special Provision

The Contractor is notified that, during foundation field investigations for the culvert located at STA 20+375 located on Highway 61 in Township of Blake, rock fragments (cobble to boulder) and concrete debris material have been encountered buried near the bottom of the embankment fill. The Contractor shall take into account these obstructions in soils for designing and constructing of trenchless techniques and the temporary protection system.

Appendix F

Non-Standard Special Provision (NSSP) - Pipe Installation by Trenchless Method



eNGLOBE

PIPE INSTALLATION BY TRENCHLESS METHOD – Item No.

Special Provision

CONSTRUCTION SPECIFICATION FOR THE INSTALLATION OF PIPES BY TRENCHLESS METHOD

1.0 SCOPE

This Special Provision covers the requirements for the installation of pipes by a selected trenchless method.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications, or publications:

Ontario Provincial Standard Specifications, General

OPSS 180 General Specification for the Management of Excess Materials

Ontario Provincial Standard Specifications, Construction

OPSS 182 Environmental Protection for Construction in Waterbodies and On Waterbody Banks
OPSS 401 Trenching, Backfilling, and Compacting
OPSS 402 Excavating, Backfilling, and Compacting for Maintenance Holes, Catch Basins, Ditch Inlets
and Valve Chambers
OPSS 403 Rock Excavation for Pipelines, Utilities, and Associated Structures in Open Cut
OPSS 404 Construction Specification for Support Systems
OPSS 409 Closed-Circuit Television (CCTV) Inspection of Pipelines
OPSS 490 Site Preparation for Pipelines, Utilities, and Associated Structures
OPSS 491 Preservation, Protection, and Reconstruction of Existing Facilities
OPSS 492 Site Restoration Following Installation of Pipelines, Utilities and Associated Structures
OPSS 510 Construction Specification for Removal
OPSS 517 Construction Specification for Dewatering
OPSS 539 Construction Specification for Temporary Protection Systems

Ontario Provincial Standard Specifications, Material

OPSS 1004 Material Specification for Aggregates - Miscellaneous
OPSS 1350 Material Specification for Concrete - Materials and Production
OPSS 1440 Steel Reinforcement for Concrete
OPSS 1802 Material Specification for Smooth Walled Steel Pipe
OPSS 1820 Material Specification for Circular and Elliptical Concrete Pipe
OPSS 1840 Material Specification for Non-Pressure Polyethylene (PE) Plastic Pipe Products
OPSS 1841 Material Specification for Non-Pressure Polyvinyl Chloride (PVC) Plastic Pipe Products

CSA Standards

A3000 Cementitious Materials Compendium
B182.6 Profile polyethylene (PE) sewer pipe and fittings for leak-proof sewer applications

B182.8	Profile Polyethylene (PE) Storm Sewer and Drainage Pipe and Fittings
B182.13	Profile Polypropylene (PP) Sewer Pipe and Fittings for Leak-proof Sewer Applications
C22.1	Canadian Electrical Code
W59	Welded Steel Construction

American Society for Testing and Materials (ASTM) International Standards

A 252M-19	Standard Specification for Welded and Seamless Steel Pipe Piles
C-33	Standard Specification for Concrete Aggregates.
C-39	Standard Test method for Compressive Strength of Cylindrical Concrete
D 2657	Standard Practice for Heat Fusion Joining of Polyolefin Pipe and Fittings
D 3350	Standard Specification for Polyethylene Plastics Pipe and Fittings Materials
D6910	Standard Specification for Marsh Funnel Viscosity of Clay Construction Slurries
F 894	Standard Specification for Polyethylene Large Diameter Profile Wall Sewer and Drain Pipe

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

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

For the purpose of this Special Provision, the following definitions apply:

Annular Space means the space between the inside edge of the opening and the outside edge of the penetrating item or inserted pipe.

Auger Jack & Bore means a method of forming a horizontal bore in the subsurface by simultaneously or alternately jacking into the ground a casing pipe and rotating a cutter head at the lead end of an auger flight with removal of material from inside the casing by using continuous-flight augers.

Backreamer or Reamer means a cutting head suitably designed for the subsurface conditions that is attached to drilling equipment and used to enlarge the bore

Bore Path means a drilled path according to the grade and alignment tolerances specified in the Contract Documents.

Boulder Number Ratio (BNR) means the number of individual boulders per m³ of cumulative boulder volume.

Boulder Volume Ratio (BVR) means the ratio between the cumulative volume of boulders and the volume of the material excavated.

Design Engineer means the Engineer retained by the Contractor who produces the design and Working Drawings and other engineering documents required of the Contractor. The Design Engineer shall be licensed to practice in the Province of Ontario.

Design Checking Engineer means the Engineer retained by the Contractor who checks the original design and Working Drawings.

Digger Shield/Hand Mining means a method of forming a horizontal bore in the subsurface by essentially simultaneously jacking a casing pipe, with or without a protective shield at the lead end, into the ground while

tunnelling and removal of earth and rock is completed using manually-operated tools (e.g., pneumatic spades, rams, shovels, breaker bars, etc.) or a “digger” type shield with a hydraulic excavator arm or “road-header” rock cutting machine to remove materials from inside the shield and liner pipe.

Drilling Fluids means a mixture of water and additives, such as bentonite, polymers, surfactants, and soda ash, designed to block the pore space on a bore wall, reduce friction in the bore, and to suspend and carry cuttings to the surface.

Drilling Fluid Hydraulic Fracture or “Frac Out” means a condition where the drilling fluid’s pressure in the bore is sufficient to fracture the soil and/or rock materials and allow the drilling fluids to migrate to the surface at an unplanned location.

Earth Pressure Balance (EPB) means a tunnelling system that provides support to the excavated face of the ground and resistance to groundwater inflow through the pressure of mixed earth, rock and any drilling fluids or additives (spoil) as maintained by and in a chamber behind the cutting face of a tunnel boring machine through which spoil can pass only by manner of controlled-load relieving gates or an internal screw-conveyor that is separate from subsequent spoil conveyance systems (e.g., flight augers, belt conveyor, spoil bucket rail cars, etc.). Trenchless systems that apply pressure to the excavated face of the ground only through mechanical and jacking forces on metal parts of the machinery (e.g., steel parts of cutting tools, adjustable gates or doors at cutting face, etc.) will not be considered equivalent to EPB systems.

Excavation means all materials encountered regardless of type and extent and shall include removal of natural soil, boulders, cobbles, wood and fill regardless of means necessary to break consolidated materials for removal.

Environmentally Sensitive Area (ESA) means areas specified in the Contract Documents that are prohibited from entry or use.

Fill means man-made mixture of previously placed or handled materials such as sand, clay, silt, gravel, broken rock, sometimes containing organic and/or deleterious materials, placed in an excavation or other area to raise the surface elevation.

Guidance System means an electronic system capable of indicating the position, depth and orientation of the drill head during the directional drilling process.

Hand Mining means a method of forming a horizontal bore in the subsurface by simultaneously jacking ahead while tunnelling advances using hand-mining (man-entry operation or “Jack and Mine”) or a “digger” type shield with a hydraulic excavator arm to remove materials from inside the liner pipe.

Horizontal Directional Drilling (HDD) means a surface-launched trenchless technology for the installation of pipes, conduits, and cables. HDD creates a pilot bore along the design pathway and reams the pilot bore in one or more passes to a diameter suitable for the product, which is pulled into the prepared bore in the final steps of the process.

Inadvertent Returns means the unexpected flow of fluids, saturated materials (or flowing soil) towards the drilling rig that typically originated from an artesian aquifer encountered during the drilling process.

Loss of Circulation means the discontinuation of the flow of drilling fluid in the bore back to the entry or exit point or other planned recovery points.

Microtunnelling means an underground method of constructing a passage by using a microtunnelling boring machine (MTBM) or hand mining using a shield to support the opening.

MTBM means a microtunnelling boring machine.

Pilot Bore means the initial bore to set directional controlled horizontal and vertical alignment between the connecting points.

Pipe means pipe culverts, pipe storm and sanitary sewers, watermain pipe, conduits, and ducts.

Pipe Jacking means a method for installing steel casing, concrete pipe or other acceptable material in the subsurface utilizing hydraulically operated jacks of adequate number and capacity for the smooth and uniform advancement of the casing or pipe.

Pipe Ramming means a method for installing steel casings utilizing the energy from a percussion hammer to advance a steel casing with a cutting shoe attached at the front end of the casing.

Project Superintendent means an individual representing the Contractor that oversees the trenchless or tunnelling operation qualified to provide the services specified in the Contract Documents.

Pullback means that part of the HDD method in which the drilling equipment is pulled back through the bore path to the entry point.

Reaming means a process for enlarging the bore path.

Rock means natural beds or massive fragments, or the hard, stable, cemented part of the earth's crust, igneous, metamorphic, or sedimentary in origin, which may or may not be weathered and includes boulders having a volume of 0.5 m³ or greater.

Shaft means an excavation used as entry and/or exit points, alternatively called entry/exit pits, from which the trenchless method is initiated for the installation of the pipe product.

Slurry Pressure Balance (SPB) means a tunnelling system that provides support to the excavated face of the ground and resistance to groundwater inflow through the pressure of slurry as maintained by and in a chamber behind the cutting face of a tunnel boring machine (TBM) or microtunnelling boring machine (MTBM), through which spoil can pass only by manner of controlled-pressure and controlled flow slurry pumping systems.

Slurry means a mixture of soil and/or rock cuttings, and drilling fluid.

Soil means all soils except those defined as rock, and excludes stone masonry, concrete, and other manufactured materials.

Spoil means mix of earth cuttings, rock cuttings, water (groundwater or added water), bentonite, polymers and/or other additives that is discharged from the trenchless construction systems.

Strike Alert means a system that is intended to alert and protect the operator in the case of inadvertent drilling into an electrical utility cable. The strike alert system consists of a sensor and an alarm connected to the drill rig and a grounding stake. The alarm may be audio or visual or both.

TBM means a tunnel boring machine.

Trenchless Contractor means the subcontractor retained by the Prime Contractor qualified to provide the services specified in the Contract Documents.

Trenchless Installation means an underground method of constructing a passage open at both ends that involves installing a pipe product by auger jack & boring, pipe ramming, horizontal directional drilling, or tunnelling.

Tunnelling means an underground method of constructing a passage using a tunnel boring machine (TBM) operated by personnel within the tunnel, a microtunnelling boring machine (MTBM) operated by personnel at a remote control station or excavation using a shield to support the opening and protect workers.

Zone of Influence means a zone defined by lines projected outward and upward at 45 degrees from horizontal to the ground surface from the vertical and horizontal alignment of the pipe constructed using trenchless/tunnel methods.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design

4.01.01 General

The Contractor shall determine the most appropriate method of trenchless installation for each pipe crossing for each location within the terms of this specification.

The trenchless installation method selected for each pipe crossing shall be designed for the subsurface conditions in accordance with the Contract Documents.

The detailed design of the installation method selected to carry out the Work as specified in the Contract Documents shall be completed.

Culvert at Station 14+588 (Blake Township)

Based on the ground conditions at the culvert site crossing Highway 61, Station 14+588, Township of Blake for installing culvert at a new alignment, Jack and bore, Horizontal Directional Drilling (HDD), pilot tube tunnelling, manual tunnelling, tunnel digging machine are not considered suitable options for culvert installation.

Culvert at Station 16+215 (Blake Township)

Based on the ground conditions at the culvert site crossing Highway 61, Station 16+215, Township of Blake for installing culvert at a new alignment: Jack and bore, Horizontal Directional Drilling (HDD), pilot tube tunnelling, manual tunnelling, tunnel digging machine are not considered suitable options for culvert installation.

Culvert 19+250 (Blake Township)

Based on the ground conditions at the culvert site crossing Highway 61, Station 19+250, Township of Blake for installing culvert at a new alignment: Jack and bore, Horizontal Directional Drilling (HDD), pilot tube tunnelling, manual tunnelling, tunnel digging machine are not considered suitable options for culvert installation.

Culvert at Station 20+040 (Blake Township)

Based on the ground conditions at the culvert site crossing Highway 61, Station 20+040, Township of Blake for installing culvert at a new alignment, Jack and bore, Horizontal Directional Drilling (HDD), pilot tube tunnelling, manual tunnelling, tunnel digging machine are not considered suitable options for culvert installation.

Culvert at Station 20+200 (Blake Township)

Based on the ground conditions at the culvert site crossing Highway 61, Station 20+200, Township of Blake for installing culvert at a new alignment: Jack and bore, Horizontal Directional Drilling (HDD), pilot tube tunnelling, manual tunnelling, tunnel digging machine are not considered suitable options for culvert installation.

Culvert at Station 20+375 (Blake Township)

Based on the ground conditions at the culvert site crossing Highway 61, Station 20+375, Township of Blake for installing culvert at a new alignment: Jack and bore, micro-tunnelling, pilot tube tunnelling, and manual tunnelling are not considered suitable options for culvert installation.

Culvert at Station 26+422 (Crooks Township)

Based on the ground conditions at the culvert site crossing Highway 61, Station 26+422, Township of Crooks for installing culvert at a new alignment: Jack and bore, micro-tunnelling, pilot tube tunnelling, and manual tunnelling are not considered suitable options for culvert installation.

4.02 Submission Requirements**4.02.01 Qualifications**

At least two weeks prior to construction, the names of the Project Superintendent, and Trenchless Contractor shall be submitted to the Contract Administrator.

4.02.01.01 Project Superintendent

The Project Superintendent shall have a minimum of five (5) years experience on projects with similar geology, scope and complexity, using the similar type of equipment required for this project.

During construction, the Project Superintendent shall not be changed without written permission from the Contract Administrator. A proposal to change the Project Superintendent shall be submitted at least one week prior to the actual change in Project Superintendent.

4.02.01.02 Trenchless Contractor

The Trenchless Contractor shall have a minimum of five (5) years experience on projects with similar geology, scope and complexity, using the similar type of equipment and materials of the type that meet the minimum requirements for this project.

4.02.02 Working Drawings

Three (3) sets of Working Drawings for the selected trenchless installation method, and a Request to Proceed shall be submitted to the Contract Administrator two weeks (2) prior to the commencement of the Work or as per the Contract Documents.

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

All Working Drawings shall bear the seal and signature of the Design Engineer and Design Checking Engineer.

Information and details shown on the Working Drawings shall include, but not limited to the following:

a) Plans and Details:

- i. Plans and profiles defining all horizontal and vertical alignment positions and positions of all utilities and other infrastructure within the zone of influence of the work.
- ii. A work plan outlining the materials, procedures, methods and schedule to be used to execute the Work.
- iii. A list of personnel, including backup personnel, and their qualifications and experience.
- iv. A traffic control plan.
- v. A safety plan including the company safety manual and emergency procedures.
- vi. The Working Area layout.
- vii. An erosion and sediment control plan that includes a contingency plan in the event the erosion and sediment control measures fail.
- viii. A contingency plan with specific details of the manner in which rock or boulders will be broken and removed from the face and the face will be protected to prevent soil loss into the liner.
- ix. A drilling fluid management plan, if applicable, that addresses control of frac-out pressures, any potential environmental impacts and includes a contingency plan, detailing emergency procedures in the event that the fluid management plan fails.
- x. Lighting, ventilation and fire safety details as may be required by applicable occupational health and safety regulations.
- xi. Excavated materials disposal plan.
- xii. Locations of protection systems.
- xiii. Contingency plans for the following potential conditions:
 - Unforeseen obstructions causing stoppage.
 - Deviation from required alignment and grade.
 - Extended service disruption.
 - Damage to the existing Utilities and methods of repair.
 - Soil heaving or settlement.
 - Contaminated soil or water.
 - Alignment passing through buried structures.

b) Designs:

- i. Primary Liner/Secondary Liner design (e.g. steel liner plates, steel ribs and wood lagging, and steel casing etc.).
- ii. Design assumption and material data when materials other than those specified are proposed for use.
- iii. Drill path design, details of alignment and alignment control, maximum curvature and reaming stages.
- iv. Minimum depth of cover for trenchless installation appropriate for the highway type and pipe diameter, maximum excavation diameter, maximum annulus, alignment and grade tolerance etc.
- v. Detailed subsurface conditions along the proposed path or within the footprint of the trenchless

technology equipment or pits/shafts.

c) Materials:

- i. Certification from the manufacturer that the product furnished on the contract meets the specifications cited in the manufacturer's product specification and that the materials supplied are suitable for the application.
- ii. Manufacturer data sheets for all drilling fluids and additives for use in Earth Pressure Balance (EPB), Slurry Pressure Balance (SPB).
- iii. Manufacturer data sheets for drilling systems.
- iv. Mix designs, target rheology criteria (e.g., viscosity, density, shear strength, gel time, pressure-filtration – fluid losses under pressure, etc.) and additive dosage rates for all slurries and Earth Pressure Balance (EPB) tunnel boring machine (TBM) and microtunnelling boring machine (MTBM) operations.
- v. The proposed grout mix design for grouts to be used for lubricating jacking pipe and for filling of voids and annular spaces.
- vi. Compressive strength of concrete pipe products.
- vii. Pipe class for all steel pipe products.
- viii. Steel for Permanent Casings:
 - One copy of a mill test certificate certifying that the steel meets the requirements for the appropriate standards for permanent casings shall be submitted to the Contract Administrator at the time of delivery.
 - Where mill test certificates originate from a mill outside Canada or the United States of America, the information on the mill certificates shall be verified by testing by a Canadian laboratory. The laboratory shall be certified by an organization accredited by the Standards Council of Canada to comply with the requirements of ISO/IEC 17025 for the specific tests or type of tests required by the material standard specified on the mill test certificate.
 - The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the specified material requirements. The stamp shall include the appropriate material specification number, the date (i.e., yyyy-mm-dd), and the signature of an authorized officer of the Canadian testing laboratory.
- ix. Slurry, drilling fluids, and tunnelling fluids:
 - Type, source, and physical and chemical properties of bentonite, polymer or other additives;
 - Source of water;
 - Method of mixing;
 - Water to solids ratio and the mass and volumes of the constituent parts, including any chemical admixtures or physical treatment employed to achieve required physical properties;
 - Details of procedure to be used for monitoring physical properties of slurry, drilling fluids and tunneling fluids or EPB spoils; and
 - Method of disposal of the slurry, drilling fluids and associated spoil.

d) Upstream/Downstream Portal Installation Procedure:

- i. Access shaft or entry/exit pit details, as applicable.

- ii. Face support and other temporary support details, if applicable.
- e) Primary Liner/Secondary Liner Installation and Grouting Procedure:
- i. Excavation and pipe installation procedures, including methods to handle obstructions and prevent soil cave-in.
 - ii. Details of tunnelling equipment/methods to be used for the works.
- f) Excavation and Dewatering:
- i. Equipment and methods for control, handling, treatment, and disposal of groundwater and water or fluids introduced by the Contractor;
 - ii. Equipment and methods for maintaining control of ground inflow at the excavation face during excavation;
 - iii. Equipment and methods for removal of cobbles and boulders;
 - iv. Manufacturer data sheets for each TBM, shield, tunnelling system or drilling system noting all intermediate and final cut dimensions, and methods and equipment for controlling and measuring drilling fluid, Slurry Pressure Balance (SPB) and Earth Pressure Balance (EPB) pressures;
 - v. Methods for measuring excavated volumes or weights of earth and rock materials cut from ground on a per meter or per pipe basis up to a maximum of 3 m long intervals per measurement;
 - vi. Target operating pressures (minimum and maximum) and range of expected pressure variation for slurry or EPB spoil at excavated face or drilling fluids at lead end of drilling equipment and in annular gap between maximum excavated dimensions and outside dimensions of tunnelling equipment, drilling equipment and primary liner systems;
 - vii. Basis for setting target operating conditions (pressures, flow rates, advance rates) and the relationship of target operating conditions to ground conditions;
 - viii. Basis for selection of excavation tools (e.g., bits, TBM face tools, MTBM face tools, excavator fittings, etc.) as related to expected ground conditions;
 - ix. Jacking forces for installation of pipe, for driving of trenchless equipment forward and, in the case of Auger Jack & Bore, for advancing the lead end of the casing ahead of the lead end of the auger cutting tools.
- g) Monitoring Method:

Methods, equipment, frequency and repeatability (accuracy and precision) of data collection to be employed for measuring and monitoring shall be submitted for:

- i. Maintaining the alignment of the installation;
- ii. EPB, SPB and drilling fluid pressures at the leading edge of excavation (face), flow rates and volume or weights of spoil;
- iii. Jacking forces on pipes, linings and cutting tools;
- iv. Torque, total revolutions and revolution rates on rotating equipment such as TBM or MTBM heads, auger flights, drill bits, etc.
- v. Grout injection pressures and volumes;
- vi. Longitudinal position of all casings and excavation cutting tools (auger flight heads, TBM face, drill bit position, etc.); and

- vii. Ground displacements (heave and settlement); and noise and ground vibrations induced by trenchless construction.

4.02.03 As-Built Drawings

As-built drawings shall be submitted to the Contract Administrator in a reproducible format prior to the Contract completion.

The as-built drawings shall be dated and bear the seal and signature of the Design Engineer and Design Checking Engineer.

5.0 MATERIALS

5.01 Pipe

5.01.01 General

The product shall be concrete pipe, steel pipe or high density polyethylene pipe as specified.

All joints shall be suitable for jacking operations as specified in the Working Drawings.

Fittings shall be suitable and compatible with the class and type of pipe with which they will be used.

All fittings shall be designed to be watertight.

5.01.02 Steel Pipe

Steel pipe shall be according to ASTM A252.

All steel casing pipe shall be square cut.

Steel casing pipe shall meet a straightness tolerance of 1.5 mm/m. When placed anywhere on the pipe parallel to the pipe axis, there shall not be a gap more than 1.5 mm between a 1 m long straightedge and the pipe.

5.01.03 High Density Polyethylene Pipe

High density polyethylene (HDPE) pipe according to OPSS 1840 shall be used in accordance with ASTM D3350.

Fittings shall be according to CAN/CSA-B182.6 or ASTM F894 and suitable for the class and type of pipe with which they will be used.

Jointing of HDPE piping shall be completed according to the manufacturer's recommended procedures and ASTM D2657. Where conflicts exist between the manufacturer's instructions and ASTM D2657, the manufacturer's instructions are to be followed.

Jointing of HDPE piping to other piping materials or appurtenances shall be completed using flanged connections.

5.01.04 Concrete Pipe

Concrete pipe shall be according to OPSS 1820.

5.02 Concrete

Concrete shall be according to OPSS 1350. The concrete strength shall be as specified on the Working Drawings.

5.03 Steel Reinforcement

Steel reinforcement for concrete work shall be according to OPSS 1440.

5.04 Wood

Wood shall be according to OPSS 1601.

5.05 Drilling Fluids

Drilling fluid shall be mixed according to the Working Drawings.

Selection of drilling fluid type shall be based on the soils encountered in the subsurface investigation.

The drilling fluids shall be mixed according to the manufacturer's recommendations.

Slurry shall be mixed according to the submitted slurry design and be appropriate for the anticipated subsurface conditions. The viscosity of slurry used for SPB tunnelling shall be no less than 40 seconds Marsh Funnel viscosity, as defined by ASTM D6910, measured prior to introduction of groundwater and spoil and as required to ensure:

- a) development of appropriate filter cake at excavation face to provide slurry support pressures exceeding ground and groundwater pressures at excavation face;
- b) lubricate installation of primary liners as required;
- c) transport spoil through pipe systems.

5.06 Grout

Purging grout shall conform to the requirements of OPSS 1004 and be wetted with only sufficient water to make the mixture plastic.

6.0 EQUIPMENT

6.01 Auger Jack & Bore

Except in the case of dewatering to at least 1 m below the tunnel/bore invert for the full length of the pipe alignment, Auger Jack & Bore shall not be used and will not be permitted where subsurface conditions indicate that saturated gravel, sand and silt soils may be encountered at pipe level or within one pipe diameter above or below outside pipe dimensions.

Pipe Auger Jack & Bore equipment shall be determined by the Contractor and shall be identified in the

submission requirements specified herein.

Specific details of the equipment with which rock or boulders will be broken and removed from the face and the face will be protected to prevent soil loss into the liner shall be submitted to the Contract Administrator for information purposes prior to proceeding with the Works.

The lead end of the auger shall be maintained at least one pipe diameter inside the lead end of the casing. The auger cutting tools shall not extend to or beyond the lead end of the casing at any time unless specific exception is provided by the Ministry prior to construction. Submittals shall identify anticipated jacking forces for advancing casing ahead of leading edge of auger cutting tools in addition to friction forces that are to be overcome by jacking systems.

6.02 Pipe Ramming

Pipe Ramming equipment shall be determined by the Contractor and shall be identified in the submission requirements specified herein.

The Pipe Ramming hammer(s) shall be capable of driving the pipe casing from the entry pit to the exit pit through the existing subsurface conditions at the site without removal of soil from within the casing until the lead end of the pipe is outside the zone of influence for any overlying infrastructure.

Specific details of the equipment with which rock or boulders will be broken and removed from the face and the face will be protected to prevent soil loss into the pipe shall be submitted to the Contract Administrator for information purposes prior to proceeding with the Works.

The Contractor proposed procedure should ensure that no flow liquefaction (significant strength loss in soils) will occur to the embankment side slopes during pipe ramming operations. A pipe ramming driveability analysis and report needs to be provided by the Contractor for review and approval in advance of the construction. The ground vibration caused by pipe ramming should be measured using a seismograph and the Peak Particle Velocity (PPV) must be kept below 15 mm/s during pipe ramming operation.

6.03 Horizontal Directional Drilling

6.03.01 General

The Horizontal Directional Drilling (HDD) equipment shall consist of a directional drilling rig and a drilling fluid mixing and delivery system to successfully complete the product installation without exceeding the maximum tensile strength of the product being installed.

6.03.02 Drilling Rig

The horizontal directional drilling rig shall:

- a) Consist of a leak free hydraulically powered boring system to rotate, push, and pull hollow drill pipe into the ground at a variable angle while delivering a pressurized fluid mixture to a guidable drill head.
- b) Have drill rod that is suitable for both the drill and the product pipe installation.
- c) Contain a drill head that is steerable, equipped with the necessary cutting surfaces and fluid jets, and be suitable for the anticipated ground conditions.

- d) Have adequate reamers and down-bore tooling equipped with the necessary cutting surfaces and fluid jets to facilitate the product installation and be suitable for the anticipated ground conditions.
- e) Contain a guidance system to accurately guide boring operations.
- f) Be anchored to the ground to withstand the rotating, pushing, and pulling forces required to complete the product installation.
- g) Be grounded during all operations unless otherwise specified by the drilling rig manufacturer.

6.03.03 Drill Head

The drill head shall be steerable by changing its rotation, be equipped with the necessary cutting surfaces and drilling fluid jets, and be of the type for the anticipated subsurface conditions,

6.03.04 Guidance System

The guidance system shall be setup, installed, and operated by trained and experienced personnel. The operator shall be aware of any magnetic or electromagnetic anomalies and shall consider such influences in the operation of the guidance system when a magnetic or electromagnetic system is used.

6.03.05 Drilling Fluid Mixing System

The drilling fluid mixing system shall be of sufficient size to thoroughly and uniformly mix the required drilling fluid.

6.03.06 Drilling Fluid Delivery System

The delivery system shall have a means of measuring and controlling fluid pressures and be of sufficient flow capacity to ensure that all slurry volumes are adequate for the length and diameter of the final bore and the anticipated subsurface conditions. Connections between the delivery pump and drill pipe shall be leak-free.

6.04 Tunnelling

Tunnelling equipment shall be determined by the Contractor and shall be identified in the submission requirements specified herein. Specific details of the Tunnelling equipment included in the submission shall be provided for:

- a) rock or boulder breaking and removal;
- b) equipment used within shields for spilling, fore-poling, face drainage, breasting boards/plates and for otherwise maintaining support of the tunnel crown and face under all anticipated conditions;
- c) jacking systems;
- d) alignment control systems;

Use of rock fracturing chemicals shall only be considered subject to a field demonstration satisfactory to the Ministry prior to its use. Use of explosives is prohibited without specific application and acceptance by the Ministry prior to construction.

6.05 Microtunnelling Equipment

The Contractor shall be responsible for selecting Microtunnelling equipment which, based on past experience, has proven to be satisfactory for excavation of the soils that will be encountered.

The Contractor shall employ Microtunnelling equipment that will be capable of handling the various anticipated ground conditions.

The MTBM shall also be capable of controlling loss of soil ahead of and around the machine and shall provide continuous pressurized support of the excavated face.

- a) Remote Control System – The Contractor shall provide a MTBM that includes a remote control system with the following features:
 - i. Allows for operation of the system without the need for personnel to enter the microtunnel.
 - ii. Has a display available to the operator, at a remote operation console, showing the position of the shield in relation to a design reference together with other information such as face pressure, roll, pitch, steering attitude, valve positions, thrust force cutter head torque, rate of advance and installed length.
 - iii. Integrates the system of excavation and removal of spoil and its simultaneous replacement by product pipe. As each pipe section is jacked forward, the control system shall synchronize all of the operational functions of the system.
 - iv. The system shall be capable of adjusting the face pressure to maintain face stability for the particular soil condition encountered.
 - v. The system shall monitor and continuously balance the soil and ground water pressure to prevent loss of soil or uncontrolled ground water inflow.
 - vi. The pressure at the excavation face shall be managed by controlling the volume of spoil removal with respect to the advance rate.
 - vii. The system shall include a separation process designed to provide adequate separation of the spoil from the slurry so that slurry with a sediment content within the limits required for successful microtunnelling, can be returned to the cutting face for reuse. Appropriately contain spoil at the site prior to disposal.
 - viii. The type of separation process shall be suited to the size of microtunnel being constructed, the soil type being excavated, and the work space available at each work area.
 - ix. The system shall allow the composition of the slurry to be monitored to maintain the slurry weight and viscosity limits required.
- b) Active Direction Control – The Contractor shall provide a MTBM that includes an active direction control system with the following features:
 - i. Controls line and grade by a guidance system that relates the actual position of the MTBM to a design reference.
 - ii. Provides active steering information that shall be monitored and transmitted to the operating console and recorded.
 - iii. Provides positioning and operation information to the operator on the control console.

6.05.01 Pipe Jacking Equipment

Provide a pipe jacking system with the following features:

- a) Has the main jacks mounted in a jacking frame located in the launch shaft.

- b) Has a jacking frame that successively pushes towards a receiving shaft, a string of product pipe that follows the microtunnelling excavation equipment.
- c) Has sufficient jacking capacity to push the microtunnelling excavation equipment and the string of pipe through the ground.
- d) The main jack station may be complemented with the use of intermediate jacking stations as required.
- e) Has a capacity at least 20 % greater than the calculated maximum jacking load.
- f) Develops a uniform distribution of jacking forces on the end of the casing pipe.
- g) Provides and maintains a pipe lubrication system at all times to lower the friction developed on the surface of the pipe during jacking.
- h) Jack Thrust Blocking shall adequately support the jacking pressure developed by the main jacking system.
- i) Special care shall be taken when setting the pipe guide rails in the jacking shaft to ensure correctness of the alignment, grade, and stability.

6.05.02 Spoil Separation System

The Contractor shall determine the type of spoil separation equipment needed for each drive based on the geotechnical information available and other project constraints.

6.05.03 Electrical Equipment, Fixtures and Systems

Electrical equipment shall be suitably insulated for noise reduction. Noise produced by electrical equipment must comply with local municipal noise by-laws.

Electrical systems shall conform to requirements of the Canadian Electrical Code – CSA C22.1.

7.0 CONSTRUCTION

7.01 General

The Contractor shall notify the Contract Administrator at least 48 hours in advance of starting the work. The proposed method of pipe installation to be used by the Contractor shall be subject to the limitations presented in the following subsections.

The Contractor's Engineer shall supervise the work at all times.

A Request to Proceed shall be submitted to the Contract Administrator upon completion of each of the following operations and prior to commencement of each subsequent operation and no less than 2 weeks prior to the commencement of the trenchless installation.

- a) Site Surveying (see Clause 4.02)
- b) Excavation for pits including dewatering of excavations
- c) Jacking / Ramming / Directional Drilling of Casing / Liner
- d) Installation of the Product
- e) Grouting Operations

Operations a) to e) shall not proceed until the Contract Administrator has issued a Notice to Proceed for each proceeding operation.

7.01.01 Layout, Alignment and Depth Control

The location of the installation shall be established from the lines, elevations and tolerances specified in the Contract Documents. The pipe installation shall be to the horizontal and vertical alignments specified in the Contract Drawings. Deviations from location, alignment, grades and/or invert levels shall be corrected by the Contractor at no cost to the Ministry.

All reference points necessary to construct the pipe installation and appurtenances shall be laid out.

The Contractor shall calibrate tracking and locating equipment at the beginning of each Working Day, and shall monitor and record the alignment and depth readings provided by the tracking system every 2 m.

The Contract Administrator shall be provided with the assistance and access necessary to check the layout of the pipe installation and associated appurtenances.

The Contractor shall submit records of the alignment and depth of the installation to the Contract Administrator at the completion of the installation.

7.01.02 Construction Shafts

Construction shafts shall be specified in the Contractor's submission. The boundaries and protection of these shall be as required to contain all disturbances to areas outside of the ESA limits.

Shafts shall be maintained in a drained condition.

A minimum 2.4 m high secure fence shall be installed around the perimeter of the construction shaft area with gates and truck entrances. The fence shall be removed on completion of the work.

7.01.03 Protection Systems

The construction of all protection systems shall be according to OPSS 539.

Where the stability, safety, or function of an existing roadway, railway, watercourse, other works, ESA's, or proposed works may be impaired due to the method of operation, protection shall be provided. Protection may include sheathing, shoring, and piles where necessary to prevent damage to such works or proposed works.

7.01.04 Settlement or Heave

Any disturbance to the ground surface (settlement or heave) as a result of the pipe installation shall be immediately corrected by the Contractor, at no additional cost to the Ministry.

7.01.05 Stability of Excavation

The construction methods, plant, procedures, and precautions employed shall ensure that excavations are stable, free from disturbance, and maintained in a drained condition.

The construction methods, plant, procedures, and materials employed shall prevent the migration of soil and/or rock material into the excavation from adjacent ground.

7.01.06 Preservation and Protection of Existing Facilities

Preservation and protection of existing facilities shall be according to OPSS 491.

Minimum horizontal and vertical clearances to existing facilities as specified in the Contract Documents shall be maintained. Clearances shall be measured from the nearest edge of the largest cut diameter required to the nearest edge of the facility being paralleled or crossed.

Existing underground facilities shall be exposed to verify its horizontal and vertical locations when the outlet pipe path comes within 1.0 m horizontally or vertically of the existing facility. Existing facilities shall be exposed by non-destructive methods. The number of exposures required to monitor work progress shall be as specified in the Contract Documents.

7.01.07 Transporting, Unloading, Storing and Handling Materials

Manufacturer's recommendations for transporting, unloading, storing, and handling of materials shall be followed.

7.01.08 Trenching, Backfilling and Compacting

Trenching, backfilling, and compacting for entry and exit points or other locations along the pipe path shall be according to OPSS 401.

7.01.09 Support Systems

Support systems shall be according to OPSS 404.

If any open excavation will encroach into the highway embankment, the protection system shall satisfy the requirements for Performance Level 2 as specified in OPSS 539.

7.01.10 Dewatering

The work of this section includes control, handling, treatment, and disposal of groundwater. The Contractor shall review the foundation investigation report for reference to soil and groundwater conditions on the project site and plan a dewatering scheme accordingly.

The Contractor shall control groundwater inflows to excavations to maintain stability of surrounding ground, to prevent erosion of soil, to prevent softening of ground exposed in the excavation, and to avoid interfering with execution of the work.

The Contractor shall maintain excavations free of standing water at all times during excavation, including while concrete is curing.

Should water enter the excavation in amounts that could adversely affect the performance of the work or could cause loss of ground, the Contractor shall take immediate steps to control the inflow.

The Contractor is alerted that seepage zones of perched water within the fill materials should be expected, particularly where granular materials are excavated.

Dewatering shall be according to OPSS 517.

7.01.11 Removal of Cobbles and Boulders

The Contractor is alerted that cobbles and boulders are expected within the soil deposits at the site. Accordingly, the Contractor shall address the removal of cobbles and boulders in the proposed method of

construction. Removal of cobbles and boulders shall be expected to be routine and will not be considered obstruction. The Contractor shall immediately inform the Contract Administrator of any obstruction encountered.

7.01.12 Removal of Obstructions

This section indicates potential obstructions that may be encountered during the tunnelling operation at each location where trenchless method is selected for pipe installation. The Contractor is responsible for selecting the appropriate tunneling method and equipment based on the subsurface information and is also responsible for removal of any obstruction that may be encountered within the trenchless alignment at each location.

Culvert at Station 14+588 (Blake Township)

The Contractor is alerted that obstructions such as, but not limited to wood debris, roots, and construction debris consisting of (broken asphalt, concrete etc.) are expected within the trenchless alignment as identified in the Contract Documents. Accordingly, the Contractor shall address methods for the removal of obstructions in the proposed method of construction. The Contractor shall immediately inform the Contract Administrator of any obstruction encountered and the Contractor's expected method of and schedule for removal.

The contractor is alerted that there are organic inclusions are being buried near the bottom of the embankment fill and should be considered in the Contractor's selection of tunnelling technique and equipment. Refer to the Foundation Investigation Report for a description of subsurface conditions. Given the significant thickness of fill materials, the presence of other obstructions cannot be ruled out.

Culvert at Station 16+215 (Blake Township)

The Contractor is alerted that obstructions such as, but not limited to wood debris, roots, and organic inclusions are expected within the trenchless alignment as identified in the Contract Documents. Accordingly, the Contractor shall address methods for the removal of obstructions in the proposed method of construction. The Contractor shall immediately inform the Contract Administrator of any obstruction encountered and the Contractor's expected method of and schedule for removal.

The contractor is alerted that wood/organic inclusions have been encountered buried within the subsurface soil. Refer to the Foundation Investigation Report for a description of subsurface conditions. Given the significant thickness of fill materials, the presence of other obstructions cannot be ruled out.

Culvert 19+250 (Blake Township)

The Contractor is alerted that obstructions such as, but not limited to wood debris, roots, and organic inclusions are expected within the trenchless alignment as identified in the Contract Documents. Accordingly, the Contractor shall address methods for the removal of obstructions in the proposed method of construction. The Contractor shall immediately inform the Contract Administrator of any obstruction encountered and the Contractor's expected method of and schedule for removal.

The contractor is alerted that trace organics were observed in the native silt deposit in BH Nos. 2, 3 and 4. Occasional wood fragments were observed in the silt fill within the embankment and in the native silt in BH No. 1. Refer to the Foundation Investigation Report for a description of subsurface conditions. Given the significant thickness of fill materials, the presence of other obstructions cannot be ruled out.

Culvert at Station 20+040 (Blake Township)

The Contractor is alerted that obstructions such as, but not limited to wood debris, roots, and construction debris consisting of (broken asphalt, concrete etc.) are expected within the trenchless alignment as identified in the Contract Documents. Accordingly, the Contractor shall address methods for the removal of obstructions in the proposed method of construction. The Contractor shall immediately inform the Contract Administrator of any obstruction encountered and the Contractor's expected method of and schedule for removal.

The contractor is alerted that there are asphalt debris and wood/organic inclusions are being buried near the bottom of the embankment fill and should be considered in the Contractor's selection of tunnelling technique and equipment. Refer to the Foundation Investigation Report for a description of subsurface conditions. Given the significant thickness of fill materials, the presence of other obstructions cannot be ruled out.

Culvert at Station 20+200 (Blake Township)

The Contractor is alerted that obstructions such as, but not limited to wood debris, roots, and organic inclusions are expected within the trenchless alignment as identified in the Contract Documents. Accordingly, the Contractor shall address methods for the removal of obstructions in the proposed method of construction. The Contractor shall immediately inform the Contract Administrator of any obstruction encountered and the Contractor's expected method of and schedule for removal.

The contractor is alerted that there are asphalt debris and wood fragments inclusions are being buried near the bottom of the embankment fill and should be considered in the Contractor's selection of tunnelling technique and equipment. Refer to the Foundation Investigation Report for a description of subsurface conditions. Given the significant thickness of fill materials, the presence of other obstructions cannot be ruled out.

Culvert at Station 20+375 (Blake Township)

The Contractor is alerted that obstructions such as, but not limited to wood debris, roots, and construction debris consisting of (broken asphalt, concrete etc.) are expected within the trenchless alignment as identified in the Contract Documents. Accordingly, the Contractor shall address methods for the removal of obstructions in the proposed method of construction. The Contractor shall immediately inform the Contract Administrator of any obstruction encountered and the Contractor's expected method of and schedule for removal.

The contractor is alerted that there are rock fragments (cobble to boulder) and concrete debris material are being buried near the bottom of the embankment fill and should be considered in the Contractor's selection of tunnelling technique and equipment. Refer to the Foundation Investigation Report for a description of subsurface conditions. Given the significant thickness of fill materials, the presence of other obstructions cannot be ruled out.

Culvert at Station 26+422 (Crooks Township)

The Contractor is alerted that obstructions such as, but not limited to wood debris, roots, and construction debris consisting of (broken asphalt, concrete etc.) are expected within the trenchless alignment as identified in the Contract Documents. Accordingly, the Contractor shall address methods for the removal of obstructions in the proposed method of construction. The Contractor shall immediately inform the Contract Administrator of any obstruction encountered and the Contractor's expected method of and schedule for removal.

The contractor is alerted that there are organic inclusions (tree log remains) are being buried near the bottom of the embankment fill and should be considered in the Contractor's selection of tunnelling technique and equipment. Refer to the Foundation Investigation Report for a description of subsurface conditions. Given the

significant thickness of fill materials, the presence of other obstructions cannot be ruled out.

7.01.13 Management of Excess Material

Management of excess material shall be according to OPSS 180.

Satisfactory re-usable excavated material required for backfill shall be separated from unsuitable excavated material.

7.01.14 Site Restoration

Site restoration shall be according to OPSS 492.

7.02 Auger Jack & Bore Installation

7.02.01 Method of Installation Procedure

The installation procedure to be used shall be subject to the following limitations:

- a) Hydraulically operated jacks of adequate number and capacity shall be provided to ensure smooth and uniform advancement without over-stressing of the pipe.
- b) A suitably padded jacking head or collar shall be provided to transfer and distribute jacking pressure uniformly over the entire end bearing area of the pipe.
- c) The jacking pipe shall be fully supported in the jacking pit at the specified line and grade.
- d) Selection of the excavation method and jacking equipment shall take into consideration the conditions at each pipe crossing.

7.02.02 Pipe Installation

Concrete pipe joints shall be watertight and according to OPSS 1820, and must withstand jacking forces, determined by the Contractor.

During the jacking of the liner, the space between the liner and the wall of the excavated volume (e.g., maximum cut diameter) shall be kept filled with bentonite slurry. Upon completion of jacking, the space between the liner and the wall of the excavated volume shall be filled with grout or slurry with gel strength properties demonstrated to be sufficient to form a semi-solid or solid gap filling material, prevent ground convergence around the pipe and subsequent ground surface subsidence and prevent long-term water flow at the outside boundary of any pipe and ground.

The annular space between the liner and the product shall be fully grouted with a watertight, expandable, and stable grout.

7.03 Pipe Ramming Installation

For Pipe Ramming installation the following requirements apply:

- Only smooth walled steel pipe shall be used. Butt welding of pipe joints shall conform to CSA W59.
- Ramming equipment of adequate capacity shall be provided to ensure smooth and uniform

advancement between the shafts/pits without overstressing of the pipe. Delays shall be avoided between ramming operations.

- A Ramming head shall be provided to transfer and distribute jacking pressure uniformly over the entire end bearing area of the pipe.
- Two or more lubricated guide rails or sills shall be provided of sufficient length to fully support the pipe at the specified line and grade in the ramming pit. Pipe shall be installed to the line and grade specified.
- The ground vibration caused by pipe ramming should be measured using a seismograph and the Peak Particle Velocity (PPV) must be kept below 15 mm/s during pipe ramming operation.
- Removal of materials from within the pipe shall not be undertaken until the lead end of the pipe has passed fully through and beyond the zone of influence of any overlying infrastructure.
- Following installation of the liner pipe, all material shall be removed from the pipe to the satisfaction of the Contract Administrator.
- Any voids remaining between the pipe and the excavation wall shall be grouted as soon as the pipe is rammed.
- The annular space between the liner pipe and the product shall be fully grouted with a watertight, expandable, and stable grout.

7.04 Horizontal Directional Drilling Installation

7.04.01 General

When strike alerts are provided on a drilling rig, they shall be activated during drilling and maintained at all times.

For Horizontal Directional Drilling (HDD), the Contractor shall ensure that during pilot hole drilling the maximum degree of deviation or “dog-leg” shall be 2.5 degrees per 9 m drill pipe length. Any deviation exceeding 2.5 degrees will necessitate a pull-back and straightening of the alignment at the Contractor’s sole expense. The pilot hole exit location shall be within 0.5m of the target location.

7.04.02 Site Preparation

Site preparation shall be according to OPSS 490 and as specified herein.

The work site shall be graded or filled to provide a level working area for the drilling rig. No alterations beyond what is required for HDD operations are to be made. All activities shall be confined to designated Working Areas.

7.04.03 Pilot Bore

The pilot bore shall be drilled along the bore path in accordance with the grade, alignment, and tolerances as indicated on the Contractor’s submitted drilling plan to ensure that the product is installed to the line and grade shown on the Contract Drawings. The Contractor’s methods shall take into consideration the conditions at each crossing within the pipe alignment and shall be suitable to advance through such obstructions such as cobbles and boulders and address the potential for deflection off these obstruction and/or soil conditions.

In the event the pilot bore deviates from the submitted path, the Contract Administrator shall be notified. The Contract Administrator may require the Contractor to pullback, fill and abandon the hole and re-drill from the

location along the bore path before the deviation.

If a drill hole beneath highways, roads, watercourses or other infrastructure must be abandoned, the hole shall be backfilled with grout or bentonite to prevent future subsidence and subsurface water conveyance.

The Contractor shall maintain drilling fluid pressure and circulation throughout the HDD process, including during the initial pilot bore and during the reaming process.

The Contractor shall, at all times and for the entire length of the installation alignment, be able to demonstrate the horizontal and vertical position of the alignment, the fluid volume used, return rates, and pressures.

7.04.04 Drilling Fluid Losses to Surface (“Frac-Out”)

To reduce the potential for hydraulic fracturing of the hole during horizontal directional drilling, a minimum depth of cover of 5 m shall be maintained between the top of pipe and the surface of any pavements or beds of water courses. Sections of the pipe close to the entry and exit pit with less than 5 m cover shall be cased. The Contractor shall ensure that drilling fluid pressures are properly set and controlled for the full length of the bore to prevent frac-out for the depth of cover available between the bottom of the pavement structure (bottom of the subbase material) and the top of the bore.

Once a fluid loss or frac-out event is detected, the Contractor shall halt operations immediately and conduct a detailed examination of the drill path and implement measures to collect all fluids discharged to surface, mitigate and prevent additional fluid loss.

7.04.05 Reaming

The bore shall be reamed using the appropriate tools to a diameter at least 50% greater than the outside diameter of the product.

7.04.06 Product Installation

7.04.06.01 General

The product shall be jointed according to manufacturer’s recommendations. The length of the product to be pulled shall be jointed as one length before commencement of the continuous pulling operation.

The product shall be protected from damage during the pullback operation.

The minimum allowable bending radius for the product shall not be contravened.

Product shall be allowed to recover to static conditions from thermal and installation stresses before connections to new or existing facility are made. Product recovery time shall be according to manufacturers recommendations.

7.04.06.02 Pullback and Grouting

After successfully Reaming the bore to the required diameter, the product pipe shall be pulled through the bore path. Once the pullback operation has commenced, it shall continue without interruption until the product pipe is completely pulled into bore unless otherwise approved by the Contract Administrator.

A swivel shall be used between the reamer and the product being installed to prevent rotational forces from

being transferred to the product. A weak link or breakaway connector shall be used to prevent excess pulling force from damaging the product.

The product pipe shall be inspected for damage where visible at excavation pits and where it exits the bore. Any damage noted shall be rectified to the satisfaction of the Contract Administrator.

The pull back and Reaming operations shall not exceed the fluid circulation rate capabilities. Reaming and back pulling operations shall be planned to ensure that, once started, all reaming and back pulling operations are completed without stopping and within the permitted work hours.

The space between the pipe and the walls of the excavated volume shall be filled with grout or slurry with gel strength properties demonstrated to be sufficient to form a semi-solid or solid gap filling material, prevent ground convergence around the pipe and subsequent ground surface subsidence and prevent long-term water flow at the outside boundary of any pipe and ground.

7.05 Tunnelling Installation

7.05.01 General

Excavation of native soil and fill shall be done in a manner to control groundwater inflow to the excavation and to prevent loss of ground into the excavation.

Methods of excavating the tunnel shall be capable of fully supporting the face and shall accommodate the removal of boulders and other oversize objects from the face. Continuous ground support shall be maintained during excavation.

As the excavation progresses, the Contractor shall continuously monitor (every 2 m) indications of support distress, such as cracking, deflection or failure of support system and subsidence of ground near the excavation.

The Contractor shall provide ventilation and lighting in accordance with OSHA requirements for the entire length of the tunnel installed as tunneling progresses.

The tunnel is to be kept sufficiently dry at all times to permit work to be performed in a safe and satisfactory manner.

The Contractor shall maintain clean working conditions at all times in tunnels.

If excavation threatens to endanger personnel, the Work, or adjacent property, the Contractor shall cease excavation and make the excavation face secure. The Contractor shall then evaluate methods of construction and revise as necessary to ensure the safe continuation of the Work.

The Contractor shall maintain tunnel excavation line and grade to provide for construction of final lining within specified tolerances.

7.05.02 Tunnelling Method

The Tunnelling method shall be suitable to provide face support in changing ground conditions that may be encountered during the progress of the work. The selection of the Tunnelling method should consider the soil conditions at each pipe crossing and the presence of obstructions, such as cobbles and boulders, with respect to the tunnel alignment.

7.05.03 Primary Liner (Support System)

Primary support systems shall prevent deterioration, loosening, or unravelling of ground surfaces exposed by excavation.

The primary liner support system shall be designed and installed to achieve the intended performance requirements.

Primary liner support system shall maintain the safety of personnel, minimize ground movement into the excavation, ensure stability and maintain strength of ground surrounding the excavation.

The primary liner shall be designed to support all subsurface conditions and hydrostatic pressures and to withstand any additional loads caused by installation and grouting and shall ensure that no ground loading or other loading will be placed on the new work until after design strength has been reached.

The primary liner shall be installed so that the exterior is as tight as possible to the excavated surface of the tunnel and allows the placement of the full design thickness of the secondary lining.

Primary support systems shall be compatible with the encountered ground conditions, with the method of excavation, with methods for control of water, and with placement of the permanent lining.

All voids between the primary lining and the wall of the excavated volume shall be filled with cement grout or slurry with gel strength properties demonstrated to be sufficient to form a semi-solid or solid gap filling material, prevent ground convergence around the pipe and subsequent ground surface subsidence and prevent long-term water flow at the outside boundary of any pipe and ground. If an unexpanded liner is used, the space outside the liner plates shall be filled at least daily.

7.05.04 Secondary Liner

7.05.04.01 Placing of Grout

The void outside the finished secondary liner shall be filled with cement grout according to the Contractor's submission.

Grout shall not be placed until the lining has achieved 85% of its specified strength or 30 MPa. Grouting shall be limited to such sequences and programs as are necessary to avoid damaging any part of the works or any other structure or property. Grout mix design shall be chemically and thermally compatible with all pipe systems.

7.06 Microtunnelling

7.06.01 General

Excavation of soil, rock and fill shall be done in a manner to control and prevent groundwater inflow to the tunnel.

The MTBM shall be capable of fully supporting the face and shall accommodate the removal of boulders and other obstructions from the face. Continuous ground support shall be maintained during excavation.

The tunnel is to be kept well drained at all times to permit work to be performed in a safe and satisfactory manner.

The Contractor shall maintain clean working conditions at all times.

In the event that excavation threatens to endanger personnel, the Work, adjacent property, roadways, railways, waterways, or the public in any way, the Contractor shall cease excavation. The Contractor shall then evaluate the methods of construction and revise as necessary to ensure the safe continuation of the Work.

The Contractor shall maintain the tunnel excavation line and grade to provide for construction of the product within the specified tolerances.

7.06.02 Method of Installation

The installation procedure to be used shall be subject to the following limitations:

- The jacking pipe shall be fully supported in the jacking pit at the specified line and grade.
- Selection of the excavation method and jacking equipment shall take into consideration the subsurface conditions within the tunnel alignment.
- Perform microtunnelling operations in a manner that will minimize the movement of the ground in front of and surrounding the tunnel in conformance with the limits listed in the Contract Documents.
- Prevent damage to structures and utilities above and in the vicinity of the microtunnelling operations.
- Excavated diameter should be the minimum size required to permit pipe installation by jacking.
- Whenever there is a condition encountered which could endanger the microtunnel excavation or adjacent structures if tunnelling operations cease, continue to operate without intermission including 24-hour Working Days, weekends and holidays, until the condition no longer exists.
- Maintain an envelope of lubricant around the exterior of the pipe during the jacking and excavation operation to reduce the exterior soil/pipe friction and possibility of the pipe seizing in place.
- In the event a section of pipe is damaged during the jacking operation or a joint failure occurs, as evidenced by inspection, visible ground water inflow or other observations, the Contractor shall submit for approval his methods for repair or replacement of the pipe.

7.06.03 Casing Installation

Casing must withstand the jacking forces determined by the Contractor.

The space between the casing and the wall of the excavation shall be kept filled with lubricant during the pipe jacking operation. Upon completion of pipe jacking, the space between the casing and the wall of the excavation shall be filled with grout that is compatible with the casing.

The casing shall act as a support system to maintain the safety of personnel, minimize ground movement into the excavation, ensure stability and maintain strength of ground surrounding the casing.

The casing shall be designed to support all subsurface conditions and hydrostatic pressures and to withstand any additional loads caused by installation and grouting.

7.07 Instrumentation and Monitoring

The Instrumentation and Monitoring program shall be project specific.

The work specified in this section includes furnishing and installing instruments for monitoring of settlement (and heave) and ground stability. The Contractor's Instrumentation personnel shall include a specialist geo-engineering consulting firm specialized in installation, monitoring and maintenance of tunnelling instruments with at least 5 years experience of similar projects.

7.07.01 General

The Contractor shall furnish, install and monitor Surface Monitoring Points (SMP) and In-Ground Monitoring Points at the locations shown on the Contract Drawings.

The equipment and procedures used for settlement monitoring during construction must be capable of surveying the settlement point elevations to within a repeatability (combined accuracy and precision of equipment and methods) ± 2 mm of the actual elevation.

7.07.02 Surface Settlement Monitoring Points

Surface settlement monitoring points shall be installed on the traffic lanes and shoulders to monitor settlement and stability. The surface settlement monitoring points shall be installed centred on the tunnel alignment as arrays of three points at intervals of 5 m or less and off-set a lateral distance of 1.5 m on either side of the tunnel centerline.

Surface settlement monitoring points shall be hardened steel markers treated or coated to resist corrosion, with an exposed convex head having a minimum diameter of 12 mm and similar to surveyor's PK nails. Markers shall be rigidly affixed so as not to move relative to the surface to which it is attached. Traffic shall be managed by the Contractor using short-term lane closures in accordance with the Ontario Traffic Manual (OTM). Surface markers shall be recessed or otherwise designed for safe passage of vehicles at highway speeds and protected from snow removal equipment in the event that work occurs during snow removal seasons.

7.07.03 In-Ground Settlement Monitoring Points

In-ground settlement monitoring points shall be installed beyond the traffic lanes and shoulders to monitor settlement and stability of the ground surface between the surface settlement monitoring points and the entry and exit portals. In-ground settlement monitoring points shall be located at intervals of 5 m or less along the tunnel alignment.

In-ground settlement monitoring points shall be 12-18 mm rebar encased in a 50-70 mm, SCH40 PVC pipe, set to a depth of 1.5 m below ground surface or below frost penetration depth, whichever is greater. The assembly shall be placed in a drill hole, backfilled with uniform sand and provided with protective covers suitable for high vehicular traffic areas.

7.07.04 Installation, Replacement and Abandonment

The Contractor shall install all settlement monitoring points a minimum of two (2) weeks prior to the start of works to permit baseline surveying to be completed. The settlement monitoring points shall be clearly labelled for easy field identification. The Contractor shall submit to the Contract Administrator a site plan showing the locations of the monitoring points, a geodetic survey of the settlement monitoring points including station, offset and elevation. Instruments damaged by the Contractor's operations or other causes shall be replaced and surveyed at the time of installation within 24 hours at no additional cost. At the completion of the job, the Contractor shall abandon all instrumentations installed during the course of the Work and restore the surface at

instrument locations.

7.07.05 Monitoring and Reporting Frequency

The Contractor shall survey and otherwise obtain elevations of all settlement monitoring points at the following time intervals:

- a) Three consecutive readings at least one week prior to commencement of the work (Baseline Reading);
- b) Once per shift or once daily during tunnelling operations period whichever results in the more frequent reading intervals; and
- c) Weekly after completion of the work for one month, or until such time at which all parties agree that further movement has stopped.

All readings shall be submitted to the Contract Administrator for information purposes on a weekly basis.

Each report shall include all survey data collected in tabular and graphical format as plots of time versus settlement in comparison to survey data collected prior to commencement of the work.

7.07.06 Benchmarks

Two independent benchmarks shall be used for all settlement monitoring surveying and shall be located sufficiently outside the zone of influence such that the benchmarks are not influenced by any trenchless or other construction activity or weather conditions (e.g., frost heave). All surveying shall be reported using the geodetic datum and coordinate system as defined in the Contract Documents.

7.07.07 Vibration Monitoring

Vibration monitoring points should be established along at least three cross sections (i.e. each cross section with seven monitoring points) transverse to the centerline axis of the advancing pipe. At each cross section, one monitoring point will be located directly above the pipe, with three monitoring points on right hand and left hand side of the centerline at offsets of D, 3D and 5D, where D equals the diameter of the pipe. The ground vibration should be measured using a seismograph and the Peak Particle Velocity (PPV) must be kept below 15 mm/s. Furthermore, installation of pneumatic vibrating wire piezometers along the culvert alignment (at the base of the embankment) should be considered to alert contractor about development of excess porewater pressure during the ramming operation.

7.08 Criteria for Assessment of Roadway Subsidence/Heave

Review and Alert Levels

Based on the monitoring of the ground movement as specified in Subsections 4.02 and 7.07, the following represents trigger levels that define magnitude of movement and corresponding action:

- a) **Review Level:** If a maximum value of 10 mm relative to the baseline readings is reached, the Contractor shall review or modify the method, rate or sequence of construction or ground stabilization measures to mitigate further ground displacement. If this Review Level is exceeded, the Contractor shall immediately notify the Contract Administrator and review and discuss response actions. The Contractor shall submit a plan of action to prevent Alert Levels from being reached. All construction work shall be continued such that the Alert Level is not reached.

- b) Alert Level: If a maximum value of 15 mm relative to the baseline readings is reached, the Contractor shall cease construction operations, inform the Contract Administrator and execute pre-planned measures to secure the site, to mitigate further movements and to assure safety of public and maintain traffic. No construction shall take place until all of the following conditions are satisfied:
- i. The cause of the settlement has been identified.
 - ii. The Contractor submits a corrective/preventive plan complete with a Request to Proceed.
 - iii. Any approved corrective and/or preventive measure deemed necessary by the Contractor is implemented.
 - iv. Operations shall not proceed until the Contract Administrator has issued a Notice to Proceed for each corrective/preventive plan.

7.09 Certificate of Conformance

A Certificate of Conformance shall be submitted to the Contract Administrator upon completion of the installation of the pipe at each location. In addition, upon completion of the installation of the pipe at each location, the Contractor shall submit to the Contract Administrator a final Quality Control Certificate sealed and signed by the Design Engineer and the Design Checking Engineer. The Certificate shall state that the pipe has been installed in general conformance with the Contractor's Submission and Design Requirements, sealed Working Drawings and Contract Documents.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT

Measurement shall be by Plan Quantity Payment as may be revised by Adjusted Plan Quantity Payment in metres, following along the centreline of the pipes from centre to centre of maintenance holes or chambers (catch basins) or from/to the end of the pipe where no maintenance hole or chamber is installed, of the actual length of pipe installed by trenchless methods.

10.0 BASIS OF PAYMENT

Payment at the Contract price shall be full compensation for all labour, Equipment, and Material required for excavation (regardless of material encountered), dewatering, sheathing and shoring, settlement instrumentation and monitoring, site restoration, and all other work necessary to complete the installation as specified.

Where a protection system is made necessary because of the Contractor's operations (e.g., choice of trenchless installation method), the cost shall be included in this item and shall be full compensation for all labour, Equipment, and Materials required to carry out the work including subsequently removing the temporary protection system and performing any necessary restoration work.

Payment for connecting intercepted drains and service connections shall be made on the following basis:

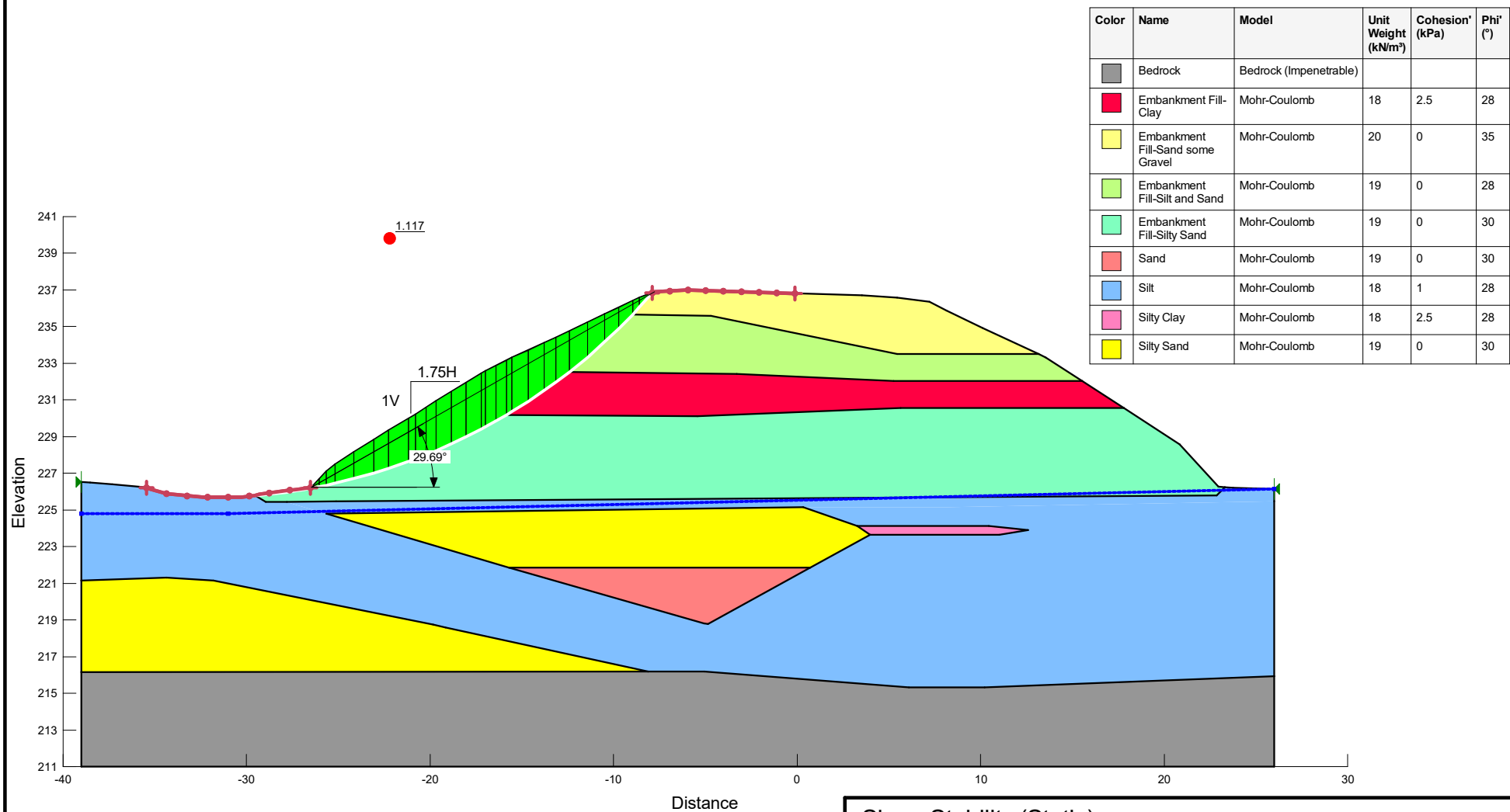
- (a) Where such drains and service connections are shown on the contract drawings the cost of connections shall be included in the contract price for pipe installation.
- (b) Where such drains and service connections are not shown on the contract drawings, the cost of connections will be considered an allowable extra to the contract.

Appendix G

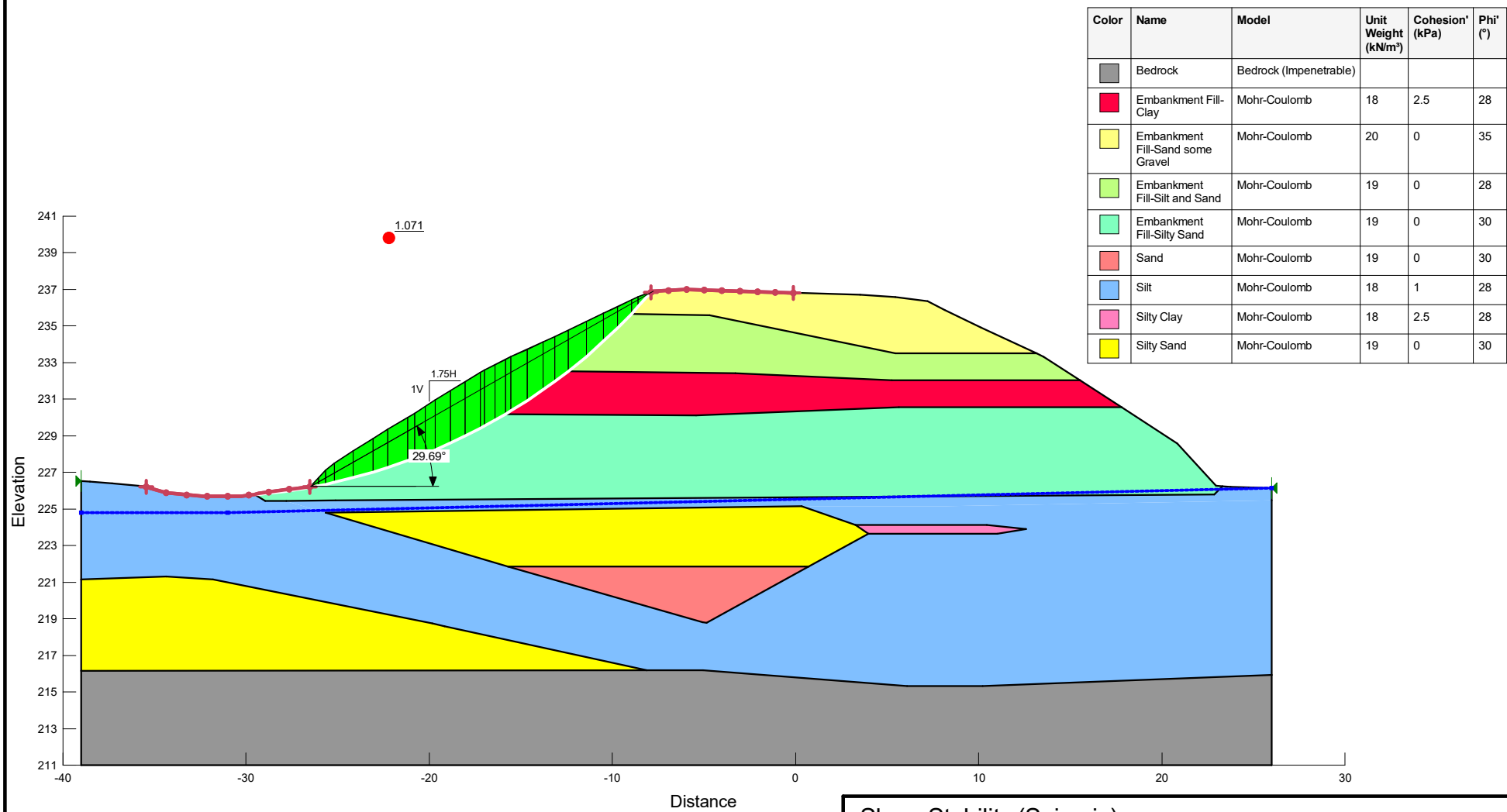
Slope Stability Assessment



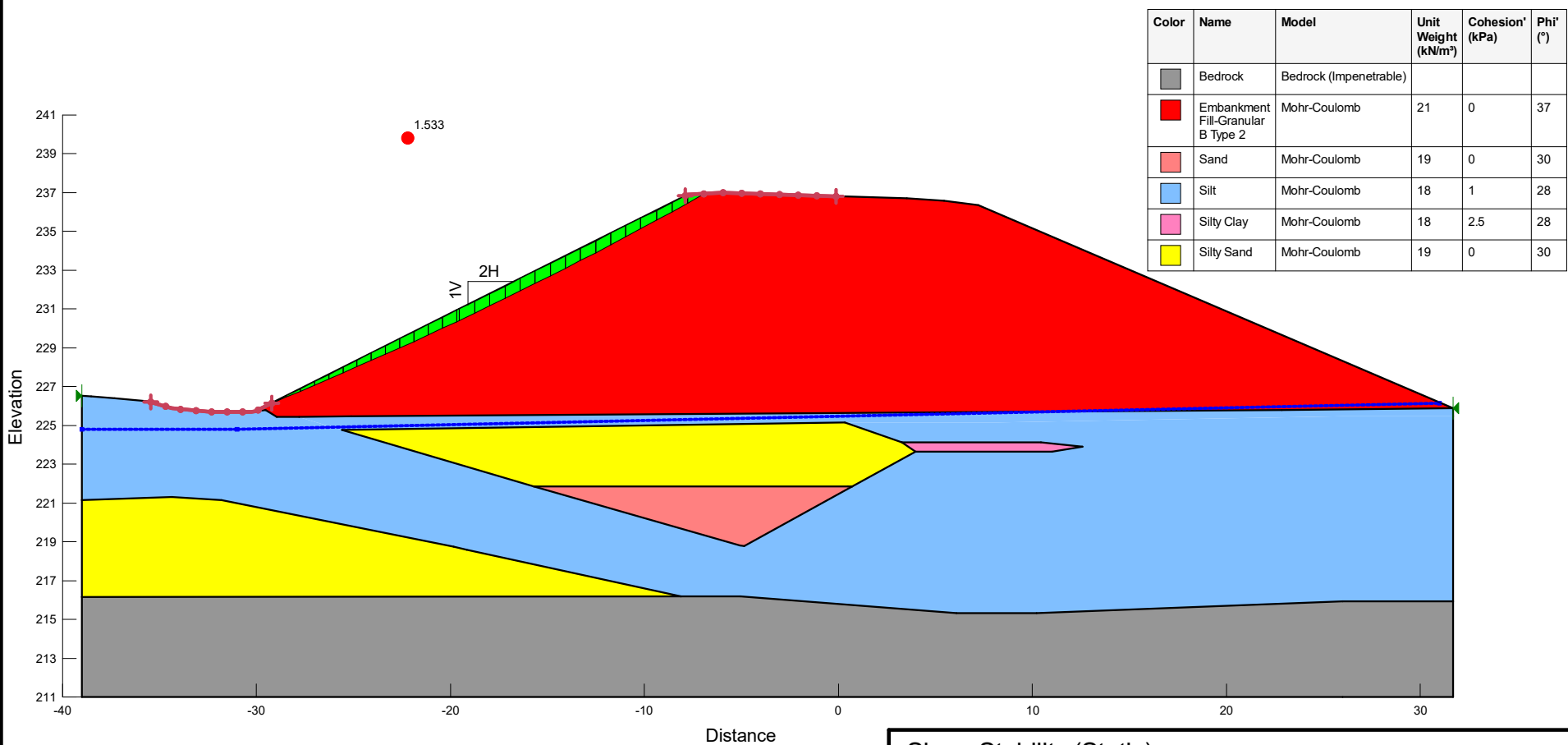
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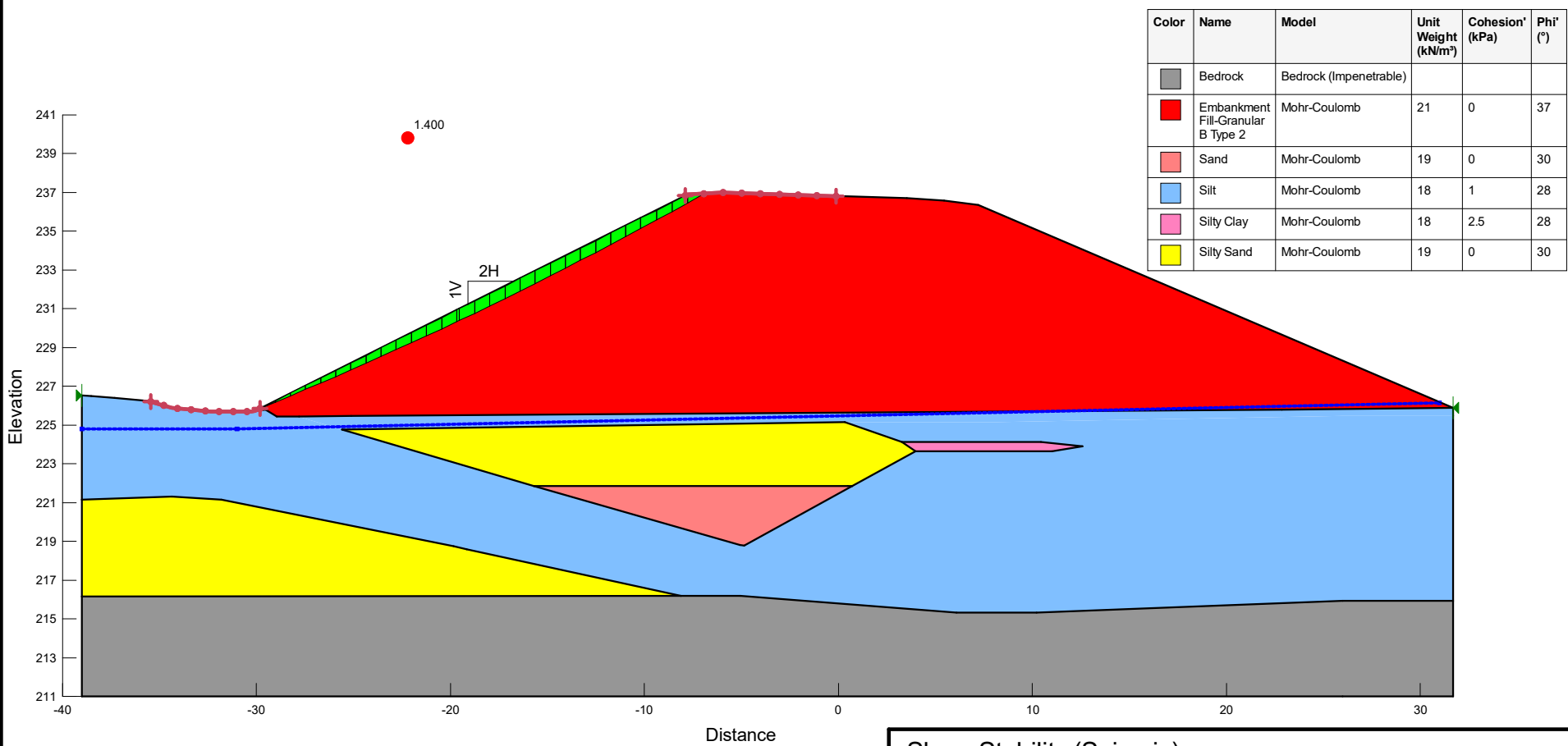


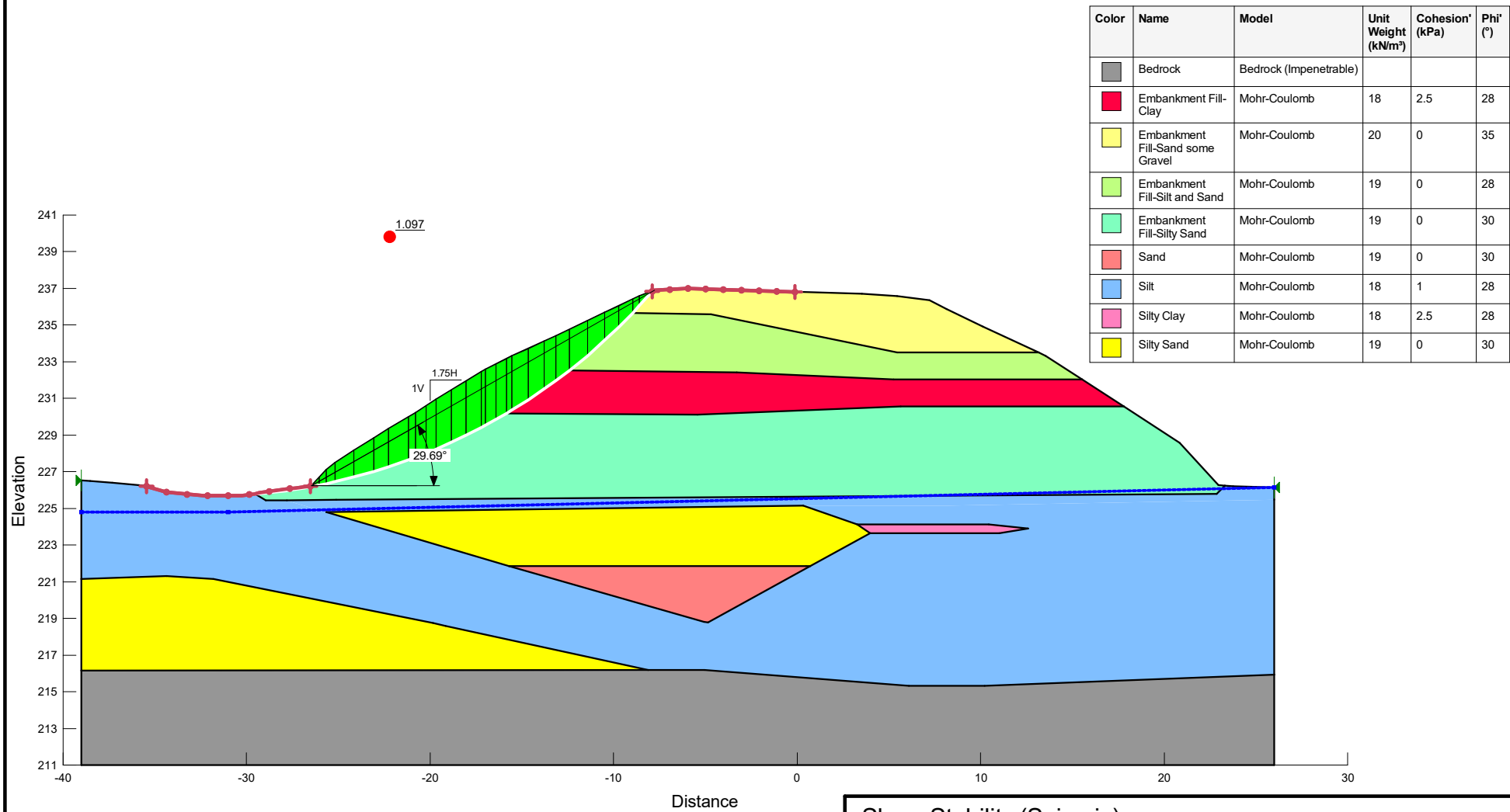
Slope Stability (Static)	S1
Hwy 61-Station 20+375-Existing Condition.gsz	
2022-03-10	Scale 1:320



Slope Stability (Seismic)	S2
Hwy 61-Station 20+375-Existing Condition.gsz	
2022-03-10	Scale 1:320







Slope Stability (Seismic)	S5
Hwy 61-Station 20+375-Assumed Acceleration for pipe ramming.gsz	
2022-03-10	Scale 1:320

