FINAL REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

Minnikau River Culverts Replacement, Highway 642, Site No. 41S-255/C, District of Kenora

Agreement No. 6014-E-0017
Assignment No. 7
GWP 6912-12-01
Geocres No. 52G-014

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Foundation Investigation and Design Report for Minnikau River Culverts Replacement
Highway 642, Site No. 41S-255/C, District of Kenora

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PART I FOUNDATION INVESTIGATION REPORT

1.1 Introduction

This foundation investigation report presents the results of a geotechnical investigation completed by **exp** Services Inc. for the replacement of the three (3) Minnikau River Culverts, located on Highway 642, about 35 km east of the junction of Hwy 516 and Hwy 642, in the District of Kenora, the Ministry of Transportation (MTO) Northwestern Region. The work was undertaken under Agreement # 6014-E-0017, Assignment No. 7 (GWP 6912-12-01). The terms of reference (TOR) were as presented in the MTO letter dated July 7, 2015.

Based on preliminary information provided and our observations on site, the existing culverts are structural plate corrugated steel pipe with diameters of about 2.44 m and lengths ranging between about 18.31 m and 18.45 m. It is understood that the existing culverts were constructed at an unknown date, and are intended to be replaced with a new culvert or culverts along the same alignment.

The purpose of the investigation was to evaluate the subsurface conditions along the alignment, to permit detailed design for the replacement of the culverts. The site specific geotechnical investigation consisted of borings, soil sampling, borehole logging, and field and laboratory testing.

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

1.2 Site Description and Geological Setting

1.2.1 Site Description

As shown on Drawing 1 (Appendix B), the Minnikau River Culverts are located on Highway 642, about 35 km east of the junction of Hwy 516 and Hwy 642, in the District of Kenora, southeast of Sioux Lookout, Ontario. At the site, Hwy 642 is a two lane roadway, with a speed limit of 80 km/h and is about 7.0 m wide from edge of pavement to edge of pavement, with sand and gravel shoulders. Based on drawings provided, the roadway embankment is about 3.4 m high with side slopes of about 2H:1V.

Highway 642 at the Minnikau River Culverts location, runs generally in a northwest-southeast direction, and the Minnikau River generally flows from east to west. However, for simplicity and for the purposes of this report a “project north” has been established and project north is oriented perpendicular to the centerline of Hwy 642 (i.e. project north is in the same direction as true north’s northeast direction). The orientation of project north is presented on Drawing 1 in Appendix B. Hereinafter, the directions indicated in this report are in referenced to project north.

During the fieldwork on August 10, 11, 12 and 27, 2015, the general site conditions were assessed. Hwy 642 runs in a generally east and west direction and the water in the Minnikau River generally flows from north to south beneath the highway. At the time of this investigation, the approximate river elevations at the inlet and outlet were about 391.42 m and 391.38 m, respectively, and the streambed elevations at the inlet and outlet were about 390.9 m and 390.7 m, respectively. The elevation of highway pavement centerline at the middle culvert centerline is about 394.10 m.

At the vicinity of the inlet and outlet of the culvert some minor vegetation was noted at both culvert ends. The surrounding area of the culvert at the east and west side of Hwy 642 was surfaced with tall grasses and forested further away from the culverts. The inlet and outlet appeared to be generally clear of debris and excess vegetation, and as such the flow does not appear to be restricted.

Select photographs are provided in Appendix A.

1.2.2 Geological Setting

According to the MNR Northern Ontario Engineering Geology Terrain Data Base Map, Ontario Geological Survey Map 5062, Scale 1:100,000, dated 1979, the underlying native soil at the site consists of sand till ground moraine overlying bedrock with a drift veneer, and subordinate landforms consisting of silt and sand glaciolacustrine plain. The topography of the site is indicated as low local relief with rolling to undulating terrain and mixed wet and dry surface conditions.

According the Ministry of Northern Development and Mines (MNDM) Bedrock Geology of Ontario, West-Central Sheet Map No. 2542, Scale 1:1,000,000, dated 1991, the bedrock geology of the site is mafic to intermediate metavolcanic rock from the Neo to Mesoarchean Era (2.5 to 3.4 Ga), and generally consists of basaltic and andesitic flows, tuff and breccia. It may also contain, chert, iron formations, minor sedimentary and intrusive rocks along with related migmatites.

1.3 Investigation Procedures

1.3.1 Site Investigation and Field Testing

The field investigation was performed on August 10, 11, 12 and 27, 2015. The field program consisted of drilling four (4) sampled boreholes (BH301 to BH304). Two (2) boreholes were located within the highway, BH301 and BH302. BH301 was located about 4.2 m east of the east culvert centerline and about 2.7 m north of the highway centerline. BH302 was located about 4.2 m west of the west culvert centerline and about 1.9 m south of the highway centerline. An additional two (2) boreholes (BH303 and BH304) were advanced off of the highway. BH303 was located about 18 m west of the west culvert centerline and about 8.2 m north of the highway centerline on the inlet/upstream side of the culvert. BH304 was located about 11 m east of the culvert centerline and about 10 m south of the highway centerline on the outlet/downstream side of the culvert. The borehole locations are shown on Drawing 1 in Appendix B.

All the boreholes (BH301 to BH304) were advanced using a CME 850 track mounted drill rig. The drill rig was equipped with hollow stem continuous flight augers and standard soil sampling equipment (includes 51 mm outside diameter split spoon samplers and *in situ* shear vane testing equipment). In addition, the CME 850 drill rig was equipped with rock coring equipment (HQ size).

BH301 was advanced to a depth of about 9.8 m below ground surface, and rock coring techniques were used to continue the borehole beyond refusal to a depth of about 13.1 m below ground surface. Rock coring techniques were not used at the remaining three boreholes.

BH302 was advanced to about 13.6 m below ground surface, and the off-road boreholes (BH303 and BH304) were advanced, to depths of about 12.5 m and 6.7 m, respectively. BH302, BH303 and BH304 were terminated at auger and/or SPT refusal depths.

The borehole locations were referenced to the MTM ON-16 NAD83 coordinate system and their ground surface elevations were surveyed by **exp** personnel. The ground surface elevations, including top of water in the river, were referenced to a geodetic benchmark (BM) provided (nail in tree root) east of the site and south of the highway. The BM elevation is 393.900 m. The location of the BM is shown on Drawing 1, in Appendix B.

During the drilling of the boreholes (BH301 to BH304), soil samples were obtained using a 51 mm outside diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586), and were generally performed at intervals of about 0.75 m. The original field (uncorrected) SPT “N” values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual and used to provide an assessment of *in-situ* compactness (cohesionless) or consistency (cohesive) soils.

Upon completion of the boreholes, groundwater level measurements were carried out in boreholes in accordance with the Ministry of Transportation guidelines. The measured groundwater levels after completion of drilling boreholes were recorded on borehole log sheets in Appendix C. The boreholes were backfilled with a mixture of bentonite and auger cuttings. The borehole decommissioning was in general accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the *Ontario Water Resources Act)*.

The fieldwork was supervised by a member of **exp**’s engineering staff who directed the drilling and sampling operation, logged borehole data in accordance with MTO and/or ASTM Standards for Soils Classification, and retrieved soil samples. All of the recovered soil samples were placed in labelled moisture-proof bags which, along with the rock cores, were brought to **exp**’s Thunder Bay laboratory for additional visual, textual and olfactory examination, and for subsequent examination by a geotechnical engineer and laboratory testing.

1.3.2 Laboratory Testing

All samples brought to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content and particle size distribution for approximately 25% of the collected soil samples. Atterberg Limits tests were carried out on select cohesive soil samples. All of the laboratory tests were carried out in accordance with MTO and/or ASTM Standards, as appropriate, at the **exp** laboratory in Thunder Bay, Ontario.

The laboratory test results are provided on the attached borehole log sheets in Appendix C as well as graphically in Appendix D.

In addition, chemical testing of two select soil samples were conducted. The soil samples were sent via courier, in a secure cooler under chain of custody, to Maxxam Analytics Inc., a CALA-certified and accredited laboratory in Mississauga, Ontario. Details of the chemical testing are discussed below and the lab results are included in Appendix E.

1.4 Subsurface Conditions

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the Borehole Records in Appendix C. Laboratory test results are provided in Appendix D. The "Explanation of Terms Used on Borehole Records" preceding the borehole logs in Appendix C forms an integral part of and should be read in conjunction with this report.

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

In general, the subsurface conditions along the proposed culvert alignment consist of a layer of fill material composed of sand, overlying native sand, overlying clayey silt overlying silt, overlying silty sand and cobbles and boulders. A more detailed summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

1.4.1 Poorly Graded Sand with Silt and Gravel Fill

Poorly graded sand with silt and gravel fill was encountered beneath the asphalt at BH301 and BH302, and beneath the rootmat at BH303. The asphalt thickness at BH301 and BH302 was about 25 mm. The rootmat thickness was about 100 mm. The fill was generally described as compact to very dense, brown, damp to moist, and containing occasional cobbles. The SPT "N" values ranged between 3 and 56 blows per 300 mm penetration, with an average "N" value of about 22. The fill extended to a depths ranging between about 1.1 m and 3.1 m below ground surface, and at elevations ranging between about 391.0 m and 391.6 m.

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

Moisture content:

- 3.8% to 16.6%

Grain size distribution:

- 17% gravel;
- 78% sand and;
- 5% silt and clay size.

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 1, in Appendix D.

1.4.2 **Peat**

Peat was encountered surfacing BH304 and beneath the fill at BH303. The peat was generally described as very soft to soft, dark brown to brown, moist to wet, containing trace sand to sandy, and trace silt. The SPT "N" values ranged between 0 (i.e. advanced by weight of hammer and rods alone) and 2 blows per 300 mm penetration, with an average "N" value of about 1. The peat thickness ranged between about 0.1 m and 1.3 m and extended to depths ranging between about 0.1 m and 2.4 m below ground surface. The peat extended to elevations ranging between about 390.2 m and 392.5 m. Laboratory testing performed on selected samples consisted of moisture content. The test results are as follows:

Moisture content:

- 11.7% to 112.6%

The results of the moisture content tests are provided on the record of borehole sheets in Appendix C.

1.4.3 **Poorly Graded Sand to Poorly Graded Sand with Silt**

Native poorly graded sand to poorly graded sand with silt was encountered beneath the fill at BH301 and BH302, and encountered beneath the peat at BH304. The native sand was generally described as very loose to loose, brown, moist to wet. Trace organics were observed at BH301 and some roots and rootlets were observed at BH304. At BH302, sand blowup, about 0.25 m to 1.0 m in thickness, was noted in the augers at about 3.8 m and 5.2 m depth, respectively. The SPT "N" values ranged between 0 (i.e. advanced by weight of hammer and rods alone) and 6 blows per 300 mm penetration, with an average "N" value of about 3. The native sand extended to depths ranging between about 0.9 m to 6.7 m below ground surface, with elevations ranging between about 387.3 m and 391.7 m.

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

Moisture content:

- 7.1% to 18.8%

Grain size distribution:

- 0% to 13% gravel;
- 81% to 96% sand;
- 4% to 6% silt and clay size.

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 20.9 to 23.8 kN/m^3.

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 1, in Appendix D.

1.4.4 Clayey Silt

Clayey silt was encountered underlying the native sand at BH301 and underlying the peat at BH303. The clayey silt was generally described as very soft to stiff, grey, and wet. At BH301, occasional 10 mm interbedded peat layers were noted in the upper 5.2 m, and at about 6.1 m depth, the clayey silt was becoming varved. The SPT "N" values ranged between 0 (i.e. advanced by weight of hammer and rods alone) and 2 blows per 300 mm penetration, with an average "N" value of about 1. Four (4) *in situ* field vane test were performed and the results at ranged between about 21 kPa and 84 kPa, respectively. The clayey silt extended to depths ranging between about 5.3 m and about 7.0 m below ground surface, and elevations ranging between 387.0 m and 387.3 m.

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution and Atterberg Limit tests. The test results are as follows:

Moisture content:

- 18.3% to 50.7%

Grain size distribution:

- 0% gravel;
- 6% to 9% sand;
- 64% to 66% silt; and
- 25% to 30% clay size.

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 16.8 to 21.0 kN/m^3. Two (2) Atterberg Limits tests were performed on representative samples of the clayey silt (BH301-S6 and BH303-S5). The results indicated that the soil is of low plasticity. The data is shown on the plasticity chart, Figure 5. The liquid limit, plastic limit and plasticity index ranged between about 24 and 27, 14 and 15, and 9 and 13, respectively.

The results of the moisture content, grain size distribution and Atterberg Limits tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution are also provided on Figure 2 in Appendix D, and Atterberg Limits tests are provided on Figure 5 in Appendix D.

1.4.5 Silt

Silt was encountered beneath the clayey silt at BH301 and BH303, and beneath the native sand at BH302 and BH304. The silt was described as very loose to compact, grey, and wet. At BH303, varved soils were noted from about 6.1 m to 7.0 m depth. The SPT "N" values ranged between 0 (i.e. advanced by weight of hammer and rods alone) and 20 blows per 300 mm penetration, with an average "N" value of about 6. Three (3) *in situ* field vane tests were performed, as some cohesive properties were noted, yielding results ranging between about 42 kPa and 133 kPa. The silt extended to depths ranging between about 3.8 m and 9.2 m below ground surface. The silt extended to elevations ranging between about 383.5 m and 388.8 m.

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution and Atterberg Limit tests. The test results are as follows:

Moisture content:

- 17% to 47.3%

Grain size distribution:

- 0% gravel;
- 1% to 11% sand;
- 84% to 90% silt; and
- 9% to <89% clay size.

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 17.2 to 21.2 kN/m^3. Four (4) Atterberg Limits tests were performed on representative samples of the silt (BH301-S10, BH302-S10, BH303-S9 and BH304-S3) as some cohesive properties were noted. The results indicated that the soil is of low plasticity and the soil contained more cohesionless properties than cohesive properties. The data is shown on the plasticity chart, Figure 6. The values of the liquid limit, plastic limit and plasticity index ranged between about 19 and 21, 15 and 16, and 4 and 6, respectively.

The results of the moisture content, grain size distribution and Atterberg Limits tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution are also provided on Figure 3 in Appendix D, and Atterberg Limits tests are provided on Figure 6 in Appendix D.

1.4.6 **Silty Sand to Sandy Silt with Gravel**

Silty sand was encountered beneath the silt at BH301, BH302 and BH303, and sandy silt with gravel was encountered underlying the silt at BH304. The silty sand to sandy silt with gravel was generally described as very loose to very dense at depth, grey, and wet. Occasional cobbles were noted at BH302 and BH304. Sand blowup in the augers was noted at BH301, BH302 and BH303, and the blowup thickness ranged between about 300 mm and 2.4 m. The SPT “N” values ranged between 0 (i.e. advanced by weight of hammer and rods alone) and 72 (i.e. SPT refusal) blows per 300 mm penetration, with an average “N” value of about 20. Two SPT N values of 100 were noted at refusal / termination depths and are unlikely to be representative of the silty sand or sandy silt with gravel. The silty sand and sandy silt with gravel extended to depths ranging between about 6.7 m and 13.6 m below ground surface, with elevations ranging between about 380.1 m and 385.9 m.

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

Moisture content:

- 8.4% to 19.0%

Grain size distribution:

- 0% to 15% gravel;
- 19% to 68% sand;
- <32% to 59% silt; and

- 1% to <45% clay size.

Total saturated unit weights have been calculated based on the moisture contents and are estimated to range from about 20.8 to 23.4 kN/m^3.

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figures 3 and 4, in Appendix D.

1.4.7 Cobbles and Boulders

Cobbles and boulders were encountered underlying the silty sand at BH301. The borehole was extended using rock coring techniques about 3.3 m into the cobbles and boulders layer to a depth of about 13.1 m (380.9 m elevation) below ground surface.

1.5 Groundwater and Surface Water Conditions

Information on groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling. The groundwater levels encountered in the boreholes are shown on the borehole logs and presented below in Table 1.1.

Seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year and lower levels during drier periods.

Table 1.1. Groundwater data

Borehole	Date Completed	Date Measured	Ground Surface Elevation[2]	Depth to Water[3]	Groundwater Elevation
BH301	Aug. 10/15	Aug. 10/15	394.01	1.78	392.23
BH302	Aug. 11/15	Aug. 11/15	394.04	2.85	391.19
BH303[4]	Aug. 27/15	Aug. 27/15	392.63	1.07	391.56
BH304	Aug. 12/15	Aug. 12/15	392.57	0.30	392.27
Minnikau River WL Upstream (North) Side	--	Aug. 12/15			391.42[5]
Minnikau River WL Downstream (South) Side	--	Aug. 12/15	--	--	391.38[5]

Borehole	Date Completed	Date Measured	Ground Surface Elevation[2]	Depth to Water[3]	Groundwater Elevation

Notes:
1) All units in metres.
2) Elevations surveyed are referenced to a geodetic benchmark (BM) provided (nail in tree root) east of the site and south of the highway. The BM elevation is 393.900 m.
3) Depths are relative to ground surface.
4) Drilling at BH303 was completed on August 12, 2015. The borehole was left open to measure groundwater levels prior to backfilling on August 27, 2015.
5) Indicates highest elevation value of the top of surface water at Minnikau River.

1.6 Chemical Analyses

Two soil samples were selected for chemical analyses and were sent via courier, in a secure cooler under chain of custody, to Maxxam Analytics Inc., a CALA-certified and accredited laboratory in Mississauga, Ontario. The analytical laboratory results are presented in Appendix E, and are summarized in Table 1.2, below.

Table 1.2. Corrosivity Chemical Analysis

Borehole	pH (unitless)	Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (µS/cm)
BH302-S5	5.99	<20	<20	27,000	38
BH303-S4B	7.34	<20	<20	5,600	178

PART II ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

2.1 General

This section of the report provides geotechnical design recommendations for the replacement of the three (3) Minnikau River Culverts, located on Highway 642, about 35 km east of the junction of Hwy 516 and Hwy 642, in the District of Kenora, the Ministry of Transportation (MTO) Northwestern Region. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site and presented in **Part I-Foundation Investigation Report**. The interpretation and recommendations provided are intended solely to permit designers to assess foundation alternatives and design the new culvert and replacement. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

Based on information included in the TOR, it is understood that the existing three culverts are structural plate corrugated steel pipes with diameters of about 2.44 m and lengths ranging between about 18.31 m and 18.45 m. It is understood that the existing culverts were constructed at an unknown date, and they are inspected in July 2013. The inspection found that existing culverts are in very poor condition. It is reported that the barrels are rusted and damaged at the inlet and outlet sides. Settlements of the middle and south barrels are reported as well. Erosion of the embankment slopes above the middle culvert was also observed. It is understood that the existing culverts are going to be replaced with a new culvert or culverts along the same alignment, as well as that the road grade will be the same as that at the location of the existing culvert. The size and type of the new culvert is not defined at the time of writing this report. However for preliminary design purposes, the following options are being considered for the replacement: steel sheet pile abutment with precast concrete decking, rigid frame box culvert/culverts (precast or cast-in place), rigid frame open footing culvert/culverts (precast or cast-in-place), and corrugated steel plate culvert/culverts.

This part of the report addresses the geotechnical design of the foundation for the new culvert by providing geotechnical design parameters at the Ultimate Limit State (ULS) and Serviceability Limit States (SLS) as well as other geotechnical parameters that may be required in accordance with the latest edition of the *Canadian Highway Bridge Design Code (CHBDC) (CAN/CSA-S6-14)*, the *Canadian Foundation Engineering Manual (CFEM)* (2006), *MTO Gravity Pipe Design Guidelines* (May 2007) and generally accepted good practice. Pertinent construction issues from a geotechnical standpoint are examined in general accordance with the Terms of Reference from the MTO letter dated May 27, 2015. The assessment involved review of options for replacement of the existing culvert along the same alignment with a final selection to be made by the designer, based on the optimum solution.

2.2 Expected Ground Conditions

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

a. Hwy 642 is a two lane roadway, with a speed limit of 80 km/h and is about 7.0 m wide from edge of pavement to edge of pavement, with sand and gravel shoulders. Based on drawings provided, the roadway embankment is about 3.4 m high with side slopes of about 1H:1V. The current elevation of the crest of the roadway is about 394.10 m.
b. The highway embankment consists of compact to very dense poorly graded sand with silt and gravel fill (~3.1 m thick).
c. The embankment fill is underlain by very loose to loose poorly graded sand to about 4.0 m to 6.7 m (~0.9 m to 3.6 m thick) below ground surface, followed by very soft to stiff clayey silt to about 7.0 m (~1.3 m thick) below ground surface in BH301, and loose silt to about 9.2 m (~2.5 m thick) below ground surface in BH302. The thin layer of very loose silt (~0.7 m thin) underlies clayey silt in BH301. The silt is underlain by very loose to very dense silty sand to about 9.8 m to 13.6 m (~2.1 m to 4.4 m thick) below ground surface and followed by a layer of cobbles and boulders.
d. At the inlet and outlet native deposits of very soft to soft peat, very soft to soft clayey silt, very loose to compact silt, very loose silty sand and loose to compact sandy silt with gravel are encountered below ground surface.
e. The foundation soil at the invert of the new culvert is anticipated to be native poorly graded sand with silt at Elev. 391.0 m underline by very soft to soft clayey silt in the northern portion of the culvert (see BH301). Typical ‘N’ values of the sand with silt layer within the zone of influence ranged from 1 to 6.
f. The groundwater table in the embankment fill is expected to be at approximate elevation 391.5 m, or slightly higher. However, seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year (such as spring thaw and late fall) and lower levels during drier periods. Some groundwater mounding within the embankment and perched water would be anticipated.

2.3 Structure Foundations

For preliminary design purpose, several options are being considered for the replacement:

- frame box culvert/culverts (precast or cast-in-place),
- rigid frame open footing culvert/culverts (precast or cast-in-place) supported on deep foundations,
- corrugated steel plate culvert with concrete/culverts footing supported on deep foundations, and
- steel sheet pile abutments with precast concrete deck.

Based on the subsurface information obtained from the boreholes, the native sand with silt at Elev. 391 m is considered suitable for the support of frame box culvert/culverts. However, for the rigid frame open footing culvert/culverts and corrugated steel plate culvert/culverts with concrete footing supported on deep foundations is more preferable alternatives from a geotechnical/foundation perspective than the shallow foundations considering the presence of soft clayey silt and/or loose sand at the foundation level of Elev. 388.5 m (below the frost line of 2.5 m). The option of steel sheet pile abutments with precast deck is also possible alternative for this site. However, driving these piles could be difficult considering that the experience in the general area shows that in many cases the surface of the bedrock can frequently be uneven and unpredictable. This has been confirmed with this investigation finding the different levels of practical refusal as well as the layer of cobbles and boulders in BH301 which could be the highly fractured weathered zone above the bedrock surface.

It should be noted that the choice of culvert type will also depend on parameters such as the initial cost, maintenance costs, hydraulic performance, ease of construction, salvageability and local availability of material and equipment.

Table 2.1 Evaluation of foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
Rigid frame box culvert	2	▪ Straightforward construction ▪ Reduce construction period, consequently traffic management and water control period ▪ Reduce excavation depth	▪ Dewatering system required ▪ Require heavy lifting equipment ▪ Require bedding material	▪ Low to medium	▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of leaking from joints if not properly installed
Rigid frame open footing concrete culvert	4	▪ Wider span may consider to maintain existing channel ▪ High geotechnical resistance available ▪ Can incorporate dowels to enhance lateral resistance	▪ Deeper excavation or below water excavation may required ▪ Dewatering system required ▪ Possible uneven bedrock surface ▪ Require placement of lean concrete	▪ Likely more expensive than Option 1	▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of delay in construction due to deeper excavation below water if proper dewatering is not maintain ▪ Higher scour risk
Corrugated Steel Pipe Culvert	3	▪ Straightforward construction ▪ Reduce construction period, consequently traffic management and water control period	▪ Dewatering system required ▪ Require bedding material ▪ Limited design life ▪ Potential for Corrosion	▪ Low to medium	▪ Risk of unacceptable differential settlements if the entire foundation is not supported on the competent soil ▪ Risk of structure segment loss due to corrosion

Options	**Rank**	**Advantages**	**Disadvantages**	**Relative Costs**	**Risks/ Consequences**
		▪ Reduce excavation depth			
Steel Sheet Pile abutment with precast decking	1	▪ Environmentally friendly ▪ Easy to construct ▪ No need for dewatering and cofferdam ▪ serve as dual purpose of support culvert foundation and retaining backfill ▪ reduce construction period	▪ Require pile driving equipment ▪ May require anchors to support possible later movement ▪ Difficulties for sheet pile driving through cobbles and boulders ▪ Durability issue with sheet pile walls	▪ Medium to High	▪ Risk of frictional pile may not develop full capacity consequently risk of potential settlement ▪ May be limited steel sheet pile sections

Table 2.1 compares the structure options from a foundations design and constructability perspective. Although the foundation soils are generally good and will provide adequate support for all options listed in the table, the use of steel sheet pile abutment with precast decking is recommended.

2.3.1 **Shallow Foundations**

2.3.1.1 Geotechnical Resistance

Based on the subsurface stratigraphy encountered at this site and the assumed invert elevation of the culvert/culverts, the recommended founding depth and geotechnical resistances for a structure founded on undisturbed competent natural soils are tabulated below.

Table 2.2 Recommended spread footing design parameters

Culvert Type	Founding Elevation (m)	Assumed Footing Size (m)	Founding Soil Type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS*
Box Culvert	~391.0	6	Minimum 300 mm compacted granular material (Granular A or Granular B Type II) over native loose sand with silt	150	100

** for maximum settlement of 25 mm*

It is presumed that any soft or very loose materials are to be replaced with clean and compactable soil such as Granular B Type II. Given that no significant grade raise is planned, the anticipated maximum total settlements for the new proposed culvert are not expected to exceed 25 mm for construction done in accordance with these design parameters and assuming good construction practice including sound base preparation.

2.3.1.2 Resistance to Lateral Loads

Resistance to lateral forces/ sliding should be calculated in accordance with Section 6.10.5 of the CHBDC, using the following parameters:

Table 2.3 Recommended parameters for calculation of unfactored horizontal resistance

Interface and loading conditions	Parameters
Between Granular A pad and pre-cast concrete	Coefficient of friction (tan δ)=0.5

The listed values are unfactored; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

2.3.1.3 Frost Protection

The frost penetration depth at the Minnikau River culvert is 2.5 m according to OPSD 3090.100. Any temporary or permanent support system using shallow foundations should be provided with a minimum 2.5 m of soil cover or equivalent thermal insulation for frost protection.

2.3.2 Deep Foundations

2.3.2.1 Driven Steel Piles

Axial Resistance in Compression

Driven steel piles can be used to support the culvert footings. Piles can consist of steel (minimum 350 MPa) pipe (open or closed) or HP sections. Such piles, driven into the underlying competent soil below Elev. 381 m (to practical refusal) can be designed using the factored (0.4) resistance values in the following Table 2.4. These are only typical sizes. These values result from a static analysis based on skin friction only, and using the effective stress β method. The elastic compression at ULS should be less than 10 mm in all cases. Since there is no (or minimal) proposed grade raise, negative skin friction or drag loads are not a concern.

Table 2.4 Factored geotechnical resistance values (ULS) for driven steel piles

Pipe Size or HP Section	Factored ULS (kN) for Embedment (below pile cap) (L=10 m)
244 mm x 9 mm	110

324 mm x 10 mm	150
HP310 x 79	160

The values given in Table 2.4, above, based on static analysis considering skin friction in loose to compact sand and silt only, driving the piles to 10 m depth below the underside of the pile cap. These capacities are relatively low, noting the nature and consistency of the soil. This investigation has identified the surface of a cobble and boulder layer below the loose to compact sand and silt. Further investigation would be required to examine this layer in order to provide definitive recommendations on high capacity piles and driving conditions, if this is to be explored.

Prior to driving piles, a wave equation (WEAP) analysis should be performed in order to assess the driving stresses and the anticipated penetration resistance required to develop the required pile capacity. This analysis considers the complete driving system. Dynamic testing (PDA testing) on a number of piles with the Pile Driving Analyser must be performed near the beginning of the pile driving phase of construction to confirm the pile capacities. Alternatively, static load tests can be performed, although these are typically much more difficult to set up and are more costly.

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

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

Frost protection should be provided for pile caps, if used, and the frost depth requirement at this site is 2.5 m according to OPSD 3090.100. If required, a layer of polystyrene (such as STYROFOAM or FOAMULAR C-300) may be used to reduce frost penetration. The material should be placed above the pile cap, just below the surface (at least 300 mm below final grade) to protect it from physical damage. The edge of the polystyrene should extend at least 1.3 m from the outside edge of the underlying pile cap. The material should be installed as per the manufacturer's instructions.

Lateral Resistance

In accordance with Table C6.4 of the CHBDC Commentary (2006), for HP 310x79 piles installed at this site, the lateral resistance at ULS and lateral reaction at SLS (for 10 mm) can be taken as 110 kN and 40 kN, respectively. The values for the pipe pile sizes given above will be somewhat less. The lateral resistance is shown in Table 2.5, below.

Table 2.5 Assessed horizontal passive resistance and bearing reaction for driven steel piles

Pipe Size or HP Section	Assessed Horizontal Passive Resistance and Bearing Reaction (kN)	
	ULS	SLS (10 mm)
244 mm x 9 mm (estimated)	100	25
324 mm x 10 mm (estimated)	120	50
HP310 x 79 (CHBDC Commentary – Table C6.4)	110	40

2.3.2.2 Steel Sheet Piles

Sheet piles can be used for retaining backfill soil during excavation, as well as bearing elements to support culvert foundations for the option culvert replacement with steel sheet pile abutments and precast concrete decking. For design, a PZ-22 section can be considered. Driving of these piles could be obstructed by the layer of cobbles and boulders encountered in BH301.

Axial Resistance in Compression

The factored resistance values (per metre width of sheet pile) for the sheet piles have been calculated as 140 kN for 10 m embedment. This value is based on a static analysis, considering skin friction only (end bearing resistance is negligible), using the effective stress β method, similar to the steel piles described above. It is noted that, since the sheet piles will also be retaining the approach fills, only the embedded, outside portion of the sheet piles below the level of the creek bed is considered to contribute to axial resistance. The elastic compression at ULS should be less than 6 mm in all cases. Since there is no (or minimal) proposed grade raise, negative skin friction or drag loads are not a concern.

Lateral Resistance

For relatively short (typically less than 3 m to 4 m) abutments, a cantilever sheet pile design using the earth pressure coefficients and soil parameters provided in Section 2.4, following. Note that if this design is implemented, the precast concrete deck will likely be designed to be installed such that lateral support is provided at the top of the sheet piles.

Depending on the abutment height and steel sheet section used, additional anchorage or tiebacks may be required. Conventional practice is to incorporate either buried deadman anchors or grouted soil anchors.

Deadman anchors can be designed based on the earth pressure coefficients and soil parameters provided in Section 2.4, following. For this project, either continuous or individual concrete block anchors would likely be appropriate. The anchor resistance is provided by a combination of the dead weight and passive resistance. For the full passive resistance to be realized with no load transfer to the wall, the anchor needs to be fully beyond the active wedge acting on the wall.

Pressure grouted soil anchors can be designed in a preliminary fashion in accordance with Section 26 of the CFEM (2006). Based on the generally stiff soils at this site, the estimated factored (0.4) ULS resistance of grouted anchors would be 12 kN/m length. Detailed design would be completed following the design of the wall and the loads have been established. Normally, such anchors are supplied and installed/tested by specialist vendors/contractors.

2.4 Lateral Earth Pressure

Culvert walls at the outlet and inlet, and temporary shoring that may be required for excavation should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure is given by:

$$P = K(\gamma h + q) \text{ for non-braced cut, or } K(0.65\gamma h + q) \text{ for braced cut}$$

where P = earth pressure intensity at depth h, kPa
K = earth pressure coefficient
γ = unit weight of retained soil, kN/m^3
q = surcharge near wall, kPa
h = depth to point of interest, m

The above expression does not take into account hydrostatic pressure, which must be included for the groundwater levels measured on the site. Table 2.6 lists earth pressure parameters for given materials.

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in design.

The effect of compaction surcharge should be taken into account in the calculations of active and at- rest earth pressures. The lateral pressure due to compaction should be taken as at least 12 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to 12 kPa. This pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to culvert walls to prevent overstressing.

It is likely that bracing for the temporary support system will be required at a maximum interval of 5 m. For multiple support systems refer to *Canadian Foundation Engineering Manual* (CFEM) for apparent earth pressure distributions (CFEM, Section 26.10.3, Figure 26.8)

Table 2.6 Material types and earth pressure properties

Material	Unfactored Friction Angle φ'	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure at Rest (K_o)	Unit Weight γ kN/m^3)
Sand with Silt and Gravel Fill	36	0.26	3.85	0.41	21
Clayey Silt	28	0.36	2.77	0.53	19
Sand to Sand with Silt	31	0.32	3.12	0.48	21.5
Silt	29	0.35	2.88	0.51	19
Sandy Silt with Gravel	35	0.27	3.69	0.43	22
Silty Sand	33	0.29	3.39	0.45	21

2.5 Construction Alternatives

For the proposed culvert replacement the following methods were considered as possible alternatives for the new culvert installation at this site:

1. Full road closure followed by open cut/unsupported excavation to replace culvert
2. Construct temporary detour embankments at the site followed by open cut/unsupported excavation to expose and replace culvert
3. Half-and-half construction using roadway protection to allow excavation as maintaining signalized one lane of traffic on the existing embankment during construction. The following three options of excavation and replacement using the half-and-half approach were considered:
   - A. Construction using roadway protection and unsupported excavation of cut sides
   - B. Construction using roadway protection and braced cut sides
   - C. Construction using roadway protection and steel sheet pile abutments with precast concrete deck system

All methods considered utilize a cut and cover approach for culvert replacement which allow complete removal of the existing culverts, but it requires disruption of traffic. In contrast, a trenchless approach for culvert replacement does not require disruption of traffic. However, considering the size and nature of the existing culvert and topography of the surrounding terrain, tunneling for trenchless replacement of this culvert was not considered as an applicable option. The other trenchless methods such as pipe bursting, pipe splitting, pipe swallowing and interior replacement methods were also not considered as applicable in this project, since the size of the host pipe classify this culvert as an unsuitable candidate for these techniques. For all approaches provision must be made to maintain surface water flow to the outlet.

The following Table 2.7 summarizes advantages and disadvantages of considered construction alternatives. The table also shows assessed risk/consequences and relative costs of the considered methods. Schematic diagrams of considered alternatives are attached in Appendix H.

Table 2.7 Construction alternatives for culvert replacement (see schematic sketches in Appendix H)

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
OPTION 1 Full Road Closure using Existing Roadways and Open Cut Unsupported Excavation	• No construction of detour roads or roadway protection required • No excavation support required • Install entire new culvert at once • Straightforward construction • Short mobilization time • Low capital investment; cost saving in time and materials required for construction	• Traffic interruption • Long detour around site using other existing roads required • Large amount of soil to be excavated • Existing fills and native soils require 2H:1V side slopes to maintain stability • Erosion control of temporary cuts required • Need to temporarily control lake water • Potential claims to compensate vehicle occupants and local business for delays or time lost due to detour routes • Risk of cost overrun and inability to finish job: low	Relatively less expensive than other methods due to cost savings in time and materials required for construction, but potential claims to compensate vehicle occupants and local business for delays or time lost due to detour routes	2
OPTION 2 Temporary Local Detour and Open Cut Unsupported Excavation	• Traffic flow maintained at the site during construction • Simple detour roads can be constructed • Existing CSP culverts will be completely remove and replaced with new culvert • No excavation support required • Install entire new culvert at once	• Traffic interruption • Construction of detour embankments required at south or north side of highway • Difficulties to construct detours due to inaccessible surrounding terrain • Increased time for construction of detour • Large amount of soil to be excavated • Erosion control of temporary cuts required • Need to temporarily control lake water • Possible settlement due to new earth embankment fill • Risk of cost overrun and inability to finish job: low to moderate • Possible extra cost to purchase of private property	More expensive than full road closure due to high costs to build local detours	5

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
OPTION 3.A Half-and-half Construction with Unsupported Cut Sides	• Traffic flow maintained at the site during construction • Short mobilization time • Straight forward construction and construction procedures	• Traffic interruption • Roadway protection of up to 3.4 m high required to maintain one lane of traffic • High cost of roadway protection system • Large amount of soil to be excavated • Need to decommission the shoring system • Need to temporarily control lake water • Risk of cost overrun and instability to finish job: low to moderate	Relatively more expensive than full road closure due to high costs of roadway protection system	3
OPTION 3.B Half-and- half Construction with Braced or Anchored Cut Sides	• One or possibly two lanes of traffic flow maintained on existing road (e.g. steel decking, but costly) • Global stability of excavation enhanced by narrow geometry • Less traffic interruption than with unsupported cut sides approach • Temporary decking could be usable over braced cut to allow for excavation of both halves prior to diverting stream and backfilling • Cost savings due to limited excavation and backfill	• Traffic interruption • Roadway protection of up to 3.4 m high required to maintain one lane of traffic if steel decking is not possible • High cost of roadway protection system and/or decking • Require side shoring and bracing • Bracing (e.g. struts) may interfere with excavation • Excavation of material and placement of bracing required in limited space • Need to decommission the shoring system • Need to temporarily control lake water • Risk of cost overrun and instability to finish job: low to moderate	More expensive than full road closure and other open cut sides approach due to high costs for shoring system and temporary decking (if feasible) to maintain continuous flow of traffic	4

Installation Method	Advantages	Disadvantages	Relative Cost	Ranking
OPTION 3.C Half-and-half Construction with Installation of Steel Sheet Pile Abutments with Precast Concrete Deck	• Environmentally friendly • Easy to construct • No need for dewatering and cofferdam • No need for detour • No need to redirect existing creek water • No need for decommissioning of shoring system • Cost effective	• Traffic interruption • Roadway protection required to maintain one lane of traffic • High cost of roadway protection system • Relatively new approach for MTO • Due to possible lateral movement need an anchor system, bracing or deadman • Durability issue with sheet pile walls •Some difficulty in excavating under concrete span •Difficulties for sheet pile driving due to presence of cobbles and boulders •Risk of cost overrun and inability to finish job: low to moderate	Relatively more expensive than full road closer due to high costs of shoring abutments, but more practical	1

Based on the above list of advantages and disadvantages of the possible construction methods, we recommend the following ranking of the considered options:

1. OPTION 3.C: Half-and-half construction with the steel sheet pile abutments and precast concrete deck (Figure H3.C, Appendix H)

2. OPTION 1: Full road closure using existing roadways and open cut unsupported excavation (Figure H1, Appendix H)

3. OPTION 3.A: Half-and-half construction with unsupported cut sides (Figure H3.A, Appendix H)

4. OPTION 3.B: Half-and-half construction with braced or anchored cut sides (Figure H3.B, Appendix H)

5. OPTION 2: Temporary local detour and open cut unsupported excavation (Figure H2, Appendix H)

The following sections discuss these options in more details.

2.5.1 **Detour Options (Options 1 and 2)**

Both detour options, the option with full closure of Hwy 642 and long detours around the area using existing roadways (see Figure H1, Appendix H), and the option with the local detour embankment construction at the site to maintain the local flow of traffic during the replacement (see Figure H2, Appendix H), allow for open cut, unsupported excavation to facilitate the replacement of the existing culverts. The major advantage is that neither excavation support nor roadway protection is required with these options. The major disadvantages of both options are traffic interruption, large amounts of excavated soils and need for temporary construction unwatering and dewatering systems (i.e. cofferdams, and sumps and pumps, etc.) to prevent lake water and groundwater flow into the construction area which is the responsibility of the contractor.

All excavations at this site must be conducted in accordance with the Occupational Health and Safety Act (OHSA) and Regulations for Construction (O. Reg. 213/91). All fills (i.e. sand with silt and gravel fill) may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The native soils below the groundwater table may be classified as a Type 4 soil. To avoid disturbance of the founding subgrade and to allow placement of backfill in dry conditions, groundwater must be controlled to below the proposed invert excavation levels prior to digging to final levels. As mentioned before, the ingress of surface water must be controlled using a suitable system as well.

Temporary excavation side slopes for Type 3 soil should not exceed 1H:1V in accordance with OHSA, and 2H:1V is recommended for global stability of these deep cuts (i.e. to maintain a global factor of safety greater than 1.3) where excavation will be left open for some time. Temporary excavation side slopes for Type 4 soils should not exceed 3H:1V where applicable. There is a potential for sloughing to occur if the trench remains open for an extended period of time (i.e. > 24 hours) or during a rainfall event. In addition, some localized surficial sloughing may be experienced in areas of perched groundwater seepage (i.e. within the embankment fill).

The detour construction alternative would involve construction of a temporary on-site embankment at the one side of the existing embankment depending on the available space and suitable terrain. Compacted engineered fill for construction of the temporary detour road is recommended. Prior to construction of the temporary detour embankment, the site will need to be cleared and grubbed of any existing bushes and vegetation. All surficial topsoil (if exists), organics and softened or loosened soil should be stripped form below the proposed temporary detour road embankment. All subgrade soils should be proof-rolled prior to fill placement and embankment fill should be placed in accordance with OPSS. PROV 206 (dated November 2014).

2.5.2 **Half-and-Half Construction (Options 3)**

If a long detour using existing roadways is not available and acceptable, the half-and-half construction method should be utilized (see Figures H.3.A, H.3.B and H.3.C, Appendix H). In that method one lane of the existing highway will be used to maintain the local traffic while the other half of the existing highway will be excavated and the half of the existing culvert will be exposed. Then that portion of the existing culvert will be removed and replaced with a new culvert (or culverts), followed by rebuilding of that half of the embankment to grade. Upon completion of the new embankment, the traffic will be moved onto the new fill and the process will be repeated to complete the construction and culvert replacement.

The temporary excavation required to remove half of the existing embankment would be up to 3.4 m deep. Therefore, temporary shoring such as a soldier pile and lagging or sheet pile system will be required as a roadway protection system to allow staging excavation/construction. It will be the Contractors responsibility to design a suitable temporary support system for the MTO review prior to installation. The Contractor is to follow OPSS 538 and SP No. 902S01, regarding excavations for structures, and OPSS 539 and SP No. 105S19, regarding temporary protection systems. Recommendations for a temporary roadway protection are given in Section 2.6.Using the half-and-half construction approach, several methods of culvert replacement were considered as discussed below:

A. Construction using roadway protection and unsupported excavation of cut sides
B. Construction using roadway protection and braced or anchored cut sides
C. Construction using roadway protection and a steel sheet pile abutments with precast concrete deck system

Option 3.A could be more economical due to possible cost savings for reversible wall configuration, but it will be more disruptive to the highway embankment. Option 3.B will disrupt less of the embankment but would cost more, i.e. about 1.8 times of Option 3.A. Excavation and backfilling operations will also be more challenging with Option 3.B. Both options require decommissioning of shoring system upon completion of the work.

2.5.2.1 Option 3.A: Half-and-Half Construction with Unsupported Cut Sides

This method provides roadway protection parallel to the highway between two lanes, and allows to divert traffic to the one side and undertake open cut with sloping sides at the other side (see Figure H3.A, Appendix H). The roadway protection can take the form of reversible shoring such as a solder pile and lagging or sheet pile with rakers or anchors for horizontal support. Where the cut extends below prevailing groundwater a suitable control/system is required. Once one lane is completed the supports can be reversed and the other lane constructed in similar fashion. The shoring system would likely be decommissioned in place. Temporary surface water flow control must be developed by contractor.

Option 3.A could be more economical due to possible cost savings for reversible wall configuration, but it will be more disruptive to the highway embankment than Options 3.B and 3.C since it needs to excavate a large amount of soil.

2.5.2.2 Option 3.B: Half-and-Half Construction with Braced or Anchored Cut Sides

This method provides braced or anchored cut shoring system perpendicular to the highway for face protection and to allow culvert construction (see Figure H3.B., Appendix H). Excavation in this case would have to accommodate the necessary cross-bracing such as struts. With this option, consideration would have been given to how the new culvert sections will be installed given the relatively narrow work area and potential for obstructions form the lateral bracing using struts. Installation of tiebacks could be the solution. Temporary decking could possibly be used over the supported cut to allow for excavation of both halves prior to diverting stream and backfilling. However decking would be costly. As well as Option 3.A, decommissioning of the shoring system and temporary surface water flow control must be performed/developed by contractor.

Option 3.B will disrupt less of the embankment than Option 3.A but would cost more, i.e. about 1.8 times of Option 3.A, due to the cost of shoring system. Excavation and backfilling operations will also be more challenging with Option 3.B. Both options require decommissioning of shoring system upon completion of the work.

2.5.2.3 Option 3.C: Half-end-Half Construction using Steel Sheet Pile Abutments with Precast Concrete Deck

This option provides shoring system consisting of sheet piles perpendicular to the highway, which will serve the dual purpose of retaining backfill soil during excavation and being bearing elements to support culvert foundations after excavation (see Figure H3.C, Appendix H). As shown on Figure H3.C, the sheet piles will be installed perpendicularly in the half of the embankment at both sides of the existing culvert after installation of the roadway protection system for Stage 1 construction. Next the fill will be excavated to the designed elevation of the deck and its precast panels will be installed over the existing culverts. Than the fill below the deck panels will be excavated within construction limits for Stage 1 allowing the existing culverts to be removed. The excavation above the deck will be backfilled with a free-draining granular material up to the highway grade. The same processes will be repeated in Stage 2 construction, on the other side of the roadway protection.

The contractor should be responsible for the complete design, construction and monitoring of the described system. It is their responsibility to provide the work and design that should accommodate all relevant conditions including local and global stability for all stages of installation, including any necessary groundwater or surface water controls.

A major benefit of this method is that sheet piles will be permanently installed creating abutments for the precast concrete deck, so decommissioning of the shoring system is eliminated. Further, this method allows the use of the existing culvert to convey the lake water flow below Hwy 642 even during the construction work. However, due to the depth of the fill, some lateral movement of the abutments might be possible and installation of anchor systems, bracing or deadman might be required.

2.6 Temporary Roadway Protection

Temporary roadway protection is anticipated to be a part of the half-and-half construction approach that will be required to maintain on-site traffic during the construction. It is recommended that roadway protection system be in accordance with MTO Special Provision 105S19. The complete design, construction, monitoring and removal of the installed protection system should be a responsibility of the contractor. Due to nature of this application it is expected that much of temporary shoring will be decommissioned in place noting the high cost for removal. Decommissioning must be consistent with good practice to avoid interference with highway systems and utilities, if any. The protection system should be designed to provide protection for excavations as required by the OHSA, at locations specified in the contract, and at any locations where the stability, safety or function of an existing structure and/or utility may be impaired by construction work.

The protection system should be designed for the Performance Level 2 (for small, less important sections). The minimum requirements for monitoring should include the survey measurements of 6 m apart scaled targets attached to the shoring wall at the elevations specified. If movement approaches the allowable limit of 25 mm (Performance level 2), suitable measures should be taken to ensure stability of the protection system and to ensure that the movement does not exceed the performance level specified.

2.7 Culvert Bedding

OPSDs 802.010, 802.031 and 802.032 which are included in Appendix G provide the bedding, embedment, cover and backfill standards for the different pipe material. According to these standards the culvert bedding should consist of Granular “A” (OPSS 1010) with thickness of 300 mm beneath the culvert and extend a minimum of 500 mm horizontally on either side of the culvert edge. The bedding material should be placed in layers not exceeding 200 mm in thickness, loose measurement, and compacted to at least 95% of the Standard Proctor Maximum Dry Density (SPMDD) before a subsequent layer is placed in accordance with OPSS 514. Bedding material placed in the haunches must be compacted prior to continued placement of cover material.

Bedding on each side of the pipe shall be completed simultaneously. At no time shall the levels on each side differ more than the 200 mm uncompacted layers.

Prior to placing any fill material, the exposed native subgrade should be inspected according to OPSS 902. A non-woven geotextile separator is to be placed between the approved subgrade and the compacted fill to assist in material placement and maintain the integrity of the founding soil along the entire length of the culvert. The geotextile separator is to be a Class II non-woven material with an equivalent opening size of 75-150 μm.

For the site area, a frost penetration depth of approximately 2.5 m can occur in open, unheated areas without snow cover. At the culvert inlet and outlet, and beneath the proposed culvert, the native soils consist of lean clay with sand. This material has high frost susceptibility based upon the MTO Frost Classification guideline of percent particles between 5 to 75 μm. Therefore, non-frost susceptible materials such as sand and gravel (Granular “A”) need to be provided to the limit of frost penetration beneath the inlet and outlet of the culvert. However, considering that cold air blowing through the culvert during the winter season will freeze soil next to the culvert, a minimum 500 mm thick layer of non-susceptible material should be considered to be placed as a bedding along the entire culvert length.

2.8 Culvert Backfill

Backfill should be placed from the base of the culvert to the full height of the culvert and extend a minimum 1.3 m horizontal distance from the outside wall (as per Figure C6.20a of the CHBDC). This horizontal distance may be reduced by the use of suitable insulation (such as a heavy duty STYROFOAM). The insulation should be placed against the outside wall of the culvert from the base of the culvert to its total height. The material should be installed as per the manufacturer’s instructions.

The backfill should consist of free-draining, non-frost susceptible granular materials such as Granular A or Granular B (OPSS.PROV 1010).

All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and each lift should be compacted to at least 95% of the material’s SPMDD (Standard Proctor Maximum Dry Density). The final lift of embankment fill prior to placing pavement sub-base should be compacted to 100 % SPMDD. The Granular A base and Granular B sub-base courses (for pavement) should be compacted to 100% of the material’s SPMDD.

The use of heavy compaction equipment should be avoided immediately adjacent and above the culvert, as per MTO practice. The minimum height of fill cover above the crown of the culvert before power operated tractors or rolling equipment shall be 900 mm, unless otherwise noted by the structural engineer. During backfill placement, the height of the backfill should be maintained at approximately same level on both sides of the structure, to avoid lateral displacement of the structure.

For fills immediately below any roadway, it is recommended that Granular A or B aggregates be used. Where necessary, proper tapering as per standards should be provided. Below a depth of about 1.5 m from any finished road grade, approved compactable fill, such as select subgrade materials (SSM) can be used.

Where less than 1.3 m of earth cover is provided above the top of the culvert, a frost taper should be included as per OPSD 3101.150.

Backfilling behind any retaining (wing) walls should consist of granular materials in accordance with the MTO standards. Free draining backfill materials and perforated drains (as per Figure C6.20a of the CHBDC), suitably outleted etc. should be provided in order to prevent hydrostatic pressure build-up.

2.9 Surface Water and Groundwater Control

Cofferdams will be required at both upstream and downstream ends to envelop the construction site and keep it free of water during culvert installation. The investigation revealed that the subsurface conditions at the location of cofferdam at inlet side consist of a layer of very soft to soft peat (~1.3 m thick) underlain by very soft to soft clayey silt (~2.8 m thick), very loose silt (~3.9 m thick) and very loose silty sand (~3.3 m thick). The practical refusal is encountered at 12.5 m below the ground surface. At the outlet side, the subsurface conditions at the location of cofferdam consist of a layer of very loose to compact silt (~2.9 m thick) underlain by compact to loose sendy silt with gravel (~2.9 m thick). The practical refusal is encountered at 6.7 m below the ground surface.

Based on these geotechnical conditions, suitably designed steel sheet pile walls can be used as cofferdams at this site. Sheet piles perpendicular to the highway at least 3 m into the embankment slopes should be considered to prevent water getting in through the sides. If a cantilever system is used, an embedded depth of sheet piles can be approximately 2.0 to 2.5 times of its exposed height. The proposed sheet pile wall should be at least one meter above 100 year flood. The required minimum section modulus and embedment pile length should be designed based on the recommended design parameters.

Alternatively, a rockfill cofferdam can be used. This cofferdam will have to be constructed to the same topographic constraints as the sheet pile cofferdam. The size of material suitable for use depends on the erosion potential, stream flow velocity, etc. The rockfill cofferdam should be designed with a more impervious water barrier at the outside face to create a more watertight enclosure. Schemes involving 2 inch minus crusher run with finer facing material upstream have been successfully used in similar settings. Any required permitting must be determined.

As mentioned, which cofferdam system is best suited depends on many technical and economic factors. The advantages and disadvantages of both cofferdam systems are summarized in Table 2.8. Given the soil conditions, topography of the surrounding terrain and available space, the use of a suitably designed steel sheet pile system is recommended for the inlets or outlets of these locations. The design of these cofferdams, which are temporary retaining structures is the responsibility of the Contractor. The cofferdam must be designed to withstand the anticipated design loads and to be watertight as practically possible. The Contractor is also responsible for cofferdam's materials, construction, monitoring and removal. Cofferdams should be designed in accordance with OPSS 539 by a licensed Professional Engineer experienced in shoring design. If sheet piles are employed, piling shall be according to OPSS 903.

Table 2.8 Comparison of cofferdam systems

Option	Advantages	Disadvantages	Relative Cost	Risk/ Consequence
Steel sheet piles	• Provides more watertight base • Structural elements and seals easier to positively construct • Increased safety with appropriate design • Easily removed • Less seepage • Reusable	• More costly • More likely time consuming for installation • May present issues for seepage and/or piping • Larger machines required • May require bracing	MEDIUM TO HIGH	• Possible piping problem • May take longer to install • Environmental permits
Rockfill	• Less costly • Relatively less time consuming for installation • Native material can be usable	• Require more space for installation • Less safe • Subjected to wave erosion • Less watertight • Prone to land shifts, slides and collapse • More likely time consuming to remove	LOW TO MEDIUM	• Less stable and safe. May generate 'mud waves' • May take longer to remove • May require to install clay cutoff • More dewatering • Environmental permits

The soils encountered below the groundwater table and within potential excavation depths consist of native sand with silt and clayey silt. These soils are susceptible to disturbance from groundwater and mobilized equipment. The groundwater level needs to be controlled to at least 0.5 m below the excavation level to avoid disturbance, and any surface or groundwater seepage should be removed from the excavation prior to the culvert bedding material placement of granular backfill in the dry. In general, pumping using properly filtered sumps, and/or filtered drains placed along the base of the excavation should provide sufficient groundwater control during foundation works.

Dewatering requirements behind the cofferdams to keep the construction site dry will be impacted by water levels in the lake at the time of construction activities. Dewatering shall be carried out in accordance with OPSS 517 and OPSS 518. It is responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, water levels and flow conditions for prior approval of the MTO. The method used should not undermine the existing road embankment or adjacent side slopes. In this connection the provision of toe protection at side slopes during drawdown may be required to minimize sloughing and undercutting during dewatering.

Erosion and sediment control during culvert construction should be as per the MTO Drainage Manual, Volume 2. Silt fences and other sediment control measures should be included to protect the downstream environment from the construction activities.

2.10 Embankment Design

2.10.1 Embankment Settlement

It is not planned to change the existing embankment grade at the culvert location. Therefore, there should be negligible additional settlements under the existing embankment because the soil under the existing embankment is mostly non-cohesive. However, a settlement of about 25 mm should be allowed for due to rebound during the construction.

2.10.2 Embankment Stability

A preliminary slope stability analysis was performed to assess the global stability of the existing embankment and to check that a minimum Factor of Safety of 1.3 will be achieved for the new embankment at the location of the proposed culvert/culverts. The static slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium. The SLOPE/W computer program developed by GeoSlope International was employed for computation.

Stability assessments of existing slopes under static conditions were performed on the cross-section perpendicular to the highway at the proposed culvert location. The cross-section of the existing embankment with the approximate slope of 1H:1V was established based on **exp**'s survey data and the topographic plan provided by MTO. The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report.

Based on the borehole information, the subsoils encountered at the work area consist of embankment fill, underlain by native sand to sand with silt, clayey silt and silt. Therefore, an effective stress analysis for a long term stability assessment of the embankment slope was performed taking into consideration the subsoil conditions encountered beneath the existing embankment.

Tabulated below in Table 2.9 are the soil parameters used for the slope stability analysis. The soil parameters were generally estimated based on the results of field and laboratory investigation.

Table 2.9 Soil properties used in slope stability analysis

Material	ϕ' (degrees)	c' (kPa)	γ' (kN/m^3)
Sand with Silt and Gravel Fill	36	0	21
Peat (Very Soft to Soft)	17	3	15
Clayey Silt (Very Soft to Soft)	28	0	19
Sand to Sand with Silt (Very Loose to Loose)	31	0	21.5
Silt (Very Loose)	29	0	19
Sandy Silt with Gravel (Loose to Compact)	35	0	22
Silty Sand (Very Loose)	33	0	21
Cobbles and Boulders	38	0	22

The results of slope stability analyses are attached as Appendix F. The results of stability analyses on the existing embankment slopes shown on Figure F1 and F2 suggest that the existing embankment could be on the verge of stability due to presence of the soft peat or loose sand layers below the embankment. To achieve the factor of safety greater than 1.3 for the global stability of the new embankment with 2H:1V slopes, the soft peat layer has to be excavated and replaced by engineered fill, as confirmed by the analyses which results are shown in Figures F3 and F4.

2.11 Inlet and Outlet

2.11.1 Erosion Protection at Outlet

The detailed design for erosion protection should be carried out by the hydraulic engineer. However in general, rip-rap protection should be provided where the culvert discharges into the open creek. The rip-rap should extend approximately 5 m beyond the ends of the culvert and line the embankment slope to the spring line of the culvert. The size of the rip-rap is a function of the creek's hydrology. As a rule of thumb the thickness of the rip-rap should be a minimum of twice the median particle size, and 300 mm thick as a minimum. The rip-rap configuration at the creek bed should generally follow the OPSD 810.010, which is included in Appendix G of this report. Rip-rap placed at 1V:1H will be stable.

Where the embankment side slopes have been scarred and/or excavated (beyond rip-rap limit) to facilitate the existing culvert replacement, the scarred and/or reinstated embankment side slopes are to be vegetated with sodding, seeding or planting as necessary depending on the flow rate and volume. Should seeding be utilized, a 100 mm thick layer of topsoil should be placed along with a degradable erosion blanket to help minimize erosion until the vegetation begins to grow.

2.11.2 **Stream Bed Rip-Rap**

The stream bed rip-rap thickness is to be at least twice the median particle size, and/or 300 mm thick as a minimum as outlined by OPSD 810.010 included in Appendix G of this report.

2.11.3 **Seepage Cut-off Requirements**

The seepage cut-off requirements should be reviewed in the following context. The native sand with silt soil at the inlet and outlet side and below the culvert bedding has a high potential for migration with high seepage gradients. For the culvert replacement and new culvert installation, it is prudent to examine possible methods to avoid piping of material resulting from seepage along the culvert. For culverts the following are typical methods: (i) clay seal, (ii) steel or wooden sheet pile cutoff at the upstream end of culvert, (iii) cut-off wall incorporated in the apron slab (if one is used) of the culvert, (iv) cut-off trench constructed with geotextile, and (v) rockfill at the upstream end of the culvert barrel to terminate below the granular bedding of the culvert. Only the clay seal and cut-off trench will be addressed since the sheet pile cut-off will require the understanding of the hydraulics of the stream.

2.11.3.1 Clay Seal

Where readily available a clay seal should be placed at the inlet of the proposed culvert, to prevent the migration of material along the face of the culvert, the formation of flow paths, and any potential internal erosion within the highway embankment (OPSD 802.095, Appendix G). OPSS. PROV 1205 specifies that material used for clay seals shell be natural clay, clay mixture (1 part Bentonite powder and 3.5 parts Granular “A”) or a geosynthetic clay liner (GCL). The coefficient of permeability shall not exceed 1 x 10^{-6} cm/s.

The following outlines the installation procedures and minimum material requirement of the clay seal:

- The clay seal should be placed along the sides and top of the culvert a minimum of 1.0 m along the side of the culvert and extending out laterally 1.0 m from the culvert.
- The clay seal should be placed from the top of the culvert footings and extend along the side and the top of the culvert. The clay must not be placed below the culvert.
- The clay should have a Liquid Limit greater than 40% and a Plasticity Index greater than 0.73 x (Liquid Limit – 20%).
- The clay seal is to be place in maximum 150 mm thick lifts and compacted to 95% SPMDD within 2% of the optimum moisture content.

If the GCL is used as a clay seal its material specifications containing the physical, mechanical and hydraulic properties shall be obtained from the manufacture. It is estimated that an approximately 12 mm thick GCL should be installed a minimum 1.0 m along the side of the culvert.

2.11.3.2 Cut-Off Trench

A cut-off trench can be used at both the upstream and downstream ends of the culvert and can be incorporated when the rip-rap apron at both ends of the culvert are being installed. In general, a trench is dug across the stream alignment to well beyond the walls of the culvert and a geomembrane liner is laid on the side of the trench keyed into the culvert at the top and on the base of the trench. The trench is then backfilled with graded rip-rap.

2.12 Corrosion Protection

One (1) representative soils sample was submitted to a CALA Certified Laboratory for chemical corrosivity analysis. The samples were analyzed for chloride, sulphate, pH, electrical conductivity, resistivity, redox potential, and sulphide concentrations. The results of the corrosivity testing are summarized in section 1.6 of this report and detailed results included in Appendix E.

Based on the chemical analysis, the data in Table 1.2 indicates medium to high resistivity, which indicates a moderate to low potential for corrosion of buried metallic elements, particularly pipes and appurtenances. However, our experience in the area is that the soils are generally low to medium in resistivity, with associated higher corrosion potential. The maximum chloride content reported is <20 ppm (μg/g) i.e. <0.002% which indicates a low potential for additional corrosion. The pH level of BH302-S5 was 5.99 and indicates additional low potential for corrosion.

The water soluble sulphate content of the soils tested is <20 ppm (μg/g), i.e. <0.002% and being less than 0.10%, does not indicate the potential to corrode normal Portland cement concrete. These data also support our local experience.

2.13 Operational Constraints (OCs) and Non Standard Special Provisions (NSSPs)

In assembling contract documents, a number of OCs and NSSPs should be included to address some of the foundation/geotechnical issues that might be of concern during execution of the work. It is anticipated that the following list may apply based on current information:

(1) NSSP for mass concrete on bedrock.

(2) NSSP for sloping rock and cobble and rock piece obstructions.

(3) NSSP for dowelling.

(4) NSSP for condition surveys and monitoring during any blasting.

Appendix I presents draft of the suggested NSSPs.

These should be further assessed during planning and design development when actual approaches are more defined.

PART III CLOSURE

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

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

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

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

This Foundation Investigation and Design Report has been prepared by Ahileas Mitsopoulos, P.Eng., Nimesh Tamrakar, M.Eng, EIT., Demetri N. Georgiou, MASc. P.Eng., and Silvana Micic, Ph.D., P.Eng. It was reviewed by TaeChul Kim, P.Eng. and by Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact. The field investigation was supervised by Elwin Farkas.

Yours truly,

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PART IV LIMITATIONS AND USE OF REPORT

BASIS OF REPORT

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

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

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

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

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

RELIANCE ON INFORMATION PROVIDED

The evaluation and conclusions contained in the Report are based on conditions in evidence at the time of site inspections and information provided to exp by the Client and others. The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose as communicated by the Client. exp has relied in good faith upon such representations, information and instructions and accepts no responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of any misstatements, omissions, misrepresentation or fraudulent acts of persons providing information. Unless specifically stated otherwise, the applicability and reliability of the findings, recommendations, suggestions or opinions expressed in the Report are only valid to the extent that there has been no material alteration to or variation from any of the information provided to exp.

STANDARD OF CARE

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

COMPLETE REPORT

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

USE OF REPORT

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

REPORT FORMAT

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

Appendix A –
Site Photographs

Photo 1. Existing culvert inlet on north side of highway

Photo 2. Existing culvert outlet on south side of highway

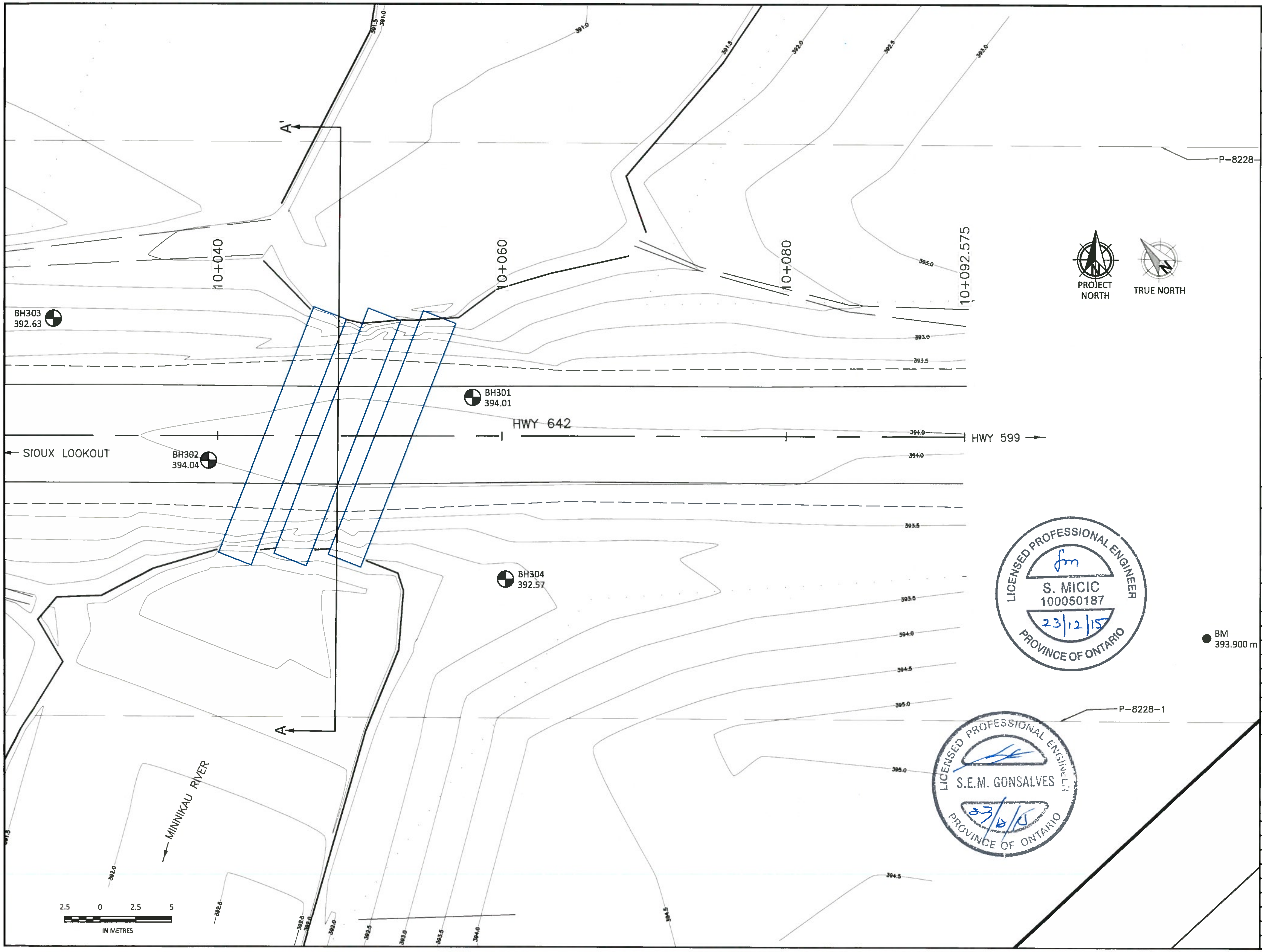
Photo 3. Facing west on Highway 642 before the existing culvert

Photo 4. Facing east on Highway 642 before the existing culvert

Photo 5. Embankment slope on north side facing west

Photo 6. Embankment slope on south side facing west

Appendix B – Drawings



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23/12/15

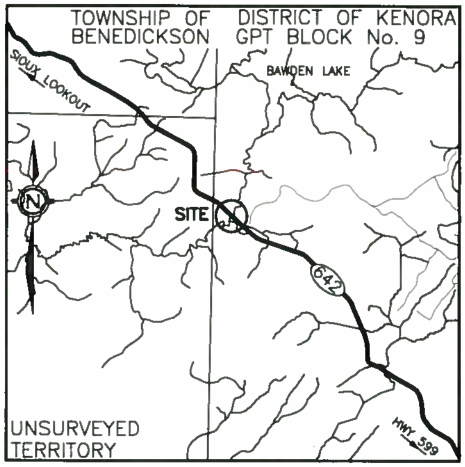
Agreement No. 6014-E-0017
Assignment No. 7
GWP 6912-12-01

MINNIKAU RIVER CULVERTS
(Highway 642, District of Kenora, ON)
PLAN

DWG
1

exp Services Inc.

KEY PLAN



LEGEND

⊕	BH301 394.01	BOREHOLE LOCATION GROUND SURFACE ELEVATION IN METRES
●	BM 393.900 m	BENCHMARK LOCATION GEODETIC ELEVATION IN METRES

BH No.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTH	EAST
BH301	394.01	5,536,130	404,875
BH302	394.04	5,536,137	404,860
BH303	392.63	5,536,148	404,867
BH304	392.57	5,536,118	404,873

NOTES

1. ALL DIMENSIONS ARE IN METRES.
2. BASE MAP PROVIDED BY CLIENT.
3. MTM COORDINATES BASE ON MTM ZONE ON-16 PROJECTION, AS PER PROVIDED FIGURE.
4. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. THE PROPOSED STRUCTURE DETAILS/WORKS ARE SHOWN FOR ILLUSTRATION PURPOSES ONLY.

REVISIONS

DATE	BY	DESCRIPTION

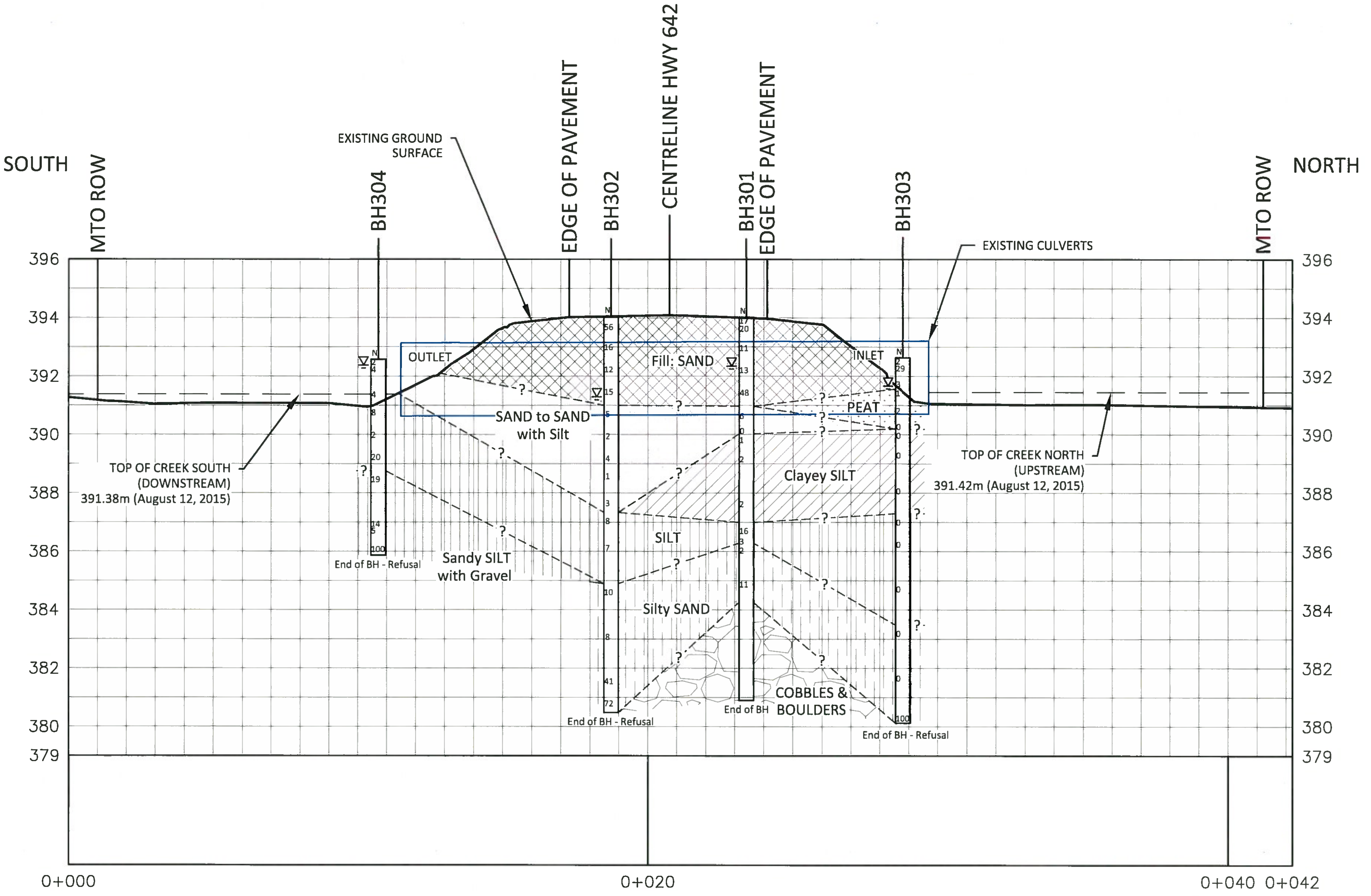
GEOCRES No. 52G-014
Project No. ADM-00223648-F0
Date: December 11, 2015
Scale : 1:250
Drawn By: RM
Checked By: AM
Checked By: DG

Agreement No. 6014-E-0017
Assignment No. 7
GWP 6912-12-01

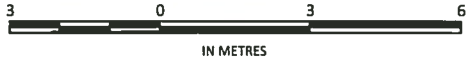
MINNIKAU RIVER CULVERTS
(Highway 642, District of Kenora, ON)
Section A-A'

DWG
2

exp Services Inc.



A - A'
PROFILE OF MINNIKAU RIVER CULVERTS



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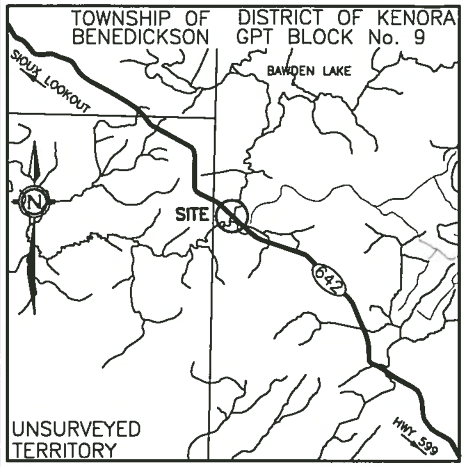
KEY PLAN



LEGEND

N STANDARD PENETRATION TEST (BLOWS/0.3 m)

∇ MEASURED WATER LEVEL IN OPEN BOREHOLE

BH No.	APPROX. ELEV. (m)	MTM COORDINATES	
		NORTH	EAST
BH301	394.01	5,536,130	404,875
BH302	394.04	5,536,137	404,860
BH303	392.63	5,536,148	404,867
BH304	392.57	5,536,118	404,873

NOTES

1. ALL DIMENSIONS ARE IN METRES.
2. BASE MAP PROVIDED BY CLIENT.
3. MTM COORDINATES BASE ON MTM ZONE ON-16 PROJECTION, AS PER PROVIDED FIGURE.
4. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. THE PROPOSED STRUCTURE DETAILS/WORKS ARE SHOWN FOR ILLUSTRATION PURPOSES ONLY.
5. RIVER WATER LEVEL IS THE HIGHEST VALUE MEASURED FOR BOTH UP AND DOWNSTREAM ELEVATIONS

REVISIONS

DATE	BY	DESCRIPTION

GEOCRES No. 52G-014	Project No. ADM-00223648-F0
Date: December 11, 2015	Horizontal Scale : 1:150
Drawn By: RM	Vertical Scale : 1:150
Checked By: AM	Checked By: DG

Appendix C –
Borehole Logs and Bedrock Core Photos

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

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

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

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

Terminology describing soil structure:

Desiccated:	having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Stratified:	alternating layers of varying material or color with the layers greater than 6 mm thick.
Laminated:	alternating layers of varying material or color with the layers less than 6 mm thick.
Fissured:	material breaks along plane of fracture.
Varved:	composed of regular alternating layers of silt and clay.
Slickensided:	fracture planes appear polished or glossy, sometimes striated.
Blocky:	cohesive soil that can be broken down into small angular lumps which resist further breakdown.

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

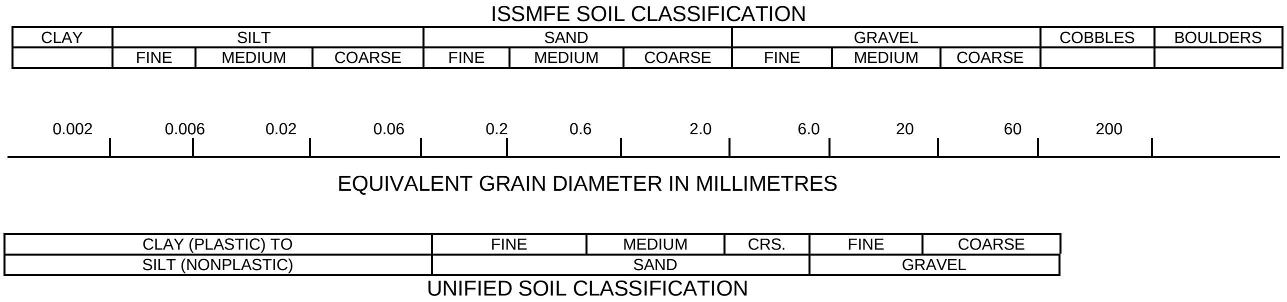
Seam: a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

Homogeneous: same color and appearance throughout.

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

Uniformly Graded: predominantly on grain size.

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



Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present and as described below in accordance with Note 16 in ASTM D2488-09a:

Table a: Percent or Proportion of Soil, Pp

	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5≤Pp≤10%
Little	15≤Pp≤25%
Some	30≤Pp≤45%
Mostly	50≤Pp≤100%

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

Table b: Apparent Density of Cohesionless Soil

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

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

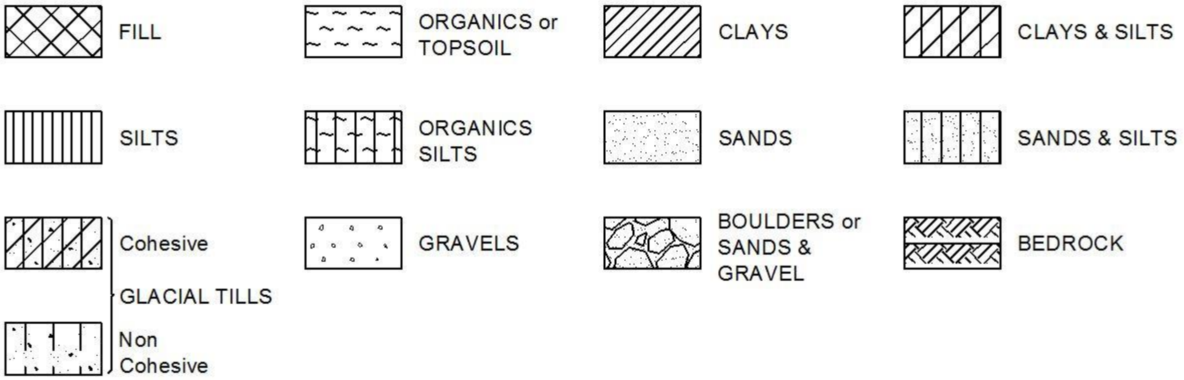
Table c: Consistency of Cohesive Soil

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

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

STRATA PLOT

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



WATER LEVEL MEASUREMENT

▽ Open Borehole or Test Pit

▼ Monitoring Well, Piezometer or Standpipe

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICALL PROPERIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

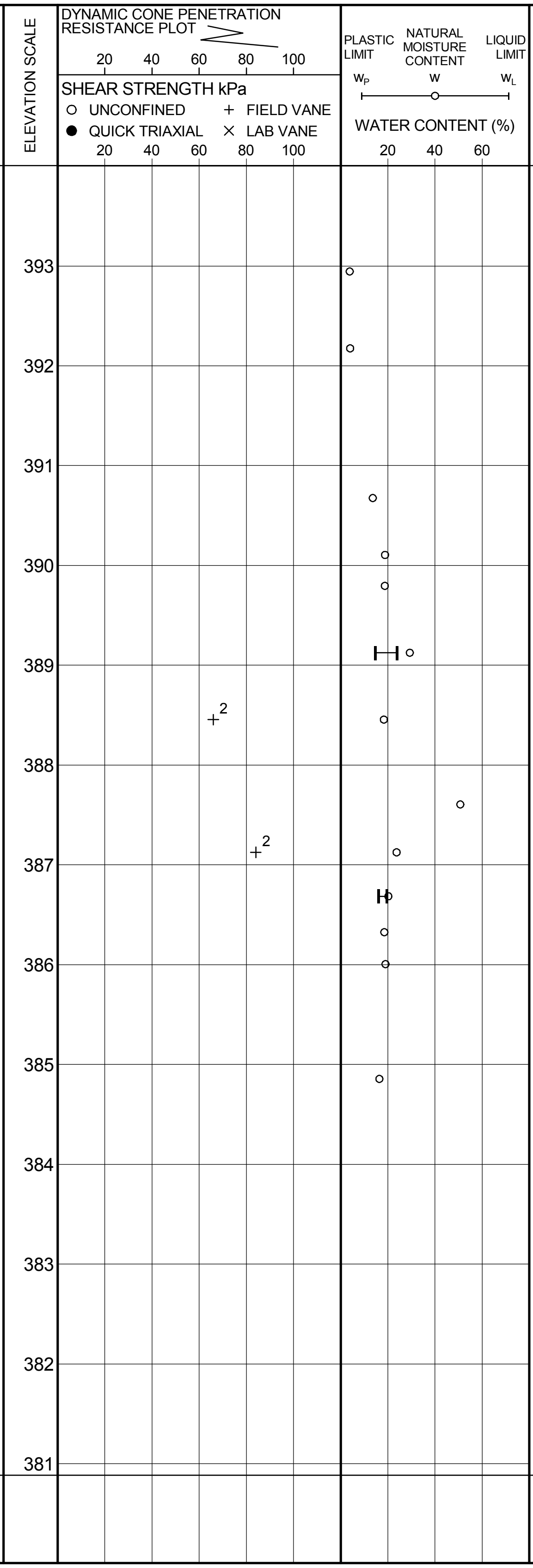
P_s	kg/m^3	Density of solid particles
γ_s	kN/m^3	Unit weight of solid particles
ρ_w	kg/m^3	Density of water
γ_w	kN/m^3	Unit weight of water
ρ	kg/m^3	Density of soil
γ	kN/m^3	Unit weight of soil
ρ_d	kg/m^3	Density of dry soil
γ_d	kN/m^3	Unit weight of dry soil
ρ_{sat}	kg/m^3	Density of saturated soil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

RECORD OF BOREHOLE No BH301

1 OF 1 **METRIC**

W.P. GWP No. 6912-12-01
LOCATION **Minnikau River Culverts (Site No. 41S-255/C) MTM ON-16 5,536,130N 404,875E**
ORIGINATED BY EF
DIST 61 HWY Hwy 642
BOREHOLE TYPE CME 850 Track Carrier / HSA / HQ
COMPILED BY AM/RM
DATUM **Geodetic**
DATE 8.10.15 - 8.10.15
CHECKED BY DG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
394.0	Asphalt									
394.0 0.0	**ASPHALT** - about 25 mm	S1A	SS	17						
	Poorly Graded SAND with Silt and Gravel (FILL) - compact to dense, brown, damp to moist, occasional cobbles in upper 0.8 m, crushed material in upper 0.1 m	S1B	SS	20						
		S2	SS	11			17	78	(5)	
		S3	SS	13	∇					
			SS	48			No recovery			
391.0 3.1	**Poorly Graded SAND with Silt** - loose to very loose, brown, wet, trace organics	S4	SS	6			13	81	(6)	
390.0 4.0	**Clayey SILT** - very soft to stiff, grey, wet, occasional 10 mm interbedded peat layers in upper 5.2 m	S5A	SS	0						
		S5B	SS	1						
		S6	SS	2			0	9	66	25
		S7	VANE				Field Vane = 66 kPa			
	- becoming varved at about 6.1 m depth	S8	SS	2						
		S9	VANE				Field Vane = 84 kPa			
387.0 7.0	**SILT** - very loose to compact, grey, wet	S10	SS	16			0	1	90	9
386.3 7.7	**Silty SAND** - very loose to compact, grey, wet	S11A	SS	3						
		S11B	SS	2			0	68	(32)	
	- about 610 mm of sand blow up at about 8.8 m depth	S12	SS	11						
384.2 9.8	**COBBLES and BOULDERS**	S13	CORE							
		S14	CORE							
380.9 13.1	**End of Borehole**									



+3, ×3: Numbers refer to Sensitivity

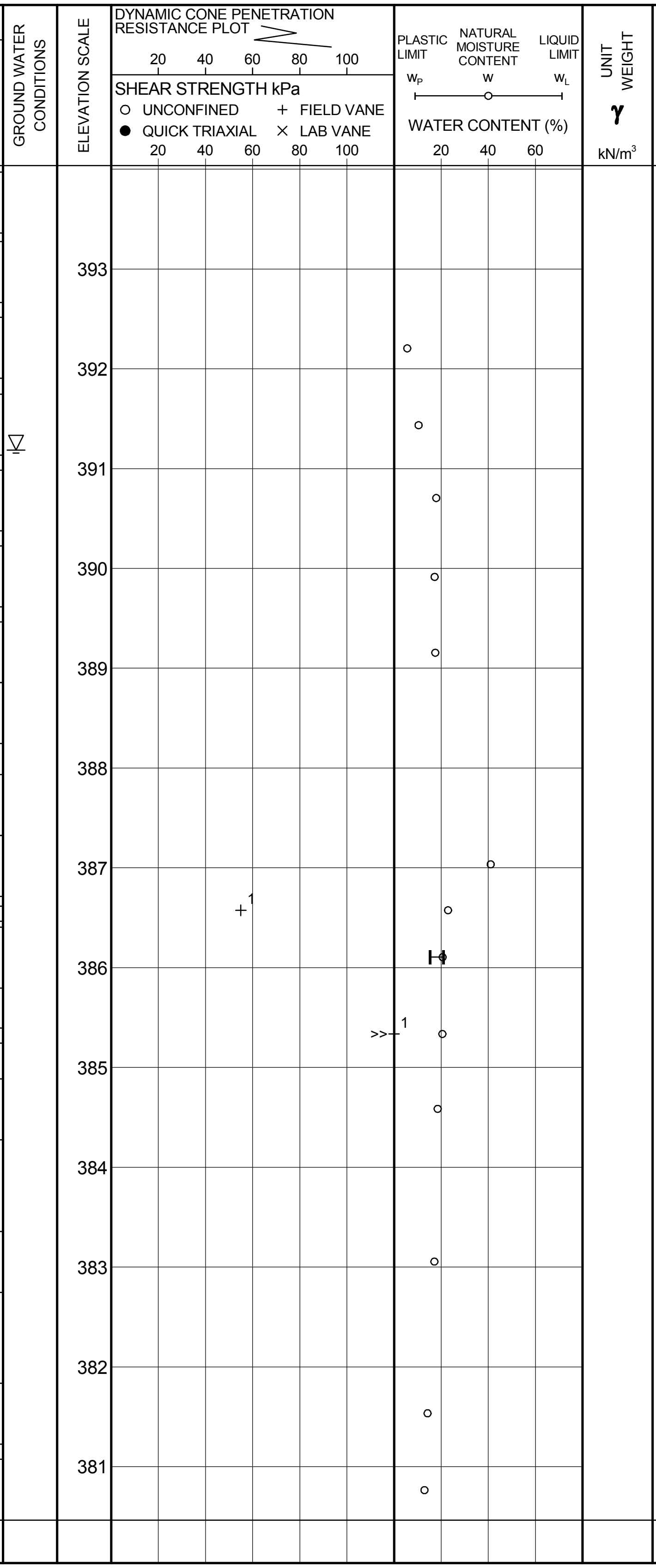
○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH302

1 OF 1 **METRIC**

W.P. GWP No. 6912-12-01
LOCATION **Minnikau River Culverts (Site No. 41S-255/C) MTM ON-16 5,536,137N 404,860E**
ORIGINATED BY EF
DIST 61 HWY Hwy 642
BOREHOLE TYPE CME 850 Track Carrier / HSA
COMPILED BY AM/RM
DATUM **Geodetic**
DATE 8.11.15 - 8.11.15
CHECKED BY DG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
394.0	Asphalt				
394.0 0.0	**ASPHALT** - about 25 mm				
	Poorly Graded SAND with Silt and Gravel (FILL) - very dense to compact, brown, damp to wet, occasional cobbles	S1	SS	56	
		S2	SS	16	
		S3	SS	12	
		S4	SS	15	
391.0 3.1	**Poorly Graded SAND** - very loose to loose, brown, wet	S5	SS	5	
	- about 250 mm sand blowup at about 3.8 m depth	S6	SS	2	0 96 (4)
		S7	SS	4	
	- about 1.0 m sand blowup at about 5.2 m depth		SS	1	No recovery
			SS	3	No recovery
387.3 6.7	**SILT** - loose, grey, wet	S8	SS	8	
		S9	VANE		Field Vane = 55 kPa
		S10	SS	7	0 1 88 11
		S11	VANE		Field Vane = 133 kPa
384.9 9.2	**Silty SAND** - loose to very dense, grey, wet	S12	SS	10	
	- about 760 mm sand blowup at about 10.7 m depth	S13	SS	8	0 60 39 1
	- about 1.2 m sand blowup at about 12.2 m depth	S14	SS	41	
	- occasional cobbles and boulders at about 12.8 m depth	S15	SS	72	
380.5 13.6	**End of Borehole** - refusal to auger				



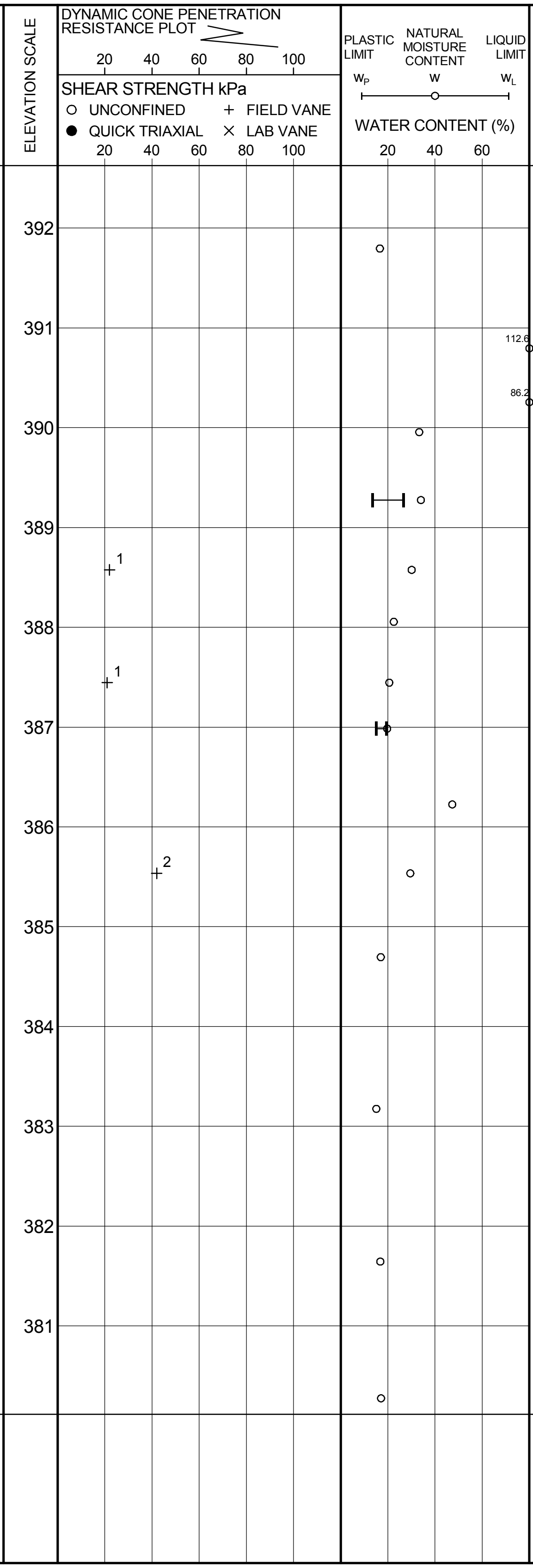
$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No BH303

1 OF 1 **METRIC**

W.P. GWP No. 6912-12-01
LOCATION **Minnikau River Culverts (Site No. 41S-255/C) MTM ON-16 5,536,148N 404,867E**
ORIGINATED BY EF
DIST 61 HWY Hwy 642
BOREHOLE TYPE CME 850 Track Carrier / HSA
COMPILED BY AM/RM
DATUM **Geodetic**
DATE 8.12.15 - 8.27.15
CHECKED BY DG



ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	GR	SA	SI	CL
392.6	Rootmat										
392.5 0.1	**ROOTMAT** - very soft, brown, moist, trace gravel	S1A	SS	2							
	Poorly Graded SAND with Silt and Gravel (FILL) - compact to very loose, brown, moist to wet, occasional cobbles and boulders, trace peat	S1B	SS	29							
		S2A	SS	3	▽						
391.6 1.1	**PEAT** - very soft to soft, dark brown, wet, trace sand, trace silt	S2B	SS	1							
		S3	SS	2							
		S4A	SS	0							
390.2 2.4	**Clayey SILT** - very soft to soft, grey, wet	S4B	SS	0							
		S5	SS	0				0	6	64	30
		S6	VANE				Field Vane = 22 kPa				
		S7	SS	0							
		S8	VANE				Field Vane = 21 kPa				
387.3 5.3	**SILT** - very loose, grey, wet	S9	SS	0				0	4	84	12
	- becoming varved from about 6.1 m to 7.0 m depth	S10	SS	0							
		S11	VANE				Field Vane = 42 kPa				
		S12	SS	0				0	11	(89)	
383.5 9.2	**Silty SAND** - very loose, grey, wet, fine grained	S13	SS	0							
	- about 2.4 m sand blowup at about 10.7 m depth	S14	SS	0				0	55	(45)	
	- about 300 mm sand blowup at about 12.1 m depth	S15	SS	100							
380.1 12.5	- becoming very dense at about 12.2 m depth **End of Borehole** - refusal to auger and SPT										

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

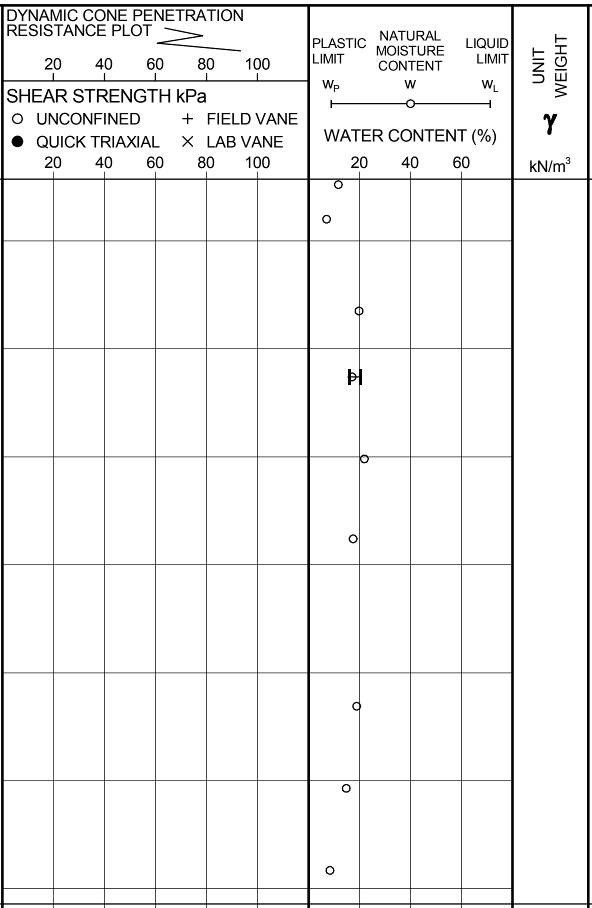
RECORD OF BOREHOLE No BH304

1 OF 1

METRIC

W.P. GWP No. 6912-12-01
LOCATION **Minnikau River Culverts (Site No. 41S-255/C) MTM ON-16 5,536,118N 404,873E**
ORIGINATED BY EF
DIST 61
HWY Hwy 642
BOREHOLE TYPE CME 850 Track Carrier / HSA
COMPILED BY AM/RM
DATUM **Geodetic**
DATE 8.11.15 - 8.12.15
CHECKED BY DG

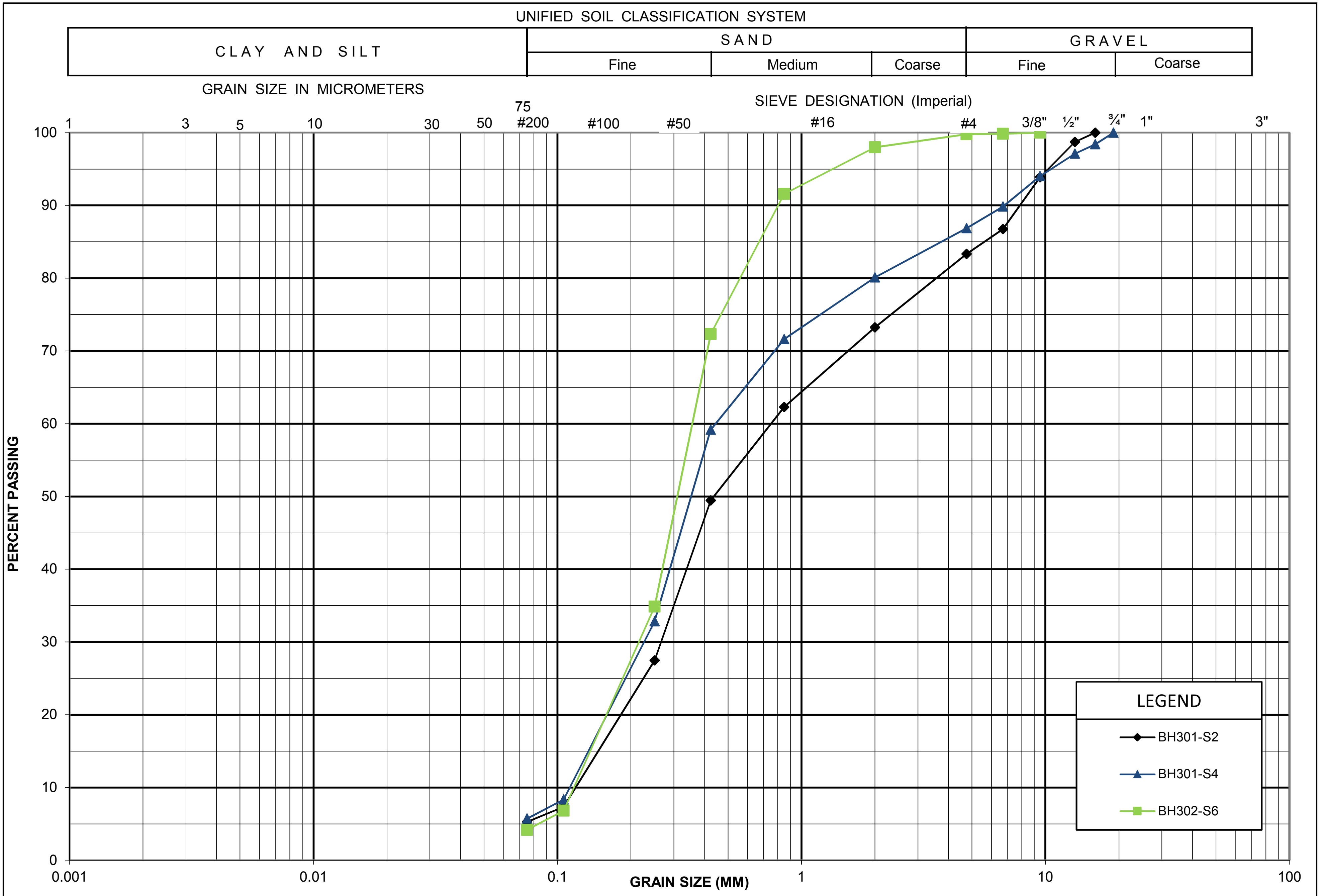
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
392.6	Peat									
392.6 0.0	**PEAT** - soft, brown, moist to wet, some sand to sandy, trace silt	S1A	SS	2						
392.5 0.1	**Poorly Graded SAND with Silt** - very loose to loose, brown, moist, some roots and rootlets	S1B	SS	4	∇	392				
391.7 0.9	**SILT** - very loose to compact, grey, moist to wet	S2	SS	4		391				
		S3	SS	8			0	4	84	12
		S4	SS	2		390				
		S5	SS	20		389				
388.8 3.8	- about 610 mm blowup at about 3.7 m depth **Sandy SILT with Gravel** - compact to loose, brown to grey at depth, wet		SS	19		388	No recovery			
		S6	SS	14						
	- occasional cobbles at about 5.2 m depth	S7	SS	5		387	15	19	59	7
	- becoming very dense at about 6.1 m depth	S8	SS	100		386				
385.9 6.7	**End of Borehole** - refusal to auger and SPT									



$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

Appendix D – Laboratory Data



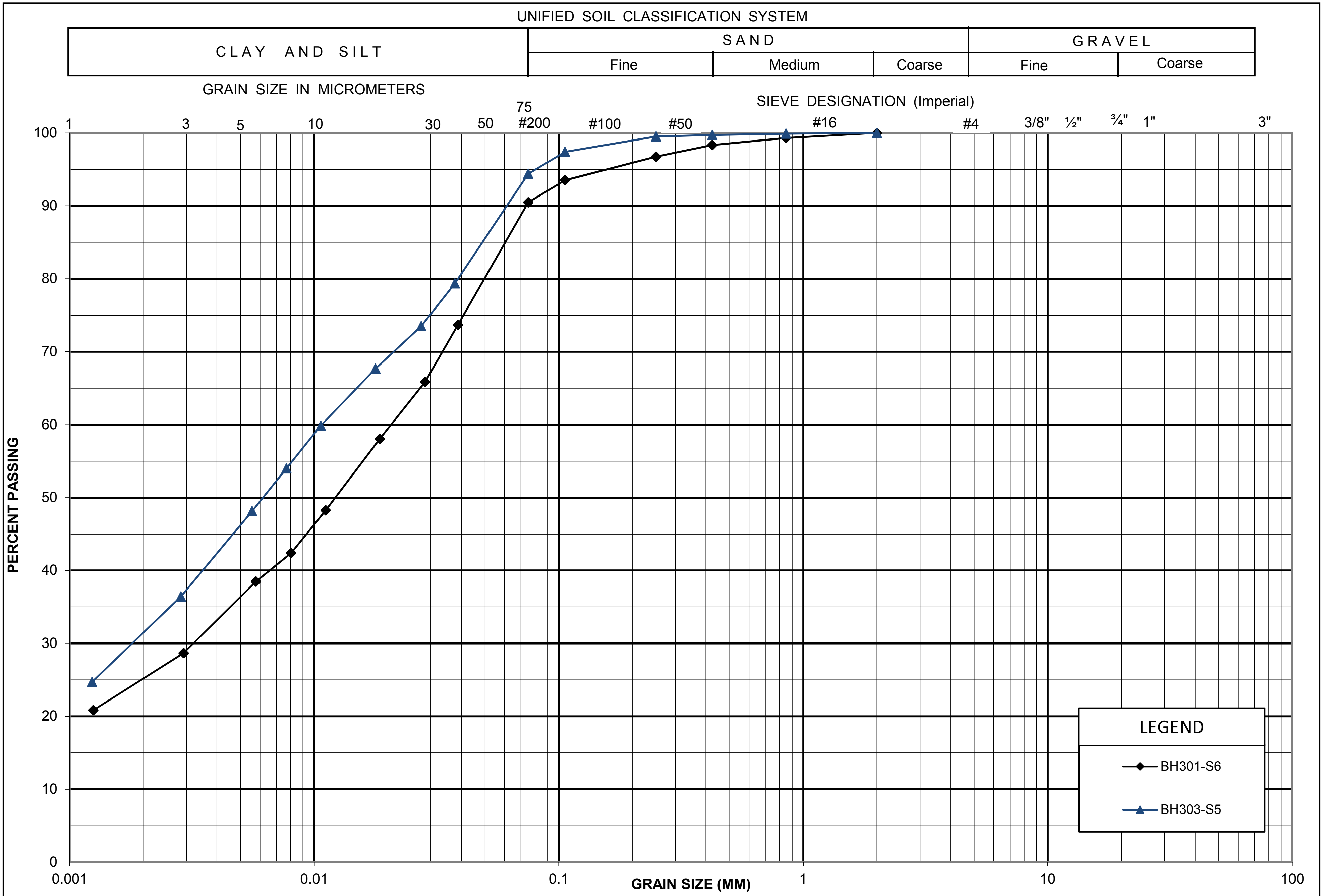
GRAIN SIZE DISTRIBUTION

SAND

FIGURE: No. 1

GWP: 6912-12-01

DATE : November 17, 2015



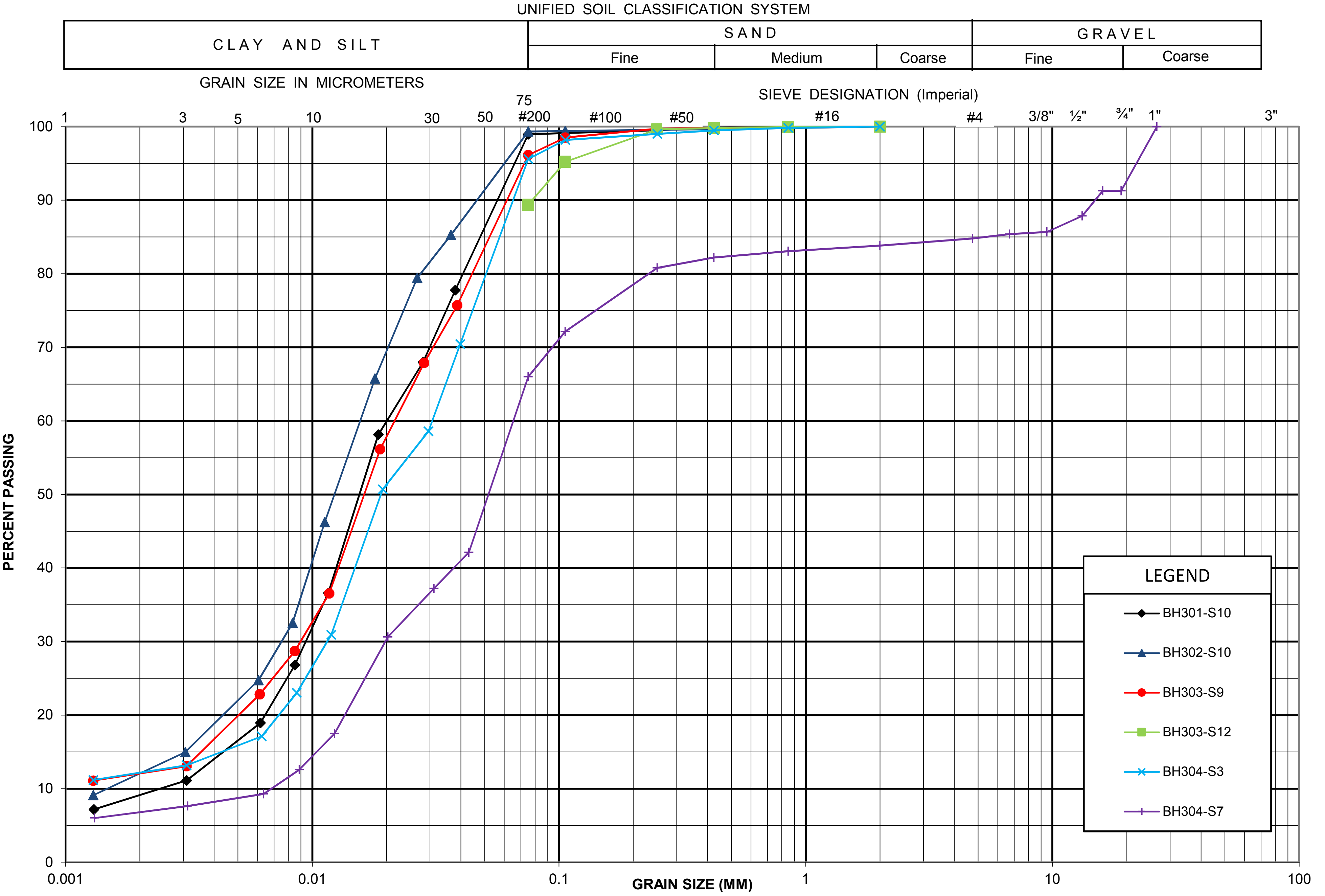
GRAIN SIZE DISTRIBUTION

Clayey SILT

FIGURE: No. 2

GWP: 6912-12-01

DATE : November 17, 2015



exp.

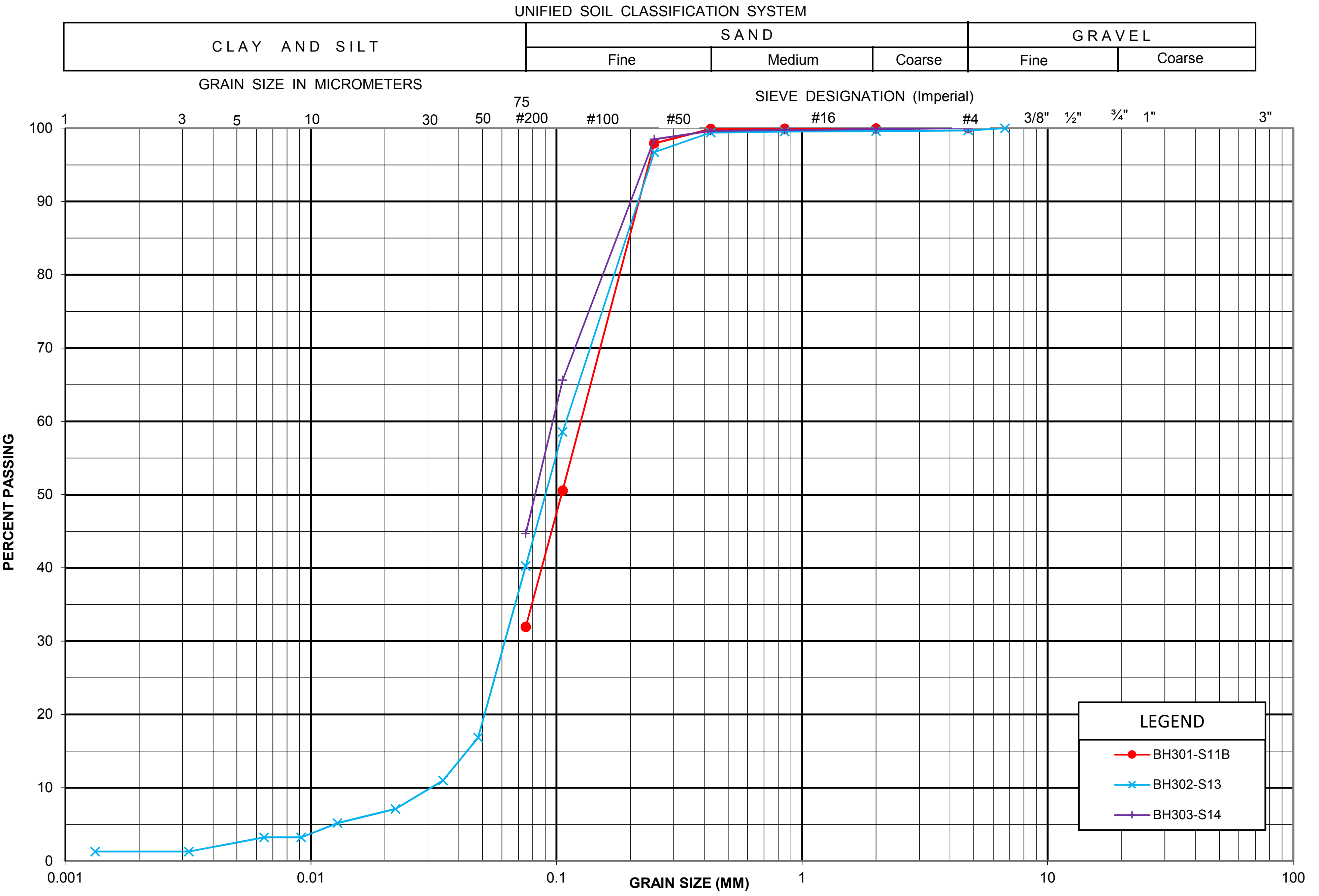
GRAIN SIZE DISTRIBUTION

SILT

FIGURE: No. 3

GWP: 6912-12-01

DATE : November 17, 2015



GRAIN SIZE DISTRIBUTION

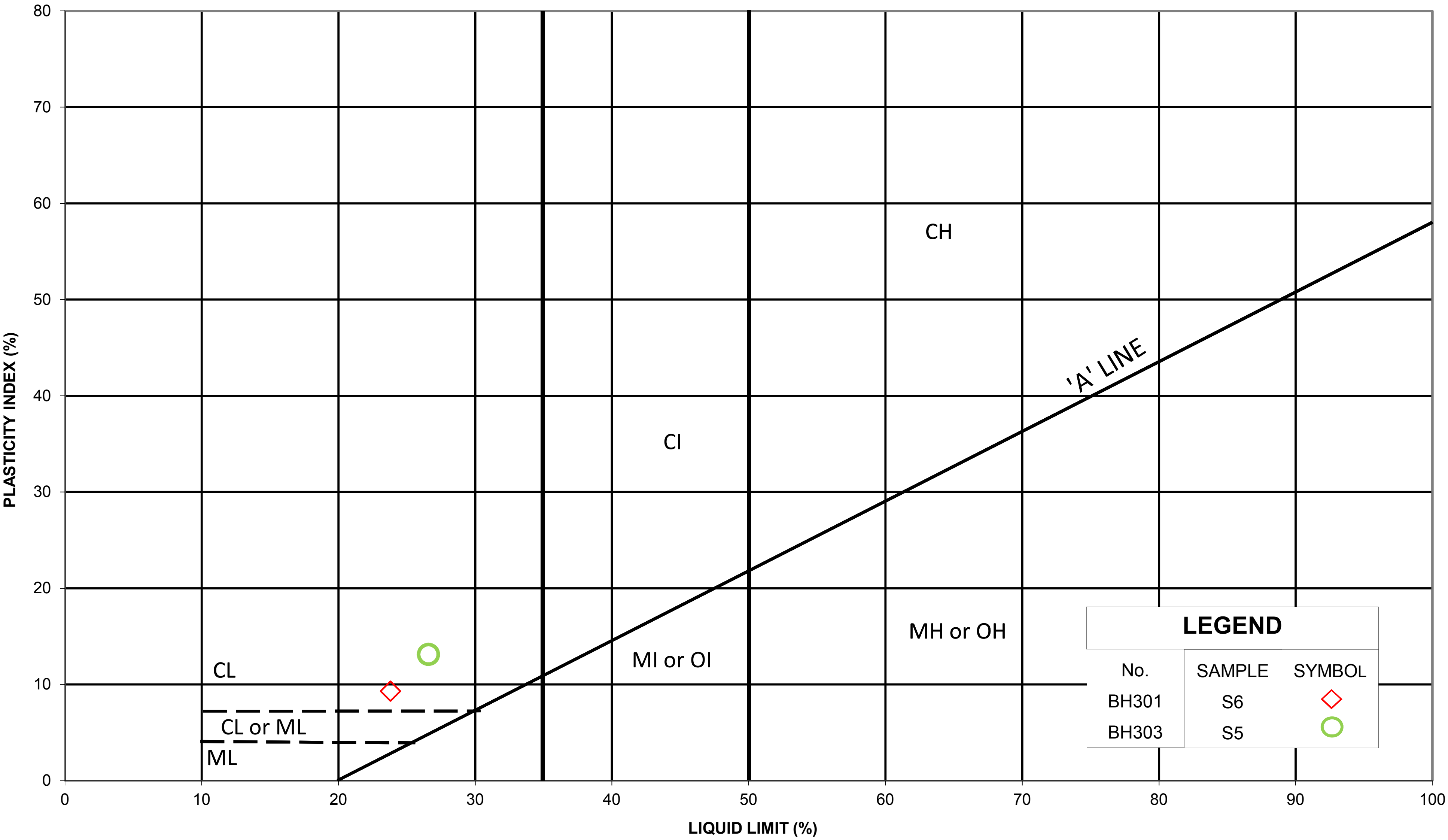
Silty SAND

FIGURE: No. 4

GWP: 6912-12-01

DATE : November 17, 2015

Minnikau River Culvert (Site No. 41S-255/C)
GWP No. 6364-14-01, Highway 642, District of Kenora, Ontario



exp.

PLASTICITY CHART

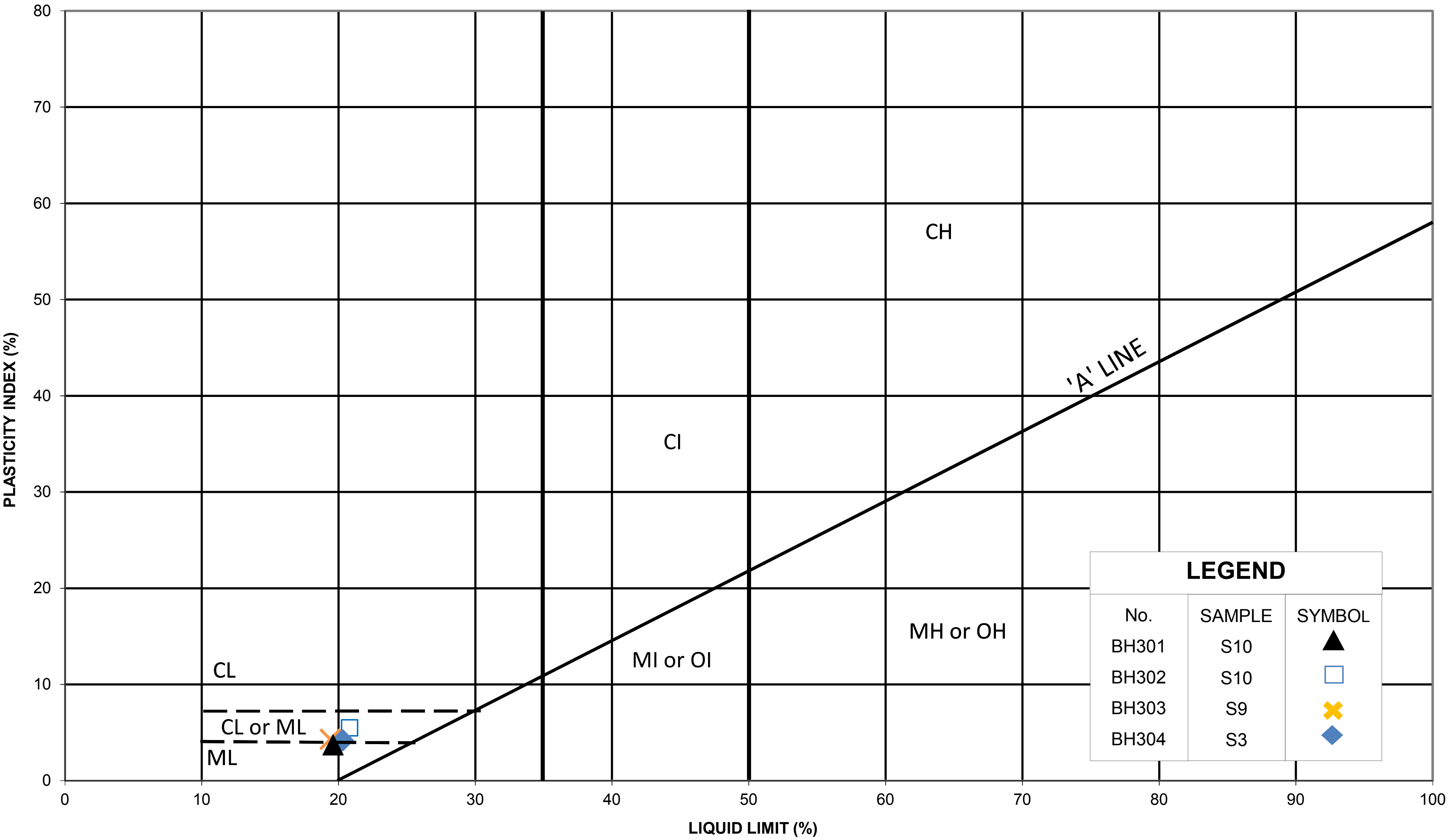
Clayey SILT

FIGURE No. 5

ADM-00223648-F0

November 5, 2015

Minnikau River Culvert (Site No. 41S-255/C)
GWP No. 6912-12-01, Highway 642, District of Kenora, Ontario



PLASTICITY CHART

SILT

FIGURE No. 6

ADM-00223648-F0

November 17, 2015

Appendix E – Chemical Analyses

Your Project #: ADM-00223648-F0
Site Location: SIOUX LOOKOUT, ONTARIO
Your C.O.C. #: NA

Attention:Ahileas Mitsopoulos/Michael S

exp Services Inc
Thunder Bay Branch
1142 Roland St
Thunder Bay, ON
P7B 5M4

Report Date: 2015/09/15
Report #: R3661790
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B5I2028
Received: 2015/09/09, 09:30

Sample Matrix: Soil
Samples Received: 6

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	6	N/A	2015/09/15	CAM SOP-00463	EPA 325.2 m
Conductivity	6	N/A	2015/09/14	CAM SOP-00414	OMOE E3138 v2 m
pH CaCl2 EXTRACT	6	2015/09/14	2015/09/14	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	6	2015/09/11	2015/09/14	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	6	N/A	2015/09/14	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics has performed all analytical testing herein in accordance with ISO 17025 and the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act. All methodologies comply with this document and are validated for use in the laboratory. The methods and techniques employed in this analysis conform to the performance criteria (detection limits, accuracy and precision) as outlined in the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act.

Maxxam Analytics is accredited for all specific parameters as required by Ontario Regulation 153/04. Maxxam Analytics is limited in liability to the actual cost of analysis unless otherwise agreed in writing. There is no other warranty expressed or implied. Samples will be retained at Maxxam Analytics for three weeks from receipt of data or as per contract.

Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.

* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Hina Siddiqui, Project Manager –Environmental Customer Service
Email: HSiddiqui@maxxam.ca
Phone# (905) 817-5700
==

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Total Cover Pages : 1

Maxxam Job #: B5I2028
Report Date: 2015/09/15

exp Services Inc
Client Project #: ADM-00223648-F0
Site Location: SIOUX LOOKOUT, ONTARIO
Sampler Initials: EF

RESULTS OF ANALYSES OF SOIL

Maxxam ID			AYK667	AYK668	AYK669	AYK670	AYK670	AYK671		
Sampling Date			2015/08/12 14:00	2015/08/14 17:00	2015/08/17 10:00	2015/08/20 01:15	2015/08/20 01:15	2015/08/11 12:00		
COC Number			NA	NA	NA	NA	NA	NA		
	UNITS	**Criteria**	**BH101-S10**	**BH104-S2**	**BH201-S9**	**BH204-S4**	**BH204-S4 Lab-Dup**	**BH302-S5**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	-	7400	5700	15000	4800		27000		4186431
Inorganics										
Soluble (20:1) Chloride (Cl)	ug/g	-	41	45	<20	57	45	<20	20	4188251
Conductivity	umho/cm	**470**	135	176	65	208	208	38	2	4188121
Available (CaCl2) pH	pH	-	7.93	7.56	7.50	6.98		5.99	N/A	4188358
Soluble (20:1) Sulphate (SO4)	ug/g	-	<20	<20	<20	<20	<20	<20	20	4188113

RDL = Reportable Detection Limit

QC Batch = Quality Control Batch

Lab-Dup = Laboratory Initiated Duplicate

Criteria: Ontario Reg. 153/04 (Amended April 15, 2011)
Table 1: Full Depth Background Site Condition Standards
Soil - Agricultural or Other Property Use

N/A = Not Applicable

Maxxam ID			AYK672		
Sampling Date			2015/08/12 13:00		
COC Number			NA		
	UNITS	**Criteria**	**BH303-S4B**	**RDL**	**QC Batch**
Calculated Parameters					
Resistivity	ohm-cm	-	5600		4186431
Inorganics					
Soluble (20:1) Chloride (Cl)	ug/g	-	<20	20	4188251
Conductivity	umho/cm	**470**	178	2	4188121
Available (CaCl2) pH	pH	-	7.34	N/A	4188358
Soluble (20:1) Sulphate (SO4)	ug/g	-	<20	20	4188113

RDL = Reportable Detection Limit

QC Batch = Quality Control Batch

Criteria: Ontario Reg. 153/04 (Amended April 15, 2011)
Table 1: Full Depth Background Site Condition Standards
Soil - Agricultural or Other Property Use

N/A = Not Applicable

TEST SUMMARY

Maxxam ID: AYK667
Sample ID: BH101-S10
Matrix: Soil

Collected: 2015/08/12
Shipped:
Received: 2015/09/09

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4188251	N/A	2015/09/15	Deonarine Ramnarine
Conductivity	AT	4188121	N/A	2015/09/14	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4188358	2015/09/14	2015/09/14	Neil Dassanayake
Resistivity of Soil		4186431	2015/09/14	2015/09/14	Cristina Carriere
Sulphate (20:1 Extract)	KONE/EC	4188113	N/A	2015/09/14	Alina Dobreanu

Maxxam ID: AYK668
Sample ID: BH104-S2
Matrix: Soil

Collected: 2015/08/14
Shipped:
Received: 2015/09/09

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4188251	N/A	2015/09/15	Deonarine Ramnarine
Conductivity	AT	4188121	N/A	2015/09/14	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4188358	2015/09/14	2015/09/14	Neil Dassanayake
Resistivity of Soil		4186431	2015/09/14	2015/09/14	Cristina Carriere
Sulphate (20:1 Extract)	KONE/EC	4188113	N/A	2015/09/14	Alina Dobreanu

Maxxam ID: AYK669
Sample ID: BH201-S9
Matrix: Soil

Collected: 2015/08/17
Shipped:
Received: 2015/09/09

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4188251	N/A	2015/09/15	Deonarine Ramnarine
Conductivity	AT	4188121	N/A	2015/09/14	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4188358	2015/09/14	2015/09/14	Neil Dassanayake
Resistivity of Soil		4186431	2015/09/14	2015/09/14	Cristina Carriere
Sulphate (20:1 Extract)	KONE/EC	4188113	N/A	2015/09/14	Alina Dobreanu

Maxxam ID: AYK670
Sample ID: BH204-S4
Matrix: Soil

Collected: 2015/08/20
Shipped:
Received: 2015/09/09

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4188251	N/A	2015/09/15	Deonarine Ramnarine
Conductivity	AT	4188121	N/A	2015/09/14	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4188358	2015/09/14	2015/09/14	Neil Dassanayake
Resistivity of Soil		4186431	2015/09/14	2015/09/14	Cristina Carriere
Sulphate (20:1 Extract)	KONE/EC	4188113	N/A	2015/09/14	Alina Dobreanu

Maxxam ID: AYK670 Dup
Sample ID: BH204-S4
Matrix: Soil

Collected: 2015/08/20
Shipped:
Received: 2015/09/09

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4188251	N/A	2015/09/15	Deonarine Ramnarine

TEST SUMMARY

Maxxam ID: AYK670 Dup
Sample ID: BH204-S4
Matrix: Soil

Collected: 2015/08/20
Shipped:
Received: 2015/09/09

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	4188121	N/A	2015/09/14	Neil Dassanayake
Sulphate (20:1 Extract)	KONE/EC	4188113	N/A	2015/09/14	Alina Dobreanu

Maxxam ID: AYK671
Sample ID: BH302-S5
Matrix: Soil

Collected: 2015/08/11
Shipped:
Received: 2015/09/09

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4188251	N/A	2015/09/15	Deonarine Ramnarine
Conductivity	AT	4188121	N/A	2015/09/14	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4188358	2015/09/14	2015/09/14	Neil Dassanayake
Resistivity of Soil		4186431	2015/09/14	2015/09/14	Cristina Carriere
Sulphate (20:1 Extract)	KONE/EC	4188113	N/A	2015/09/14	Alina Dobreanu

Maxxam ID: AYK672
Sample ID: BH303-S4B
Matrix: Soil

Collected: 2015/08/12
Shipped:
Received: 2015/09/09

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4188251	N/A	2015/09/15	Deonarine Ramnarine
Conductivity	AT	4188121	N/A	2015/09/14	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4188358	2015/09/14	2015/09/14	Neil Dassanayake
Resistivity of Soil		4186431	2015/09/14	2015/09/14	Cristina Carriere
Sulphate (20:1 Extract)	KONE/EC	4188113	N/A	2015/09/14	Alina Dobreanu

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	3.3°C

Results relate only to the items tested.

Maxxam Job #: B5I2028
Report Date: 2015/09/15

QUALITY ASSURANCE REPORT

exp Services Inc
Client Project #: ADM-00223648-F0
Site Location: SIOUX LOOKOUT, ONTARIO
Sampler Initials: EF

			Matrix Spike		SPIKED BLANK		Method Blank		RPD		QC Standard	
QC Batch	**Parameter**	**Date**	**% Recovery**	**QC Limits**	**% Recovery**	**QC Limits**	**Value**	**UNITS**	**Value (%)**	**QC Limits**	**% Recovery**	**QC Limits**
4188113	Soluble (20:1) Sulphate (SO4)	2015/09/14	113	70 - 130	102	70 - 130	<20	ug/g	NC	35		
4188121	Conductivity	2015/09/14			99	90 - 110	<2	umho/cm	0	10	117	75 - 125
4188251	Soluble (20:1) Chloride (Cl)	2015/09/15	NC	70 - 130	101	70 - 130	<20	ug/g	NC	35		
4188358	Available (CaCl2) pH	2015/09/14			99	97 - 103			1.9	N/A		

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

QC Standard: A sample of known concentration prepared by an external agency under stringent conditions. Used as an independent check of method accuracy.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spiked amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than 2x that of the native sample concentration).

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (one or both samples < 5x RDL).

Maxxam Job #: B5I2028
Report Date: 2015/09/15

exp Services Inc
Client Project #: ADM-00223648-F0
Site Location: SIOUX LOOKOUT, ONTARIO
Sampler Initials: EF

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Cristina Carriere

Cristina Carriere, Scientific Services

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam IMMEDIATE TEST 6740 Campobello Road, Mississauga, Ontario L5N 2L8 www.maxxam.ca
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266

CHAIN OF COUSTODY RECORD

Page 1 of 1

INVOICE INFORMATION

Company Name: exp Services Inc.
Contact Name: Michael Suslyk, Ahileas Mitsopoulos
Address: 1142 Roland Street
Thunder Bay, ON P7B 5M4
Phone: 807.623.9495 Fax: 807.623.8070
Email: michael.suslyk@exp.com, ahileas.mitsopoulos@exp.com

REPORT INFORMATION (if differs from invoice)

Company Name:
Contact Name:
Address:
Phone: Fax:
Email:

PROJECT INFORMATION

Quotation #:
P.O. #:
Project #: ADM-00223648-F0
Site Location: Sioux Lookout, Ontario
Site #:
Sampled By: Elwin Farkas

TURNAROUND TIME (TAT) REQUIRED

☒ Regular TAT (5-7 days)

PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS

Rush TAT (Applicable Surcharge)
☐ 1 Day (100%)
☐ 2 Days (50%)
☐ 3-4 Days (25%)

Rush Confirmation #:
Date Required:

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

REGULATION 153 (2011)

☒ Table 1 ☐ Res/Park ☐ Med/Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/Other
☐ Table ___

FOR RSC (PLEASE CIRCLE) Yes / [✓ No]

OTHER REGULATIONS

☐ CCME ☐ Sanitary Sewer Bylaw
☐ MISA ☐ Storm Sewer Bylaw
☐ PWQO Municipality:
☐ Other (Specify):
☐ REG 558 (MINIMUM 3 DAY TAT REQUIRED)

Include Criteria on Certificate of Analysis (Y/N)? Y

SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM

LABORATORY USE ONLY

CUSTODY SEAL (Y/N)		Temperature (°C) on Receipt
Present	Y	3/4/3
Intact	Y	
COOLING MEDIA PRESENT (Y / N)	Y	

ANALYSIS REQUESTED

	SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	MATRIX	# OF CONT.	FIELD FILTERED (PLEASE CIRCLE) Metals / Hg / CrVI	pH	Water Soluble Sulphate	Resistivity	Conductivity	Chloride	PHC F1 to F4 (including BTEX)	TCLP	COMMENTS / TAT COMMENTS
1	BH101-S10	12-Aug-15	14:00	Soil	2		x	x	x	x	x			
2	BH104-S2	14-Aug-15	17:00	Soil	2		x	x	x	x	x			
3	BH201-S9	17-Aug-15	10:00	Soil	2		x	x	x	x	x			
4	BH204-S4	20-Aug-15	1:15	Soil	2		x	x	x	x	x			
5	BH302-S5	11-Aug-15	12:00	Soil	2		x	x	x	x	x			
6	BH303-S4B	12-Aug-15	13:00	Soil	2		x	x	x	x	x			
7	BH203-S2	20-Aug-15	13:00	Soil	3							x		Hold testing until confirmation from **exp** Services
8	BH203-S3	20-Aug-15	13:03	Soil	4							x	x	Hold teseting until confirmation from **exp** Services
9	BH203-S4	20-Aug-15	13:05	Soil	3							x		Hold testing until confirmation from **exp** Services
10														

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME:	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME:	# JARS USED AND NOT SUBMITTED
Michael Suslyk	08-Sep-15	15:30	ASAD [illegible]	2015/09/09	09:30	

09-Sep-15 09:30
Hina Siddiqui
B5I2028
FSD ENV-746

Maxxam Analytics International Corporation o/a Maxxam Analytics

Appendix F –
Slope Stability Analysis

Minnikau River Culverts
Hwy 642
North Embankment (Upstream)
Drained Condtion

Name: Sand with Silt Gravel Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 36 °
Name: Peat (Very Soft to Soft) Model: Mohr-Coulomb Unit Weight: 15 kN/m³ Cohesion': 3 kPa Phi': 17 °
Name: Sand to Sand with Silt (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 21.5 kN/m³ Cohesion': 0 kPa Phi': 31 °
Name: Silt (Very Loose) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Clayey Silt (Very Soft to Soft) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 28 °
Name: Sandy Silt with Gravel (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Silty Sand (Very Loose) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 33 °
Name: Cobbles and Boulders Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 38 °

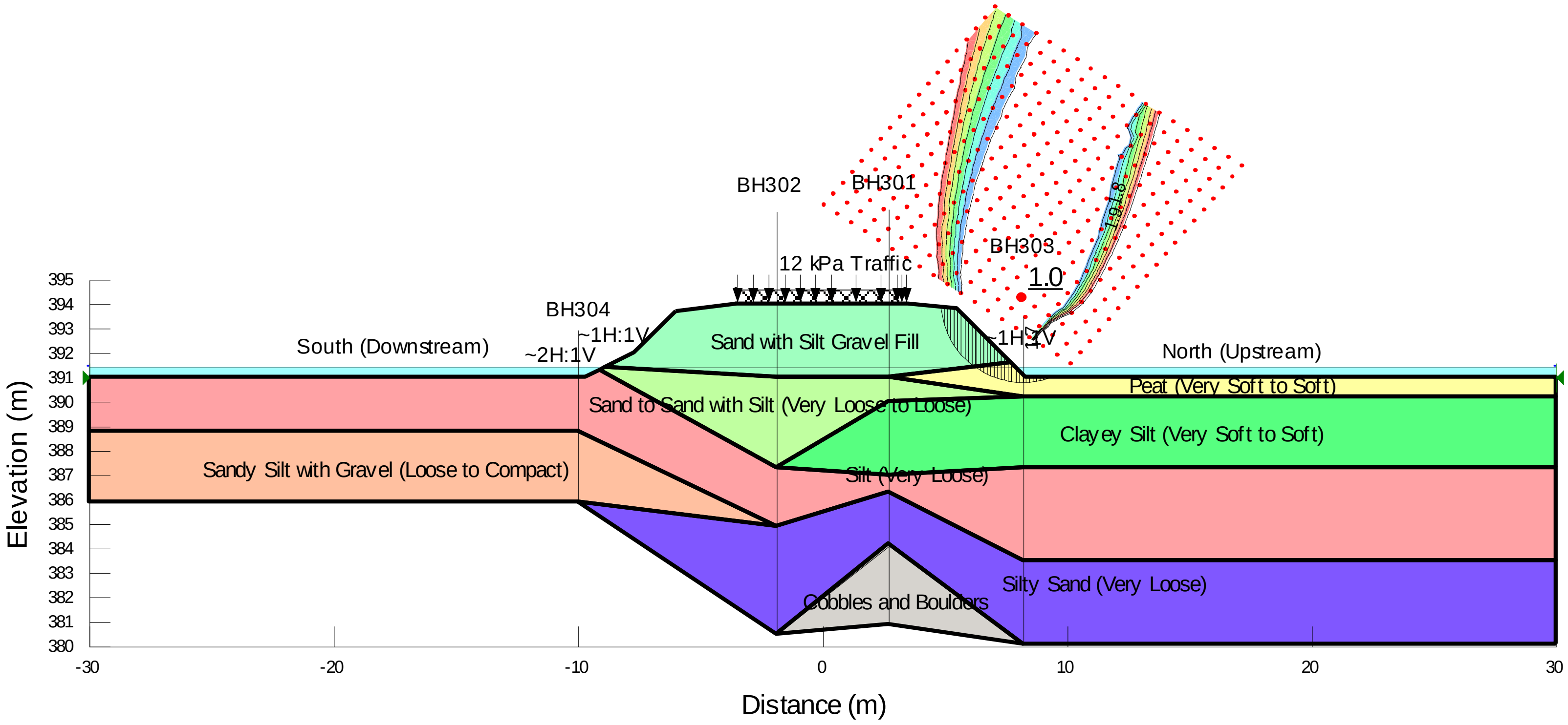


Figure F1: North embankment (upstream) – drained static condition with existing slope

Minnikau River Culverts
Hwy 642
South Embankment (Downstream)
Drained Condtion

Name: Silty Sand with Gravel Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Peat (Very Soft to Soft) Model: Mohr-Coulomb Unit Weight: 15 kN/m³ Cohesion': 3 kPa Phi': 17 °
Name: Sand to Sand with Silt (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 21.5 kN/m³ Cohesion': 0 kPa Phi': 31 °
Name: Silt (Very Loose) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Clayey Silt (Very Soft to Soft) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 28 °
Name: Sandy Silt with Gravel (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Silty Sand (Very Loose) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 33 °
Name: Cobbles and Boulders Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 38 °

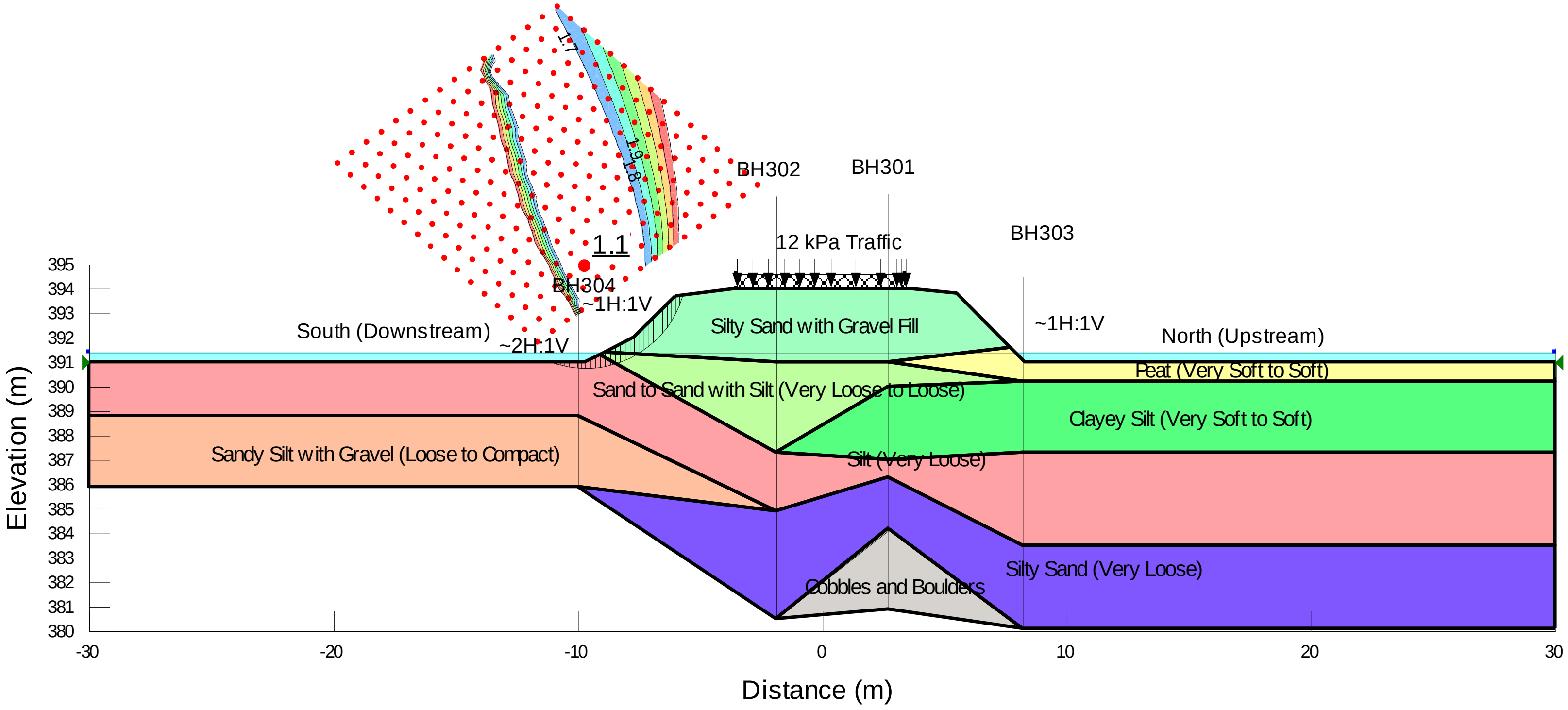


Figure F2: South embankment (downstream) – drained static condition with existing slope

Minnikau River Culverts
Hwy 642
North Embankment (Upstream)
Drained Condtion

Name: Sand with Silt Gravel Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 36 °
Name: Peat (Very Soft to Soft) Model: Mohr-Coulomb Unit Weight: 15 kN/m³ Cohesion': 3 kPa Phi': 17 °
Name: Sand to Sand with Silt (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 21.5 kN/m³ Cohesion': 0 kPa Phi': 31 °
Name: Silt (Very Loose) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Clayey Silt (Very Soft to Soft) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 28 °
Name: Sandy Silt with Gravel (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Silty Sand (Very Loose) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 33 °
Name: Cobbles and Boulders Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 38 °
Name: Engineered Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 32 °

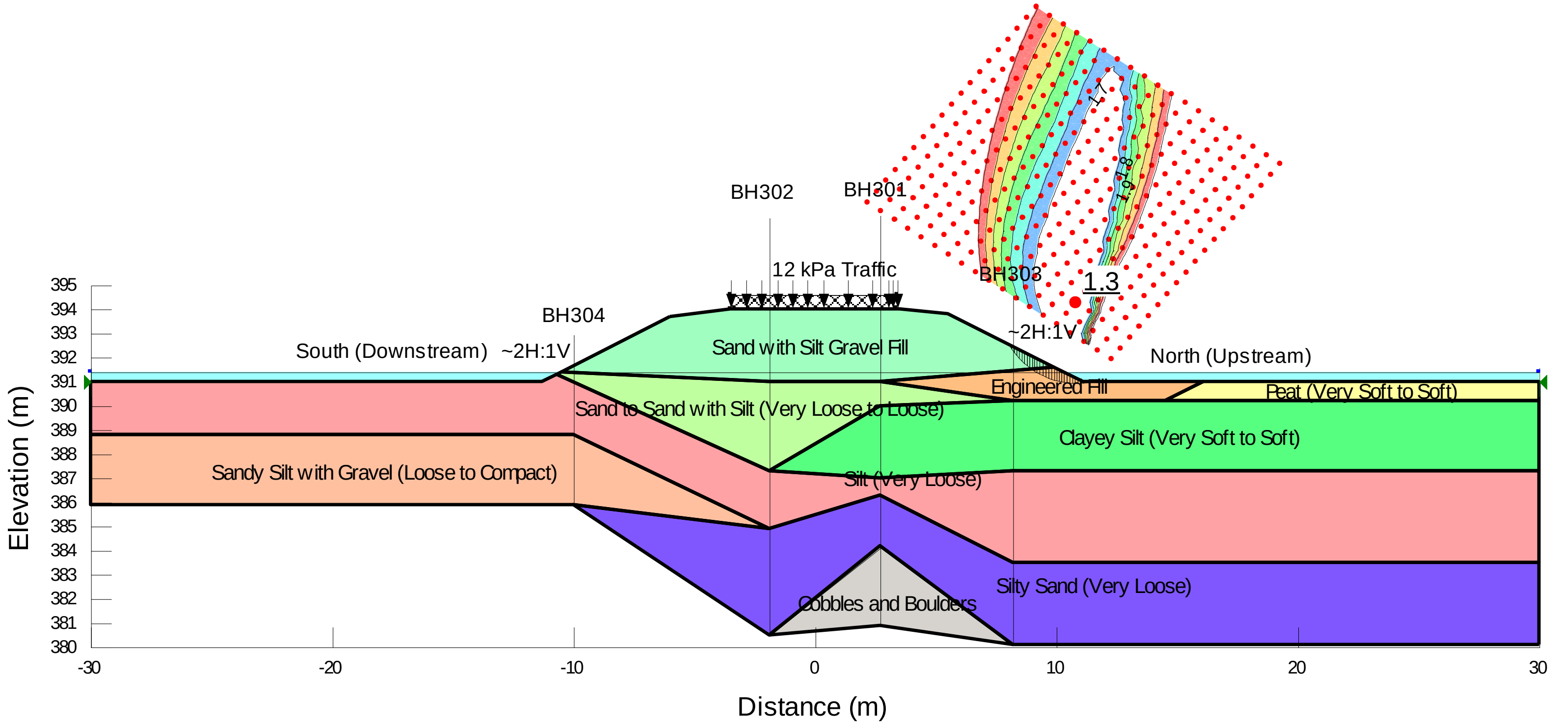


Figure F3: North embankment (upstream) – drained static condition with slope 2H:1V

Minnikau River Culverts
Hwy 642
South Embankment (Downstream)
Drained Condtion

Name: Silty Sand with Gravel Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Peat (Very Soft to Soft) Model: Mohr-Coulomb Unit Weight: 15 kN/m³ Cohesion': 3 kPa Phi': 17 °
Name: Sand to Sand with Silt (Very Loose to Loose) Model: Mohr-Coulomb Unit Weight: 21.5 kN/m³ Cohesion': 0 kPa Phi': 31 °
Name: Silt (Very Loose) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 29 °
Name: Clayey Silt (Very Soft to Soft) Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 28 °
Name: Sandy Silt with Gravel (Loose to Compact) Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Silty Sand (Very Loose) Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 33 °
Name: Cobbles and Boulders Model: Mohr-Coulomb Unit Weight: 22 kN/m³ Cohesion': 0 kPa Phi': 38 °

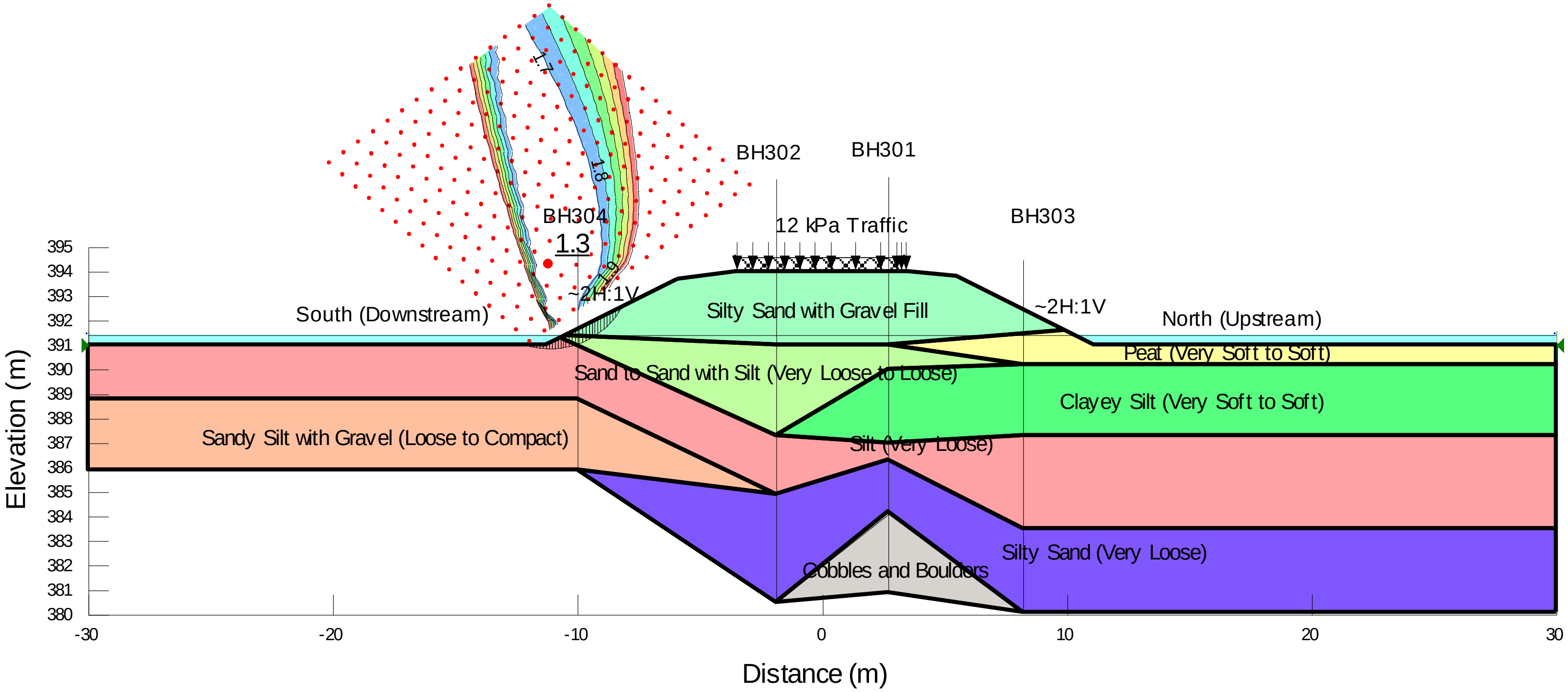
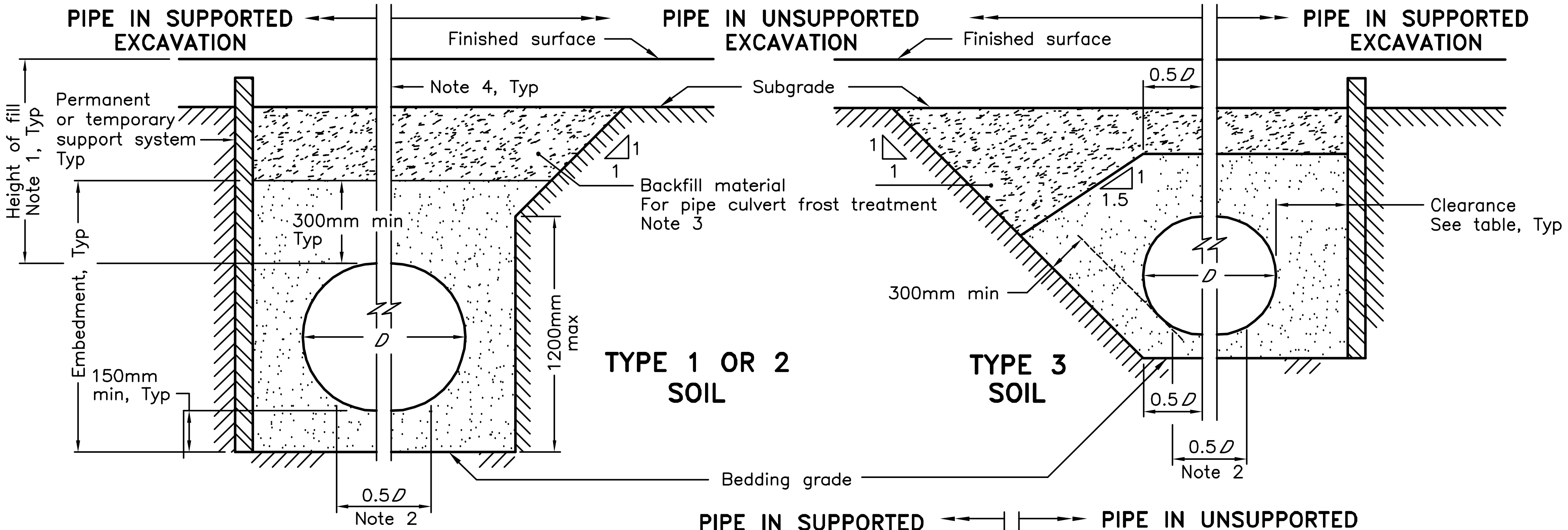
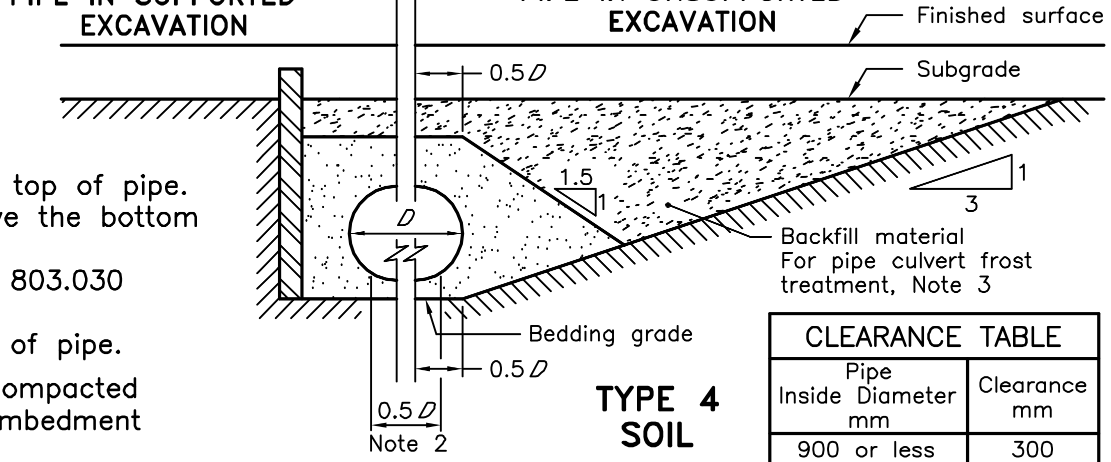


Figure F4: South embankment (downstream) – drained static condition with slope 2H:1V

Appendix G –
OPSDs





LEGEND:

D – Inside diameter

NOTES:

1 Height of fill is measured from the finished surface to top of pipe.
2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
3 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
4 Condition of excavation is symmetrical about centreline of pipe.

A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.
B Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
C All dimensions are in metres unless otherwise shown.

CLEARANCE TABLE

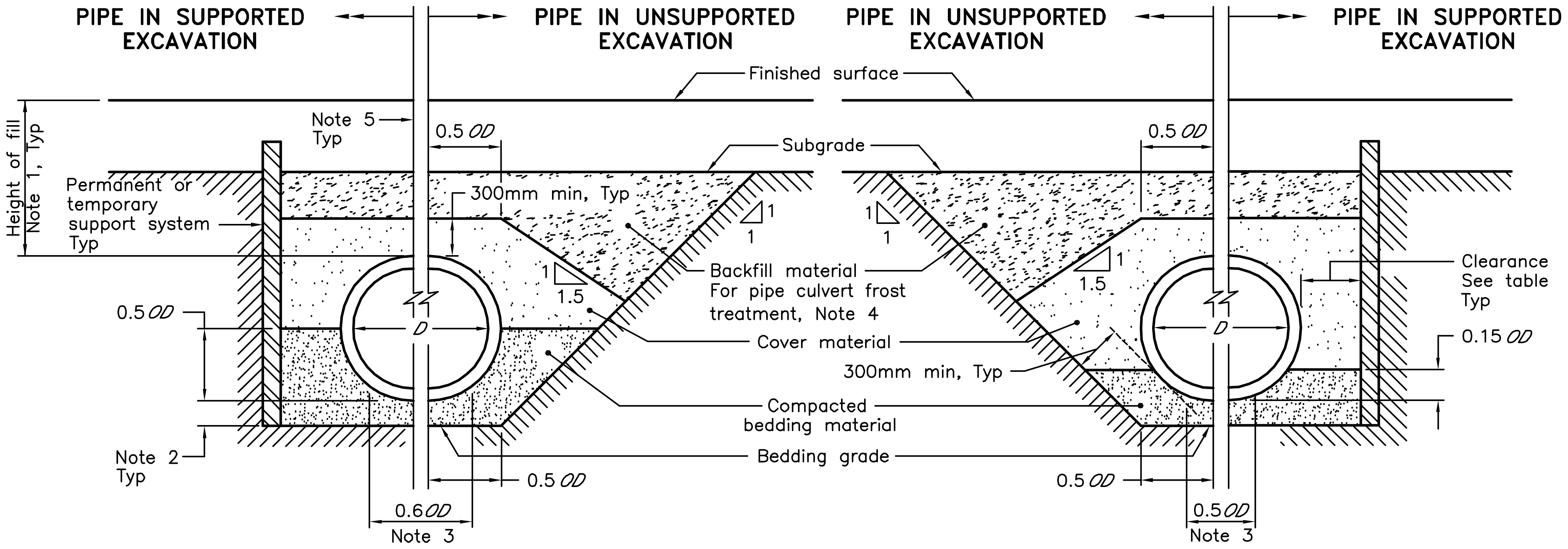
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 2

FLEXIBLE PIPE
EMBEDMENT AND BACKFILL
EARTH EXCAVATION

OPSD 802.010



CLASS B BEDDING

CLASS C BEDDING

NOTES:

1 Height of fill is measured from the finished surface to top of pipe.
2 The minimum bedding depth below the pipe shall be 0.15*D*. In no case shall this dimension be less than 150mm or greater than 300mm.
3 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
4 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
5 Condition of excavation is symmetrical about centreline of pipe.
A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
B All dimensions are in metres unless otherwise shown.

LEGEND:

D – Inside diameter
OD – Outside diameter

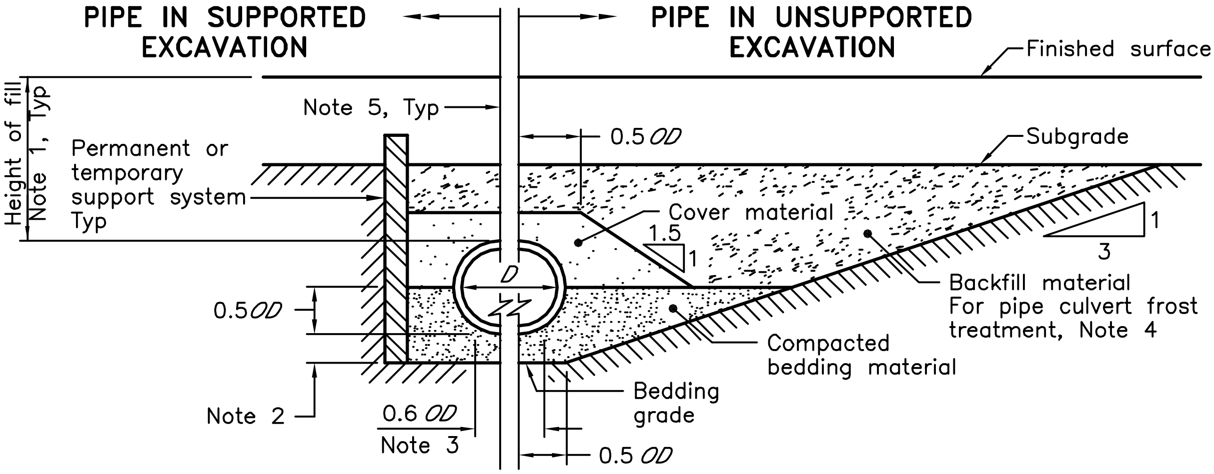
CLEARANCE TABLE

Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

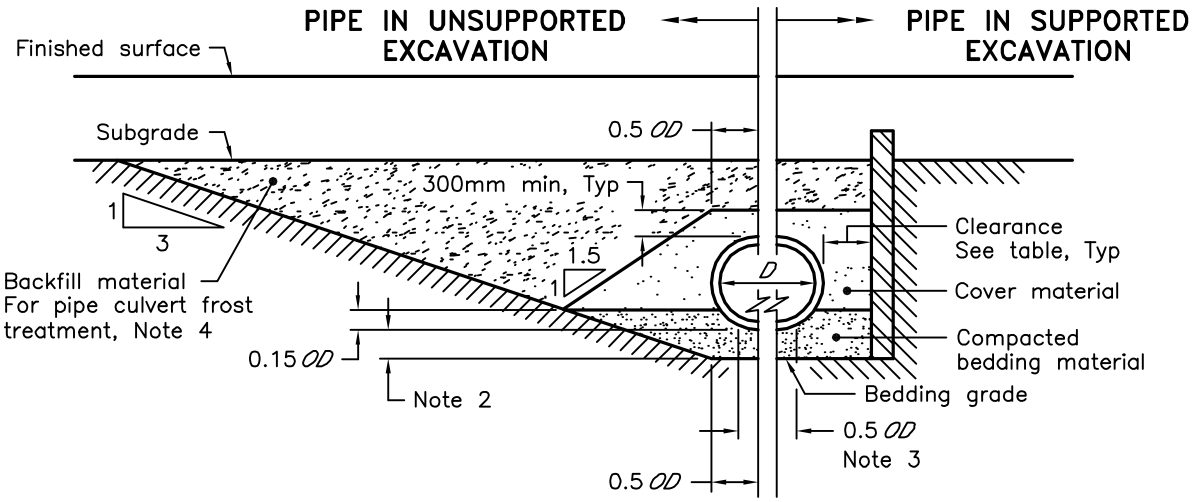
ONTARIO PROVINCIAL STANDARD DRAWING
Nov 2010 Rev 2

RIGID PIPE BEDDING, COVER, AND BACKFILL TYPE 3 SOIL – EARTH EXCAVATION

OPSD 802.031



CLASS B BEDDING



CLASS C BEDDING

CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

LEGEND:

D — Inside diameter
OD — Outside diameter

NOTES:

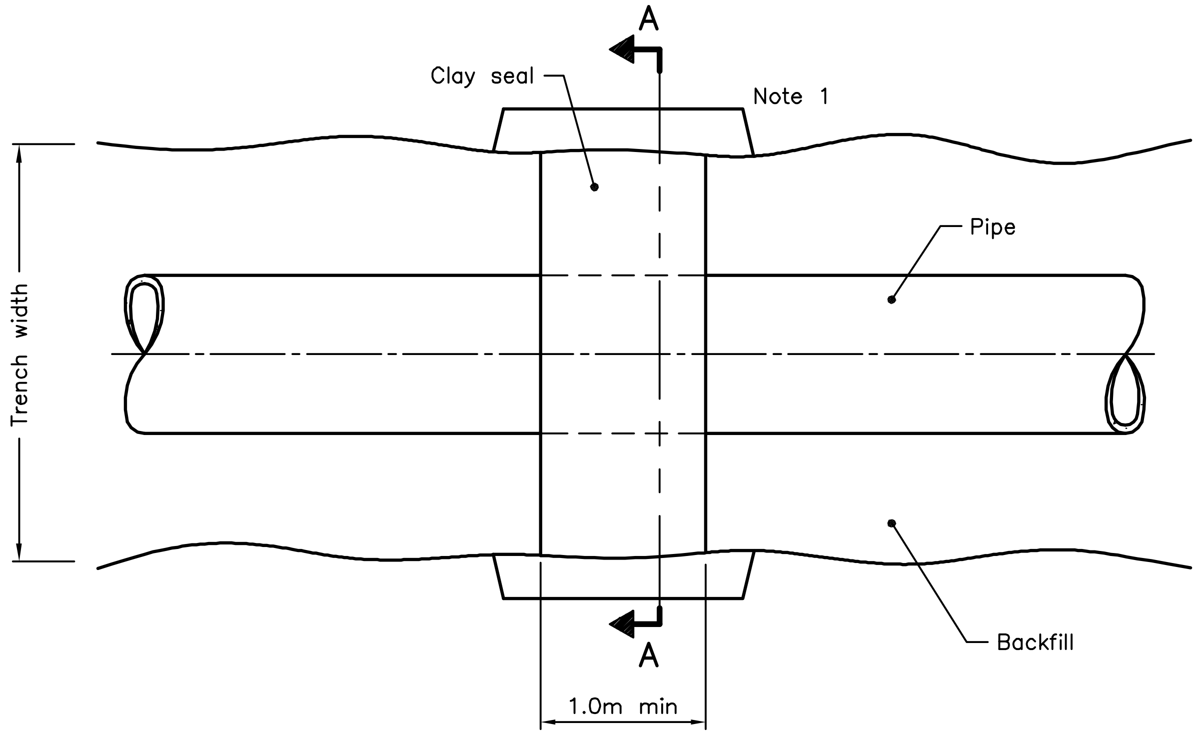
1 Height of fill is measured from the finished surface to top of pipe.
2 The minimum bedding depth below the pipe shall be 0.15*D*. In no case shall this dimension be less than 150mm or greater than 300mm.
3 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
4 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.
5 Condition of excavation is symmetrical about centreline of pipe.
A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
B All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

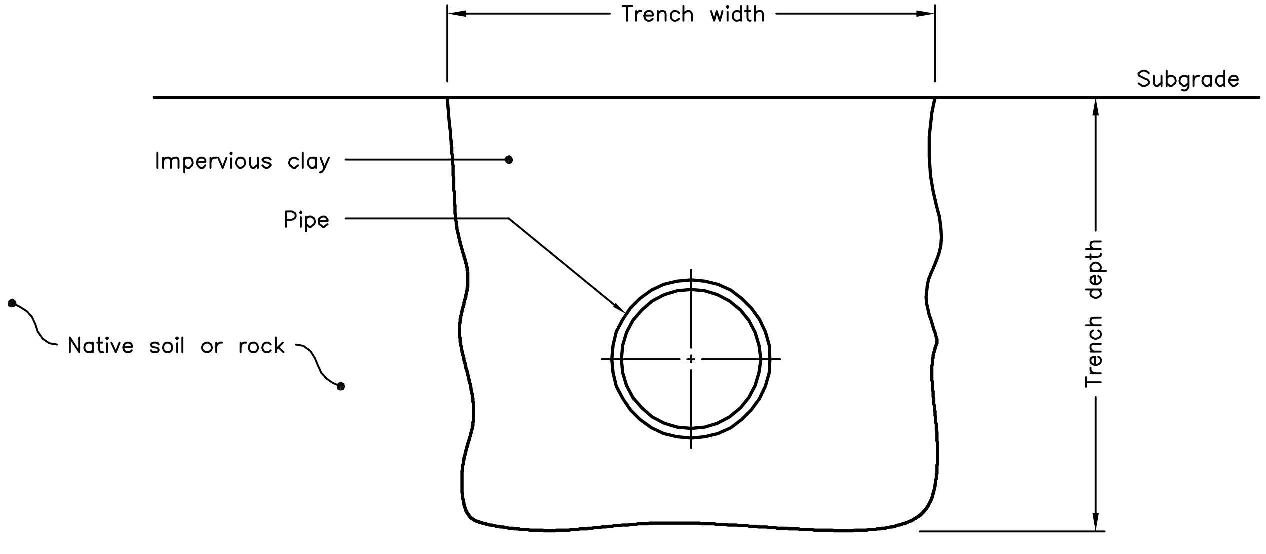
Nov 2010 | Rev 2

RIGID PIPE BEDDING, COVER, AND BACKFILL TYPE 4 SOIL – EARTH EXCAVATION

OPSD 802.032



PLAN



SECTION A–A

NOTES:

1. Key into undisturbed trench soil.

A Clay seal shall extend from bottom of trench excavation to the subgrade.
B Clay seal shall be located so that no pipe joints are within the clay seal material.
C All dimensions are in metres unless otherwise shown.

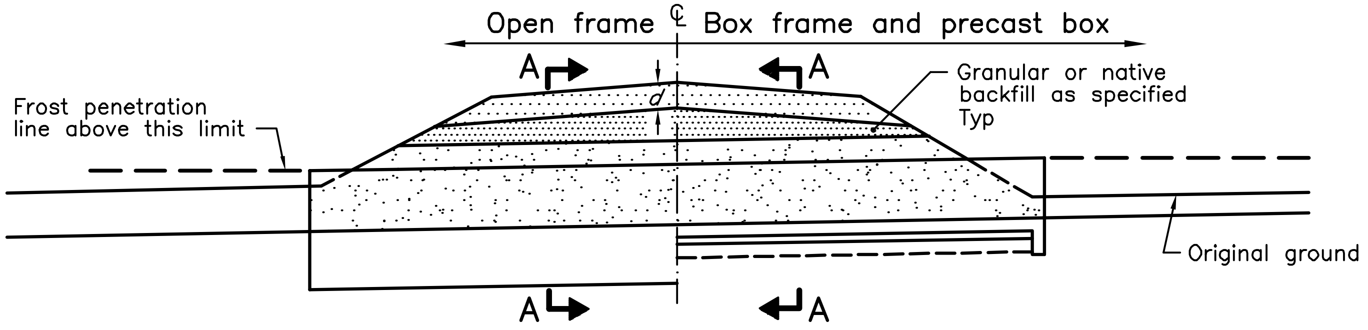
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2011 Rev 1

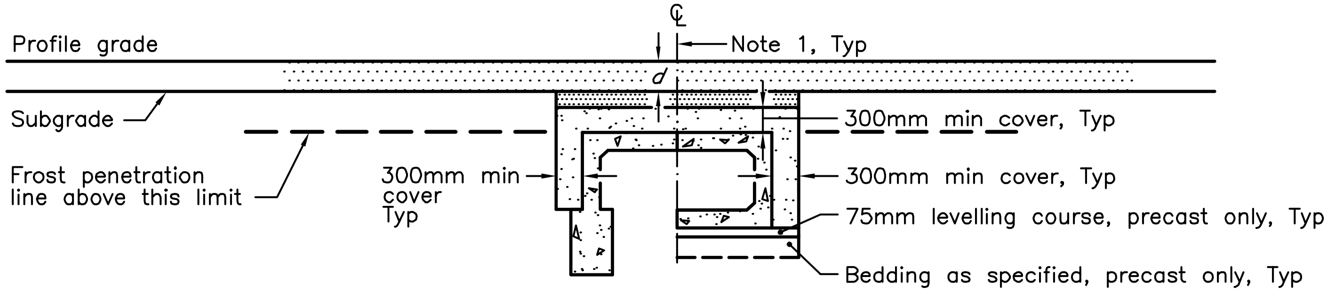
CLAY SEAL FOR PIPE TRENCHES

OPSD 802.095

FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT

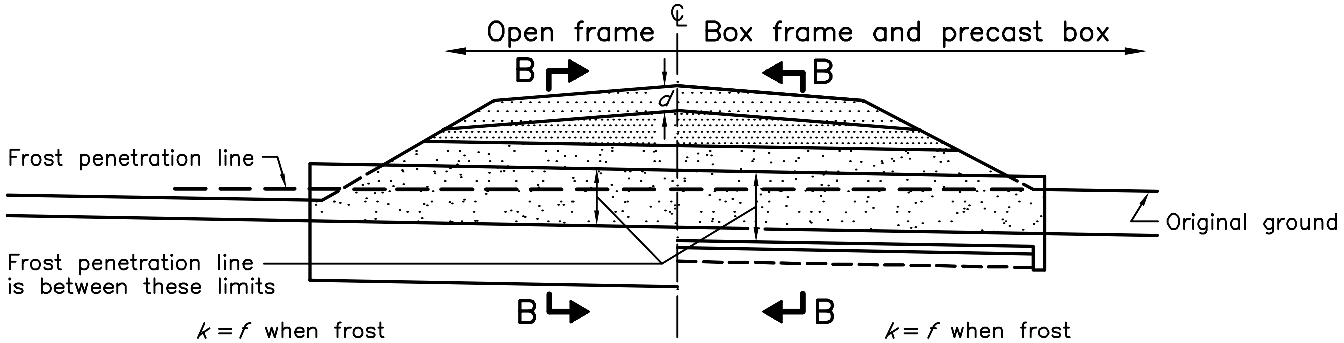


LONGITUDINAL SECTION

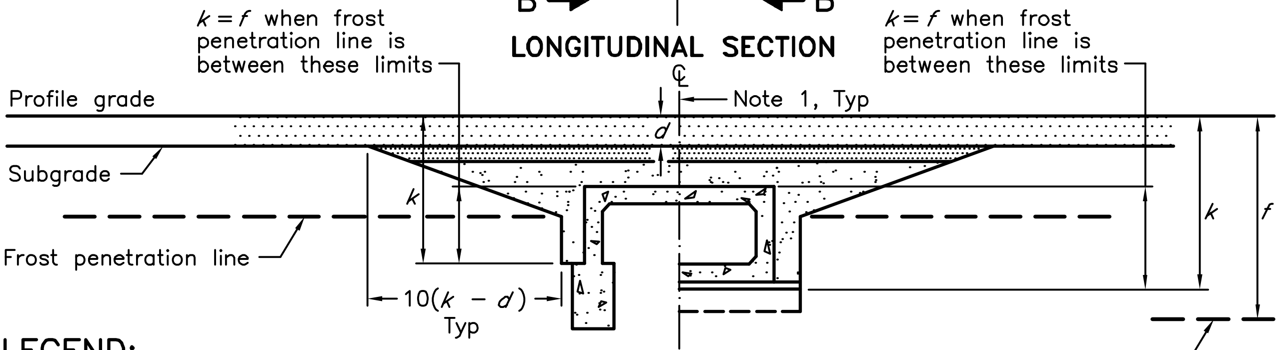


SECTION A–A

FROST PENETRATION LINE BELOW TOP OF CULVERT



LONGITUDINAL SECTION



SECTION B–B

Frost penetration line below bottom of culvert footing or slab

LEGEND:

d = depth of roadbed granular
k = depth of frost treatment below profile grade
f = depth of frost penetration below profile grade

NOTES:

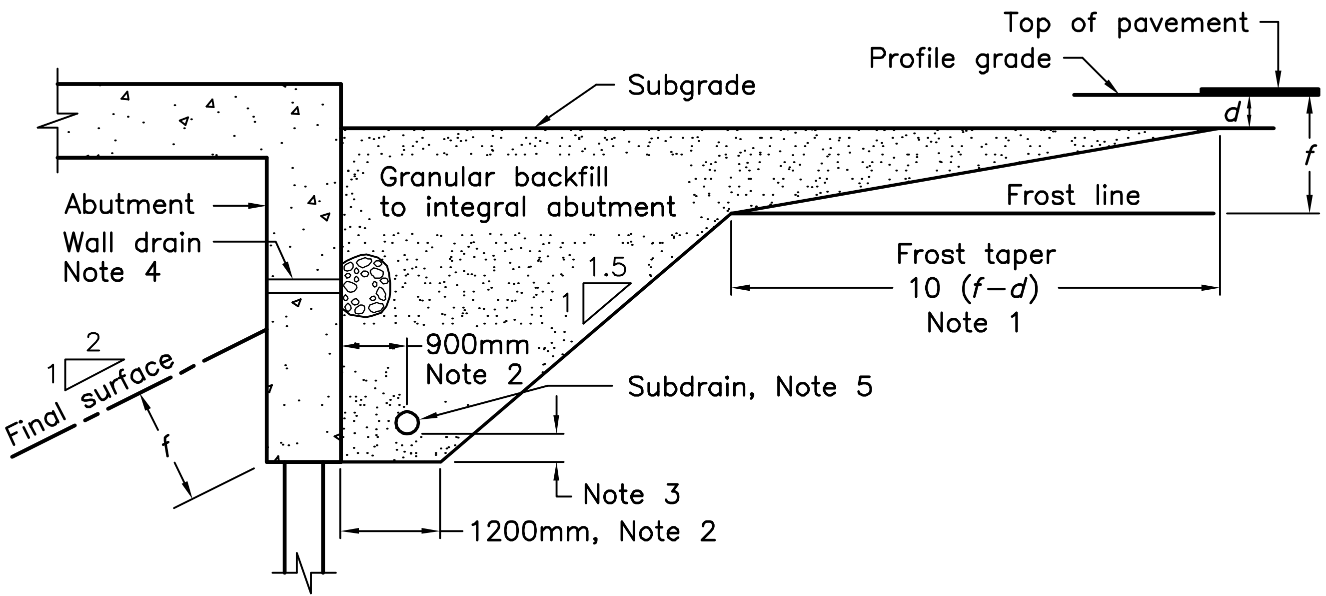
1 Condition of frost treatment symmetrical about centreline of culvert.

A Bedding, levelling, and cover material shall be granular as specified.
B The depth of roadbed granular shall be 600mm minimum.
C The maximum depth of frost treatment shall be bottom of box frame or top of footing.
D All dimensions are in millimetres unless otherwise shown.

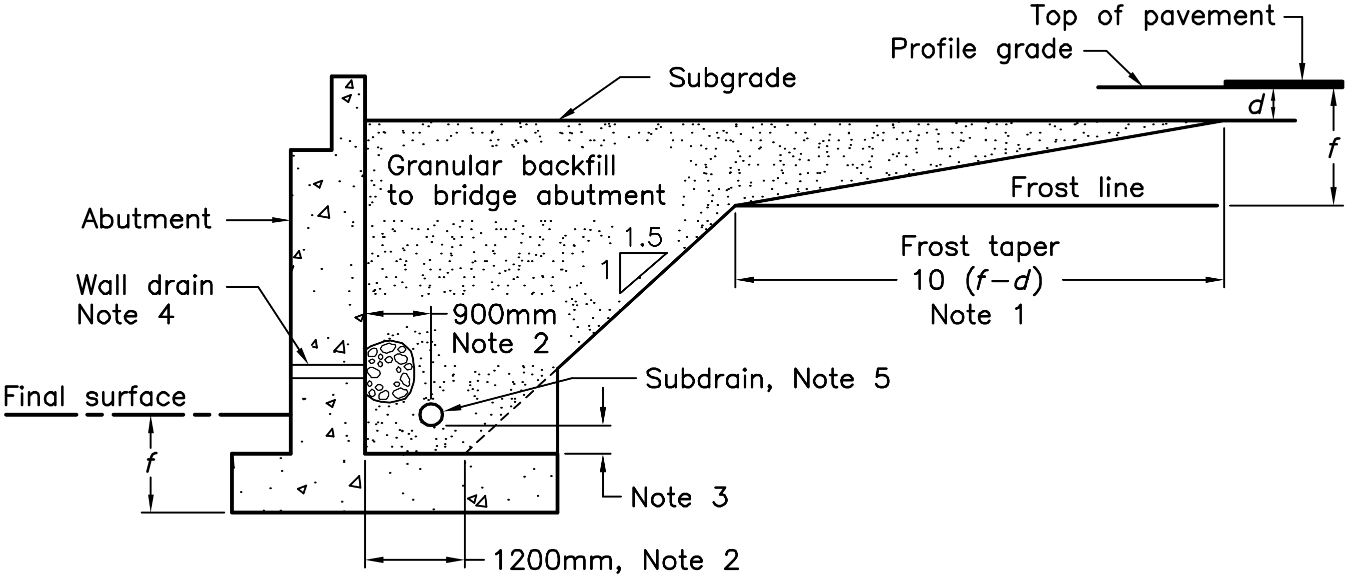
ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2010 | Rev 2

BACKFILL AND COVER FOR
CONCRETE CULVERTS WITH SPANS
LESS THAN OR EQUAL TO 3.0M

OPSD 803.010



INTEGRAL ABUTMENT



ABUTMENT

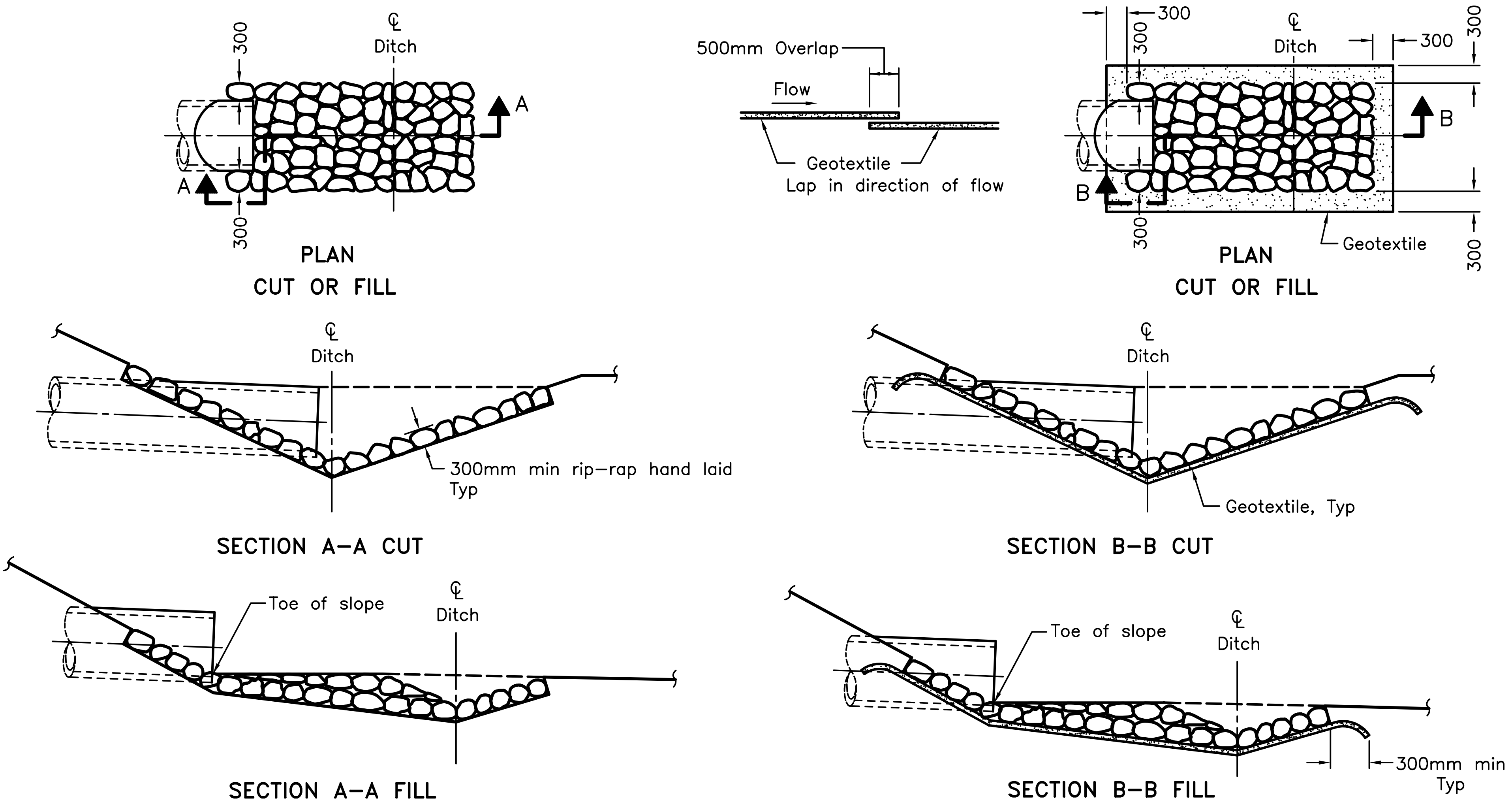
NOTES:

1 d = depth of combined base and subbase courses
  f = frost penetration depth as specified
2 Dimensions perpendicular to back face of abutment.
3 Height to be consistent with positive drainage of subdrain as specified.
4 Where specified, wall drains shall be installed according to OPSD 3190.100.
5 150mm dia perforated pipe subdrain wrapped with geotextile.
A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the backfill unless interrupted by the retaining wall or wingwall.
B Sections shown are parallel to centreline of roadway.
C Subdrain shall be installed with a 2% gradient behind wall.
D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2010 | Rev 1

WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

OPSD 3101.150



PLAN
CUT OR FILL

PLAN
CUT OR FILL

SECTION A–A CUT

SECTION B–B CUT

SECTION A–A FILL

SECTION B–B FILL

TYPE A – WITHOUT GEOTEXTILE

TYPE B – WITH GEOTEXTILE

NOTES:

A All dimensions are in millimetres unless otherwise shown.

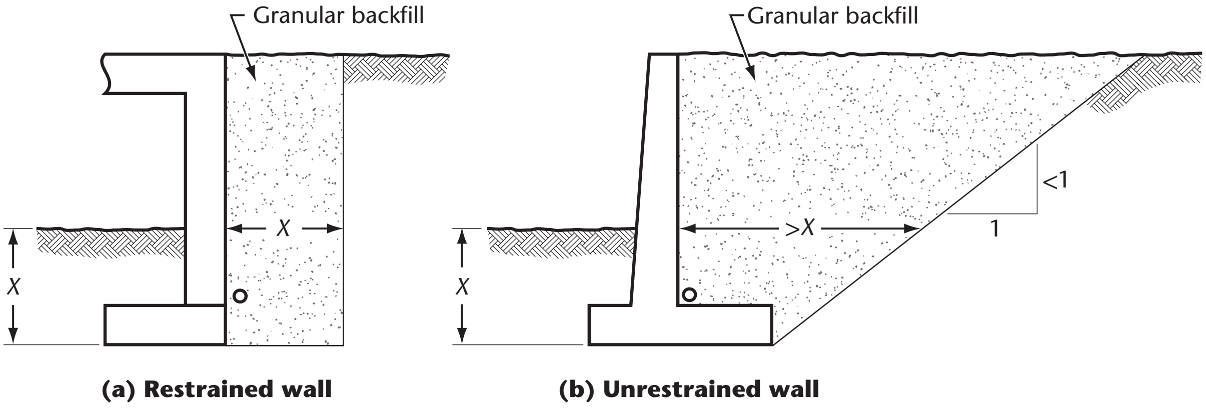
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2001 | Rev 0

RIP–RAP TREATMENT
FOR SEWER AND CULVERT OUTLETS

OPSD – 810.010

If rock fill is used as a backfill material, consideration should be given to the possible deterioration of the rockfill with time, which could result in the reduction or even the total loss of free-draining properties and, hence, increased frost susceptibility.



(a) Restrained wall **(b) Unrestrained wall**

Figure C6.20
Backfill for frost protection
(See Clause C6.9.1.)

C6.9.2 Lateral pressures

C6.9.2.1 General

Earth pressure acting on a structure depends on the relative movement of the structure, the backfill, the type of soil adjacent to the backfill, and the soil below the footing or supporting piles. Appropriate geotechnical parameters should be chosen for the calculation of lateral pressures based on recognized geotechnical theories as specified in Clause 6.9.2.2 for the backfill behind the wall. Geotechnical parameters frequently used in allowable stress design methods are applicable in limit states design pressure calculation. Where the possibility exists, hydrostatic pressure needs to be considered, e.g., in situations where walls are partially submerged or where non-free-draining backfill is used.

Clause 6.9.2.1 includes the specification of four lateral pressure conditions for design. The first two cases apply to unrestrained structures, with Item (a) applying to the sizing of the base or pile arrangement with respect to external stability, and Item (b) to the sizing of the structural sections with respect to internal stability. Such sections could be of structural concrete, structural steel, or a proprietary product.

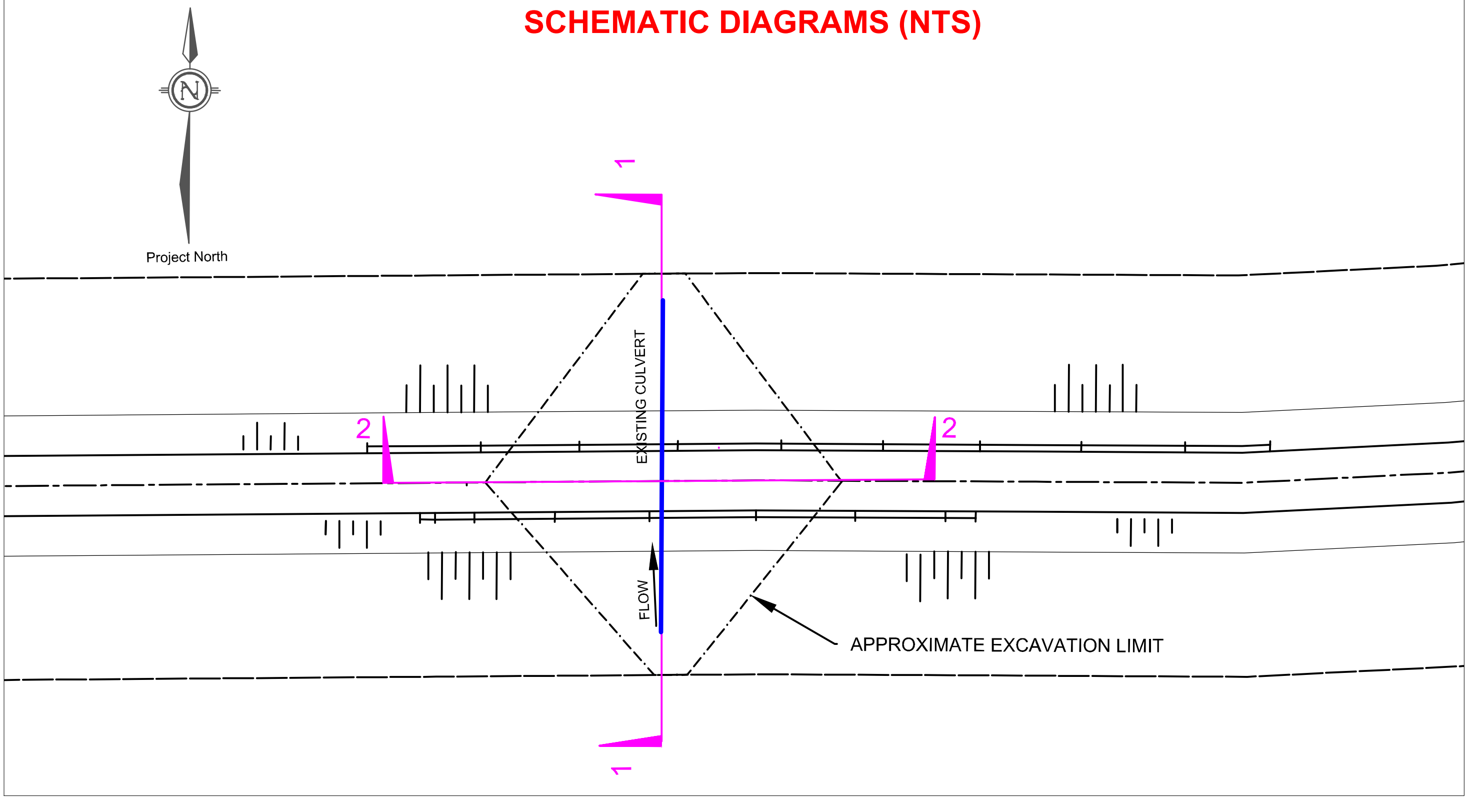
An unrestrained structure is one in which active pressure is mobilized in the backfill due to movement in the supporting structure. This movement corresponds to a rotation of approximately 0.002 about the base of a vertical wall, a horizontal translation of 0.001 times the height of the wall, or a combination of these movements. The lateral pressure applied to the wall for the condition described is an active pressure.

The supporting material will generally be more robust than what is assumed by the Geotechnical Engineer for factored conditions in design. Hence, following installation of the backfill, movement sufficient to cause active condition will generally not have taken place. Horizontal or rotational movement of the base will occur during the installation of each lift of the backfill. Wall deflection during each application and compaction of the backfill will add to the existing deformations. For such a post placement of the fill condition, Item (b) applies, the forces acting on the retaining structure being a function of the compacting equipment and the flexural stiffness of the wall. The residual horizontal pressures due to compaction are largest at the top of the wall, and this is reflected in Clause 6.9.3.

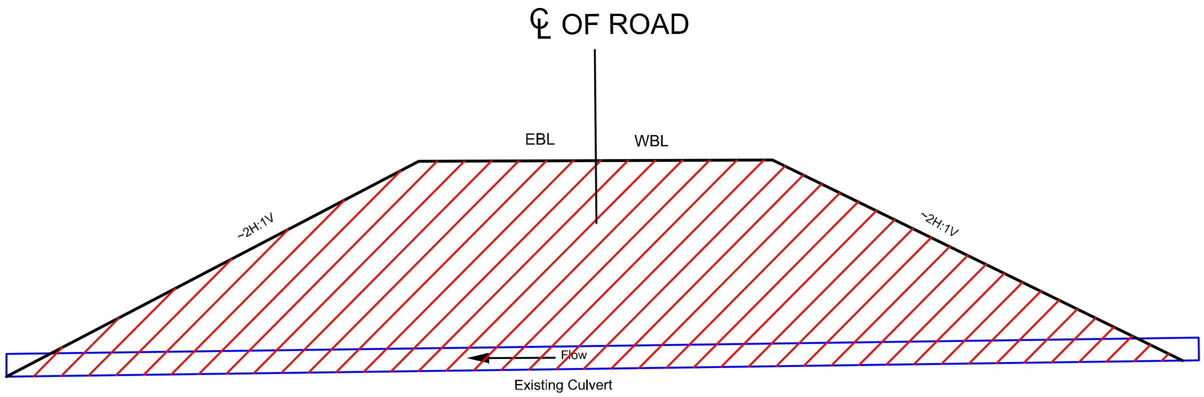
Appendix H –
Schematic Sketches for Construction Alternatives

FIGURE H.1: FULL ROAD CLOSURE USING EXISTING ROADWAYS AND OPEN CUT UNSUPORTED EXCAVATION OPTION1

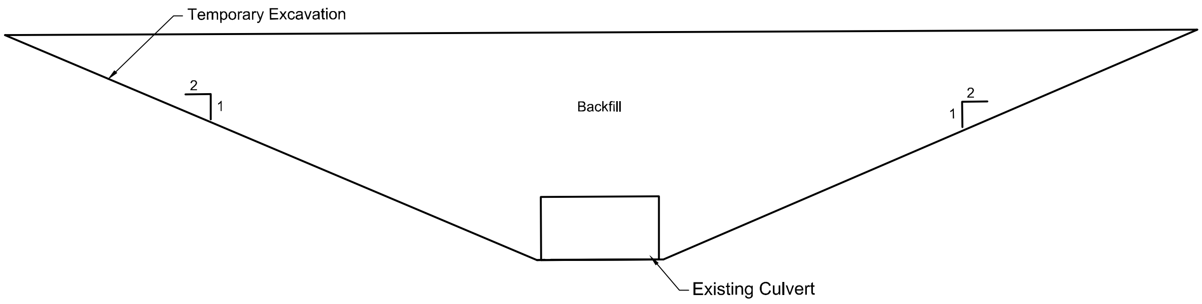
SCHEMATIC DIAGRAMS (NTS)



PLAN



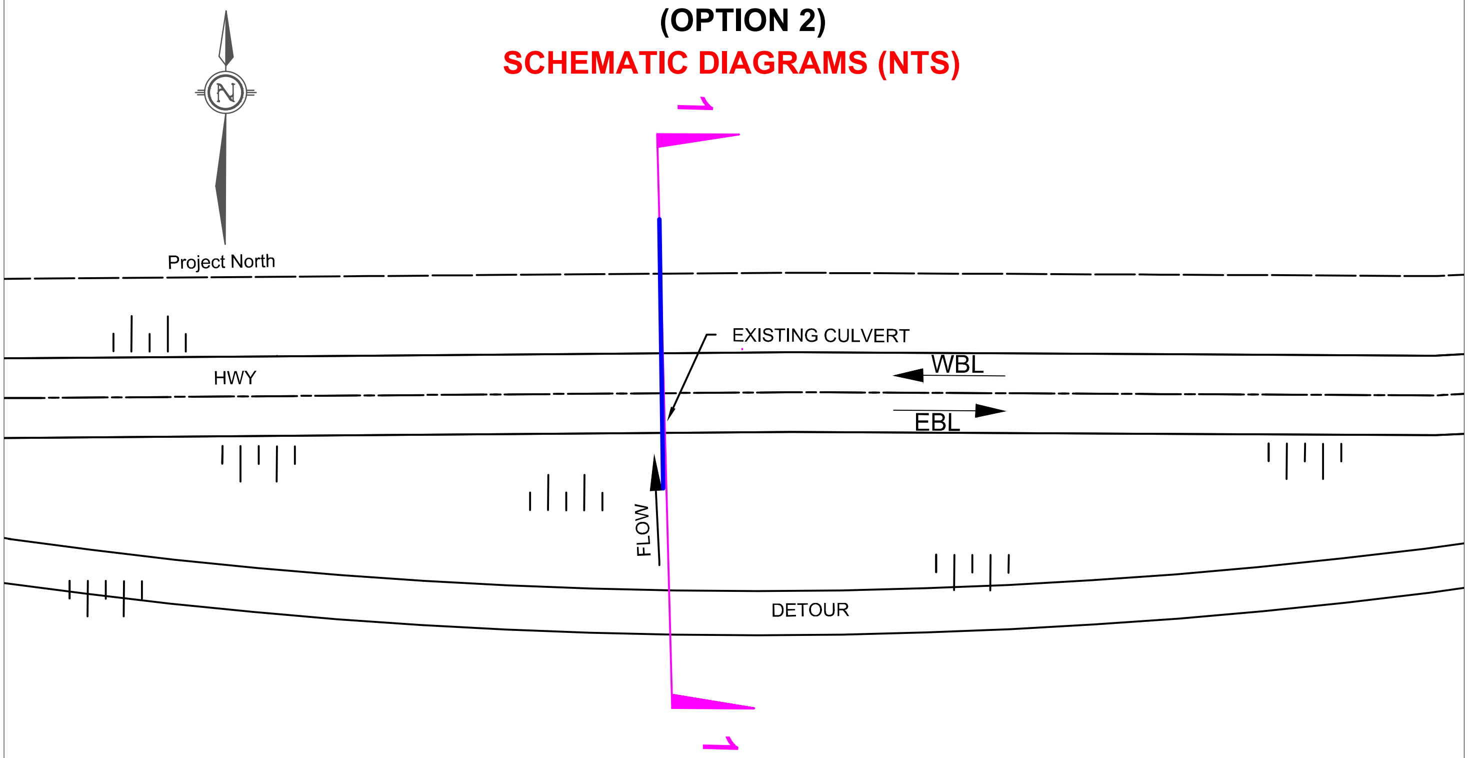
SECTION 1-1



SECTION 2-2

FIGURE H.2: TEMPORARY LOCAL DETOUR AND OPEN CUT UNSUPPORTED EXCAVATION (OPTION 2)

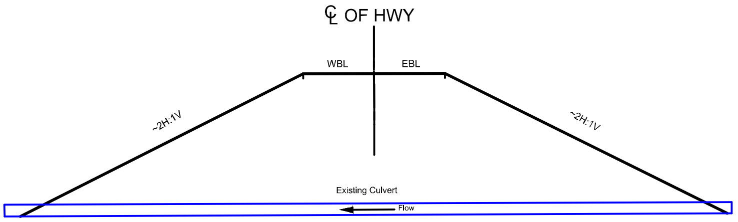
SCHEMATIC DIAGRAMS (NTS)



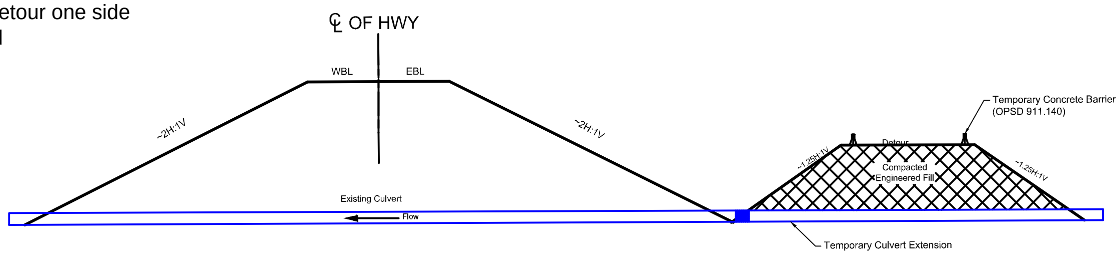
PLAN

RECOMMENDED STAGES

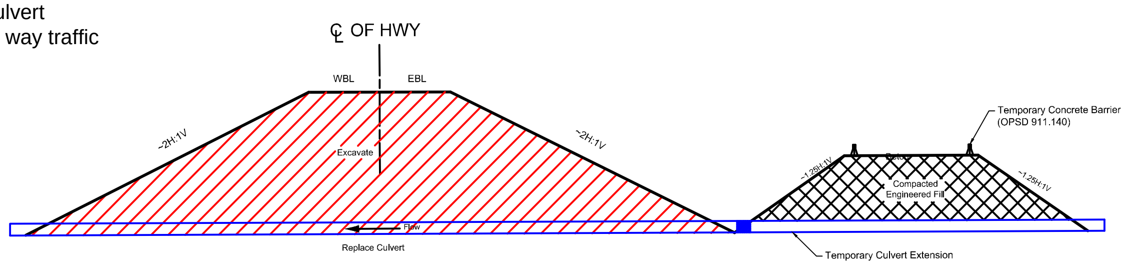
1.0 Stage 1 - Current condition



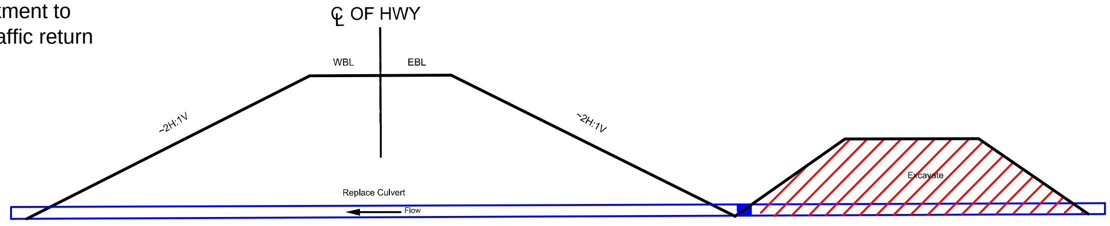
2.0 Stage 2 - Build temporary detour one side
Two-way traffic on existing road



3.0 Stage 3 - Excavation and culvert construction on other side; Two way traffic shiftted to detour



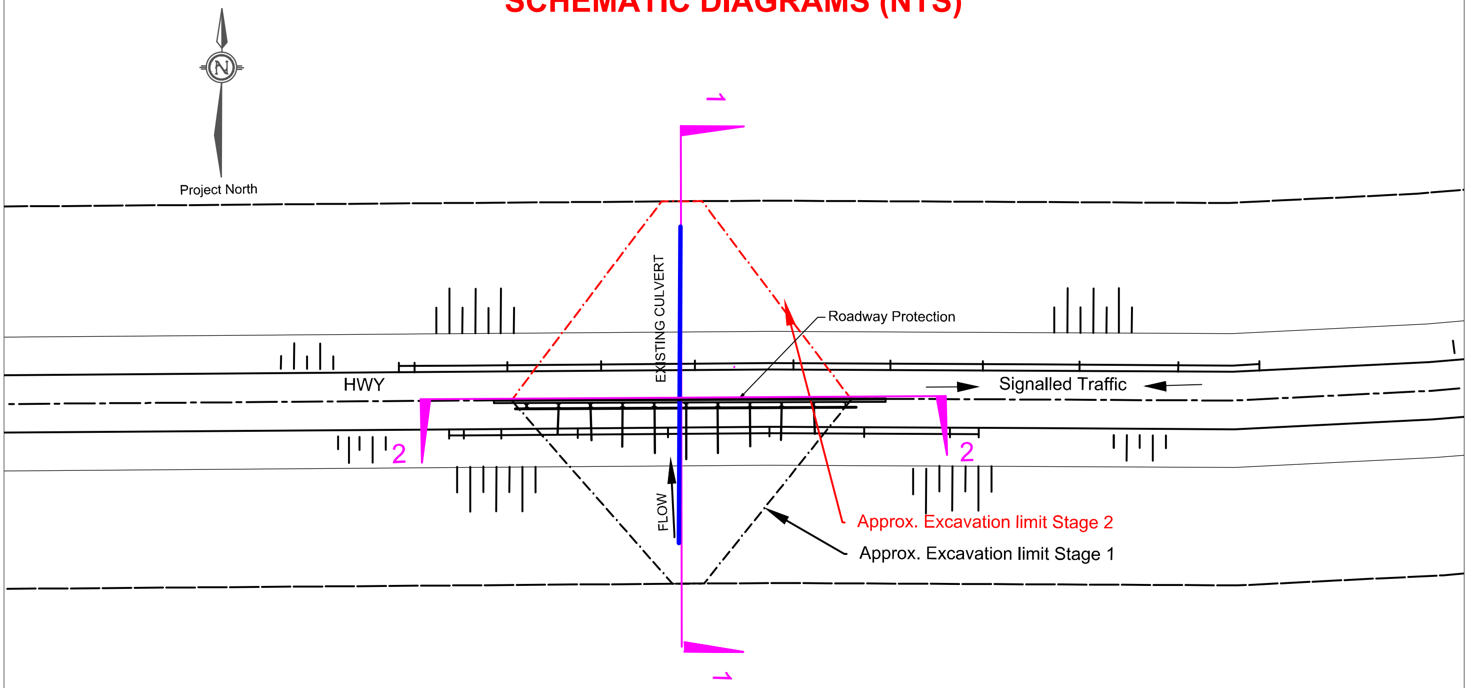
4.0 Stage 4 - Build the embankment to existing alignment; Two-way traffic return



SECTION 1-1

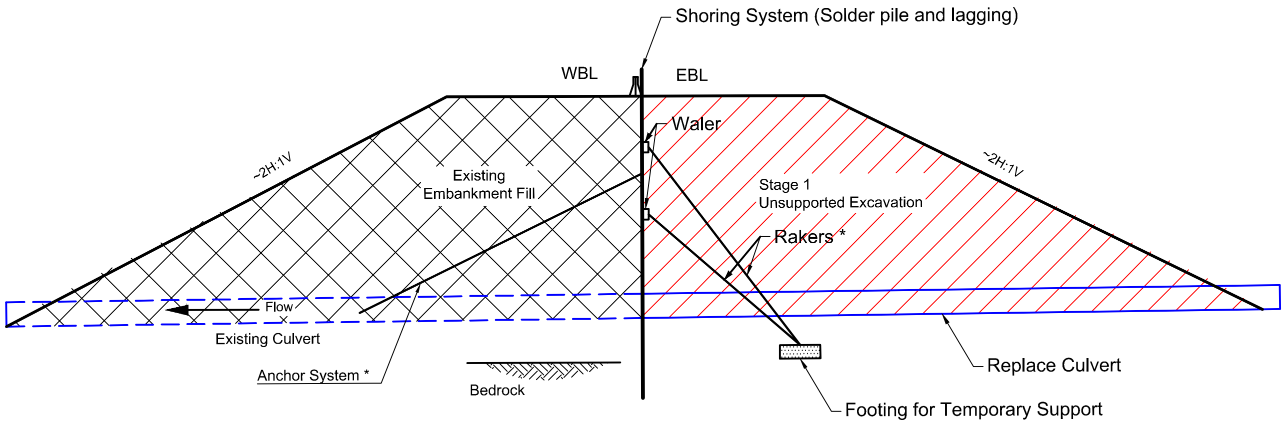
FIGURE H.3.A: HALF AND HALF CONSTRUCTION WITH UNSUPPORTED CUT SIDES

SCHEMATIC DIAGRAMS (NTS)



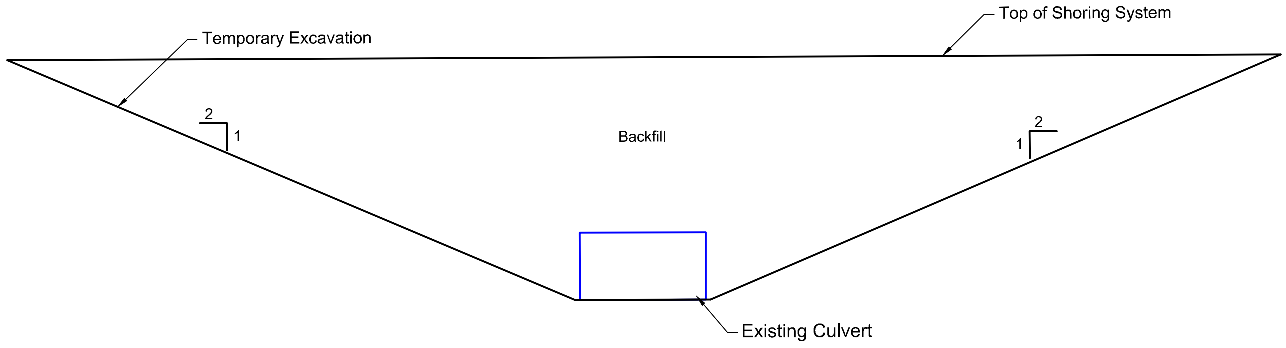
PLAN

Half and Half Construction, Shoring system with either rakers or anchor system - Unsupported Excavation



* Rakers or Anchor System

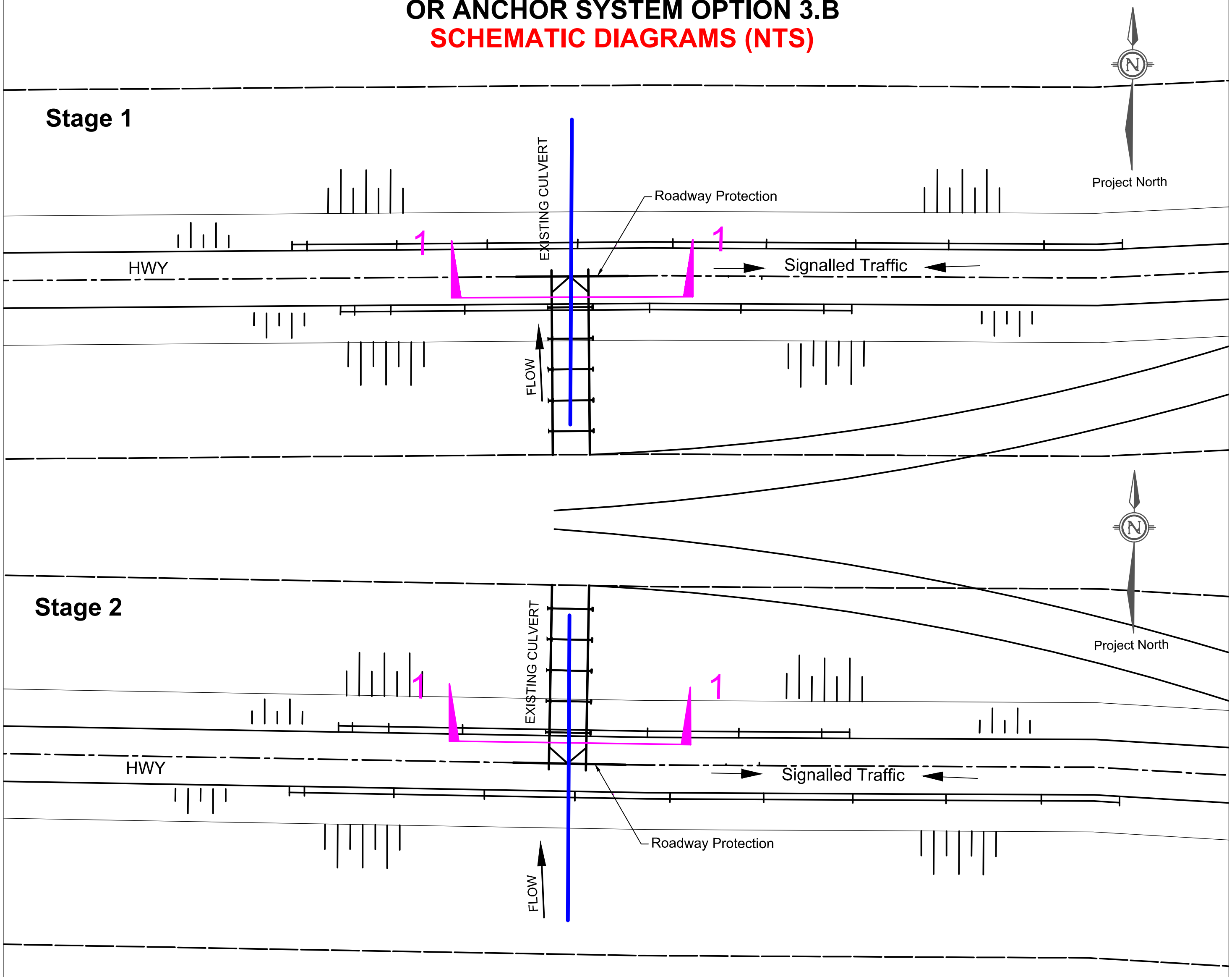
SECTION 1-1



SECTION 2-2

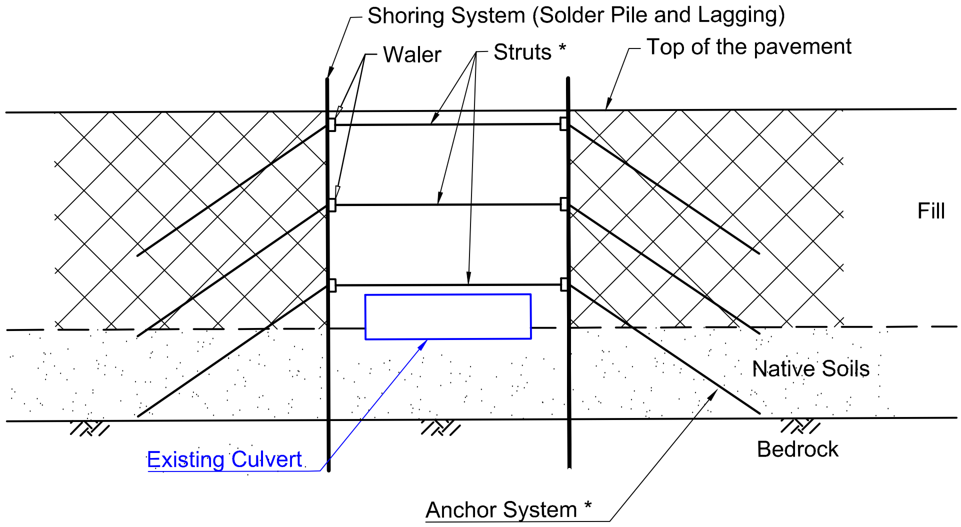
FIGURE H.3.B: HALF AND HALF CONSTRUCTION WITH BRACED CUT SIDES OR ANCHOR SYSTEM OPTION 3.B

SCHEMATIC DIAGRAMS (NTS)



PLAN

Half and Half Construction, Shoring System - Braced Cut Struts or Anchor System

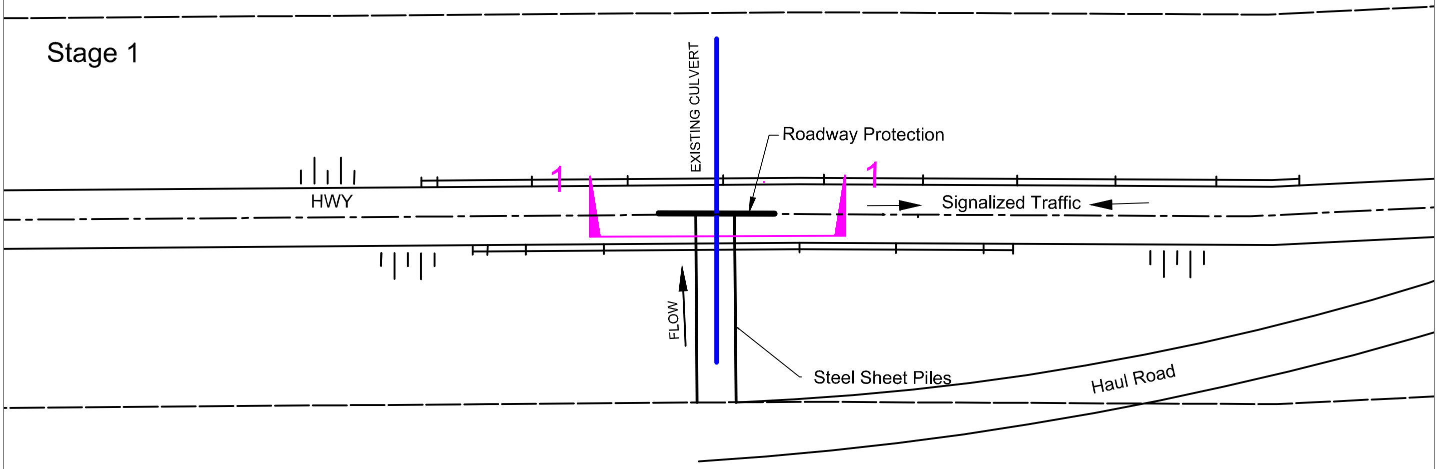


* Struts or Anchor System

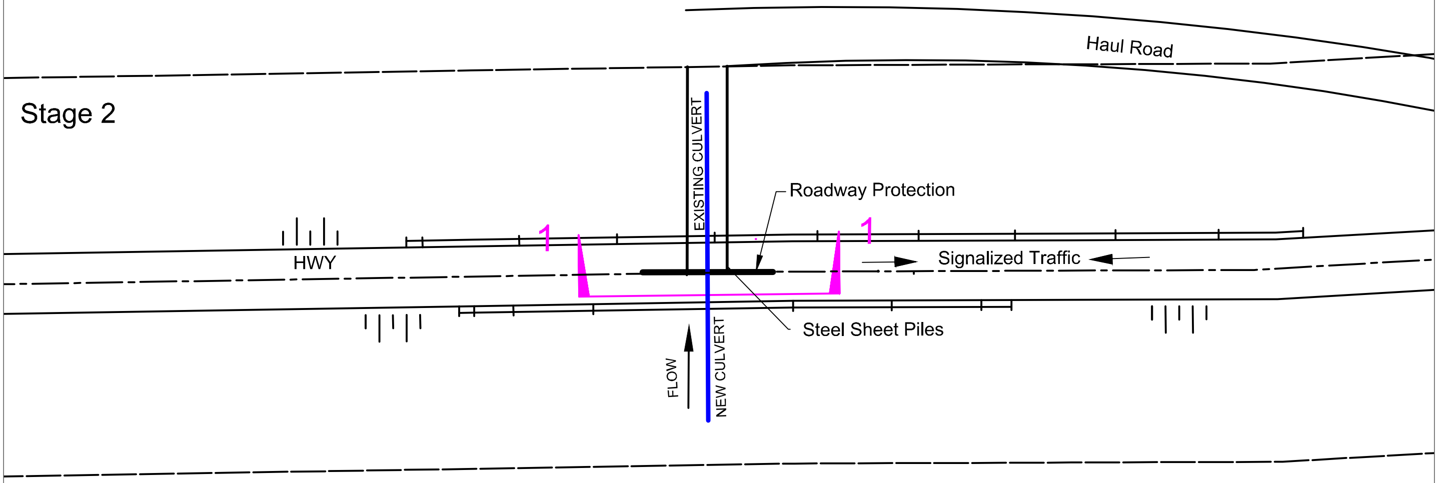
SECTION 1-1

FIGURE H.3.C: HALF AND HALF CONSTRUCTION WITH INSTALLATION OF STEEL SHEET PILE ABUTMENTS WITH PRECAST CONCRETE DECK (OPTION 3.C)

SCHEMATIC DIAGRAMS (NTS)

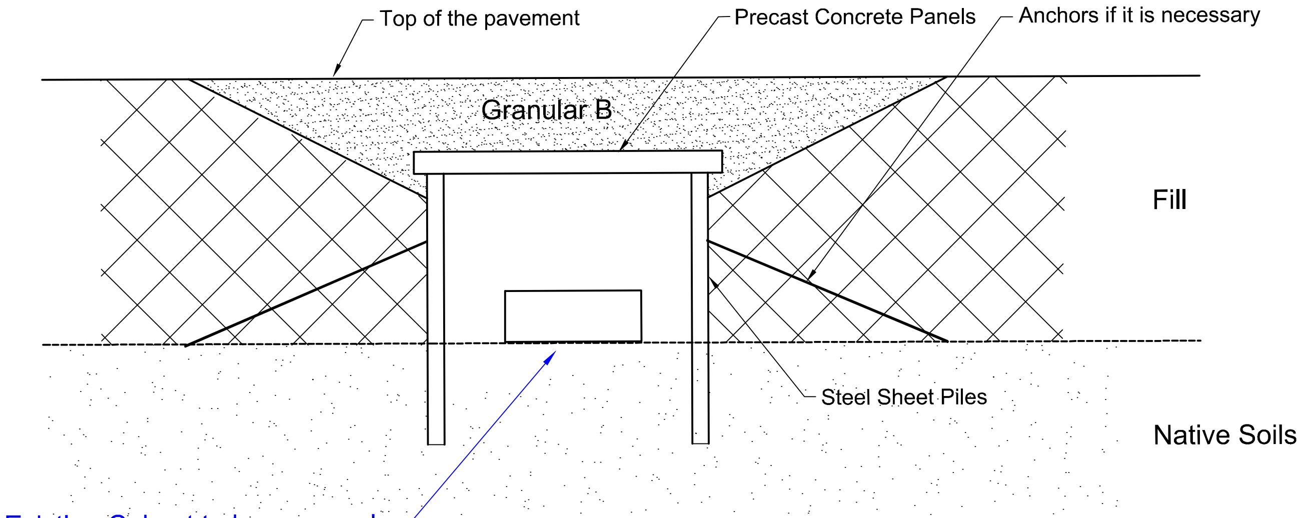


PLAN



PLAN

Half and Half Construction, Steel Sheet Pile Abutments with Precast Concrete Deck



SECTION 1-1

Appendix I-
Operational Constrains and Non Standard Special Provisions

NSSP FOR MASS CONCRETE ON BEDROCK

Scope of Work

The scope of work for the above noted tender item includes the mass concrete under the strip footings for the Flood Creek East Timber Culvert Replacement.

The Contractor should be aware that there is sloping bedrock in the area and fractured bedrock was encountered within the upper 1 m of the bedrock surface. Mass concrete volumes will vary depending on the variable intact bedrock surface.

Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS.PROV 904 "CONSTRUCTION SPECIFICATION FOR CONCRETE STRUCTURES".

Basis of Payment

Payment at the contract price for the above noted tender item includes full compensation for all labour, equipment and materials to do the required work.

NSSP FOR SLOPINIG ROCK AND COBBLES AND ROCK PIECE OBSTRUCTIONS

Scope of Work

The Contractor should be aware that there is sloping bedrock in the area and fractured bedrock was encountered within the upper 1 m of the bedrock surface. The overburden soils at the site consist of gravelly sand and silty sand fill materials which may contain cobbles and rock fragments especially near the bedrock interface.

Appropriate equipment and procedures will be required to penetrate/remove cobbles and fractured bedrock that are encountered during excavation.

Basis of Payment

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

NSSP FOR DOWELLING

Scope of Work

Work under this item is for the placement and field testing of dowels into rock.

Materials and Installation

Dowels into rock shall be constructed in accordance with OPSS.PROV 904 “CONSTRUCTION SPECIFICATION FOR CONCRETE STRUCTURES”. All reinforcing steel supplied shall be in accordance with OPSS.PROV 1440 “MATERIAL SPECIFICATION FOR STEEL REINFORCEMENT FOR CONCRETE” (dowel bars conforming to CSA Standard CSAG30.18, Grade 400).

Where dowels are to be placed in rock, holes shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete (or at least 25 MPa at 28 days).

If the hole contains water, the contractor shall remove the water otherwise a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be installed into the hole after the grout has been placed and while it is still fresh.

Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D 3689-90 and ASTM D 114381 (Re-approved 1994). Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

Performance tests shall be carried out on two rock dowels to confirm that the design load of the rock can be achieved. Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the dowel displacement shall be carried out at each load step in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4	3-1	3-2
% of Design Load	50	75	25	50	75	100	25	50	75

Cycle-Step	3-3	3-4	3-5
% of Design Load	100	110	25

Displacement measurements shall be carried out at each load step using displacement gauges with precision of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which do not meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, three additional rock dowels shall be tested at or near the same footing location as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-testing Institute (1985) as follows:

The dowels are acceptable if the total elastic movement is greater than 80% of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50% of the bond length.

Basis of Payment

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

NSSP FOR CONDITION SURVEYS AND MONITORING DURING ANY BLASTING

Scope of Work

If any blasting is required, the Blast Contractor must be fully qualified and experienced. The Blast Contractor shall outline the procedure and extent of the pre-blast survey. The blast methodology, including drill hole patterns, hole size and depths, size of blast, explosive and initiation product details, as well as all blast control procedures shell be required. Blast control procedures would include details on controlling flyrock, temporary road closures, blast signaling and site clearing procedures. Details on instrumentation, number and location of monitoring sites, blast recording and reporting procedures, and procedures to be followed in the event of excessive vibration readings are required as well.

Instrumentation or monitoring ground and air vibration effects from the blasting should be set up in accordance with the International Society of Explosives Engineering field practice guidelines (1999).

Basis of Payment

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