www.gemtec.ca

Submitted to:

McIntosh Perry
1-1329 Garders Road
Kingston, Ontario
K7P 0L8

# Foundation Investigation and Design Report
# Replacement of Culvert No. 8
# Station 14+900, Highway 11

District of Muskoka, Ontario
G.W.P.: 5461-09-00

January 18, 2024
GEMTEC Project: 102944.001
Geocres No.: 31D14-001
Latitude: 44.81946
Longitude: -79.32379

experience • knowledge • integrity

## TABLE OF CONTENTS

## LIST OF TABLES



## LIST OF DRAWINGS



## LIST OF APPENDICES



## DISTRIBUTION

**1 ELECTRONIC COPY TO MCINTOSH PERRY CONSULTING ENGINEERS LTD.**
**1 ELECTRONIC COPY TO THE MINISTRY OF TRANSPORTATION ONTARIO**

# PART A - FOUNDATION INVESTIGATION

**Foundation Investigation and Design Report**
**Replacement of Culvert No. 8**
**Station 14+900, Highway 11**
District of Muskoka, Ontario
G.W.P.: 5461-09-00

## 1.0 INTRODUCTION

GEMTEC has been retained by McIntosh Perry Consulting Engineers Ltd. (McIntosh Perry), on behalf of the Ministry of Transportation, Ontario (MTO), to carry out a foundation investigation associated with the resurfacing of Highway 11 between Severn River Bridge and Kahshe Lake Road in the Township of Morrison, District of Muskoka, Ontario (Assignment number 5019-E-0002).

This report presents the results of the foundation investigation carried out for the proposed replacement of Culvert No. 8 located beneath both the north bound lane (N.B.L.) and the south bound lane (S.B.L.) of Highway 11 in the Township of Morrison, District of Muskoka, Ontario, (G.W.P. 5461-19-00).

The scope of work for the foundation engineering services associated with the replacement was outlined in GEMTEC's Proposal dated August 17, 2023. The investigation requirements were provided by McIntosh Perry.

The work has been carried out in accordance with GEMTEC's Quality Control Plan for foundation engineering services for this project.

## 2.0 SITE DESCRIPTION AND GEOLOGY

### 2.1 Site Description

The site is located on Highway 11 approximately 0.25 kilometre(s) north of the Rainbow Circle interchange. At the site, Highway 11 runs approximately north-south and the tributary creek, east-west.

The land adjacent to the site is generally undulating to rolling and consists mainly of forested area. A residential home is located adjacent to the culvert location, along the west side of Highway 11. Occasional trees and shrubs are present along the existing highway right-of-way and the tributary creek. Bedrock outcrops are visible on both the east and west sides of the right-of-way.

The existing Highway 11 in the vicinity of the site is a divided highway with two travelled lanes in both the N.B.L. and S.B.L., gravel shoulders and a posted speed limit of 90 km/hr.

#### 2.1.1 Existing Culvert

Preliminary information provided by McIntosh Perry for this assignment indicates the existing culvert is a non-rigid frame box culvert with an unknown date of construction. The existing culvert is installed in a non-linear arrangement (one bend located near the west shoulder of the highway) with a total alignment length of approximately 44 m, a clear span of 0.9 m, and a rise of 0.6 m. The invert elevation of the existing culvert is at about 227.7 m at the east end (inlet) and 227.5 m at the west end (outlet) and the creek flows from the east to the west. The top of pavement elevation of Highway 11 at the N.B.L. and S.B.L. centrelines, in the vicinity of the existing culvert, is about 231.0 and 231.5 m, respectively. The top of the existing culvert at the N.B.L. and S.B.L. centreline is about 228.5 m and 228.2 m, respectively, corresponding to a cover of up to about 3.3 m.

The embankment sides are sloped at approximately 2.5H:1V and did not show any visible signs of distress at the time of the investigation, where visible given the thick vegetation.

Photographs showing the existing site conditions at the time of the field investigation are included in Appendix E for reference.

### 2.1.2 Proposed Culvert

At the time of reporting, limited information was available regarding the proposed culvert. It is understood that the proposed culvert will not be installed within the alignment of the existing culvert, but rather will be installed at an approximate sixty (60) degree skew to the Highway 11 alignment, and directly link the current inlet and outlet locations of the existing culvert. Therefore, it is assumed that the proposed culvert will be installed at a similar elevation as the existing culvert.

Based on the site plan details provided to GEMTEC, it is estimated that the length of the proposed culvert will be about 40 m. The staging drawings provided to GEMTEC indicate a 1,600 mm diameter CSP is being considered for the replacement.

It is also understood that both trenchless and open-cut installation techniques are being considered.

## 2.2 Regional Geology

As delineated in The Physiography of Southern Ontario[1], this section of Highway 11 lies along the borders of the minor physiographic regions known as the Number 11 Strip and the Georgian Bay Fringe, which lie within the major physiographic region of the Laurentian Highlands.

The Number 11 Strip region is characterized by a relatively flat sand plain positioned between two bedrock dominated upland areas. The sand is potentially glaciofluvial, glaciolacustrine, and /or subaqueous fan deposits in origin, deposited during the Lake Algonquin phase.

The Georgian Bay Fringe region is characterized by thinly till-covered rock knobs and ridges, with common occurrences of outcrops of bare rock. Low lying areas between the outcrops are covered by a thin layer of glacial till composed of loose, reddish, stony sandy till derived primarily from Precambrian bedrock material. The lowland areas were subsequently inundated by glacial Lake

[1.] Chapman, L. J. and Putnam, D. F., 1984. The Physiography of Southern Ontario, Ontario Geological Survey. Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000. Ontario Ministry of Natural Resources.

Algonquin resulting in the deposition of thin glaciolacustrine sediments over the till. The combined thickness of the till and glaciolacustrine sediments rarely exceeds 2 to 3 m. Other low-lying areas may be filled with organic-rich sediments.

## 3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the culvert replacement was carried out between October 2 and 4, 2023, during which time, 3 boreholes (numbered 23-01 to 23-03) were advanced at the locations indicated below and as shown on Drawing 1:

- Boreholes 23-01 and 23-03, along the alignment of the proposed culvert, on the south bound shoulder and the north bound shoulder, respectively.
- Borehole 23-02 on the interior S.B.L., adjacent the centreline barrier.

Boreholes 23-01 and 23-03 were advanced from road level at the shoulders, due to the limited access with appropriate drilling equipment to the toe of embankment areas (i.e., at the planned inlet and outlet).

The boreholes were advanced through the roadway to depths ranging from about 9.8 to 10.9 m (i.e., Elevations 221.6 to 220.3 m) below ground surface, using 108 mm inside diameter (200 mm outside diameter) continuous-flight hollow-stem augers on a truck-mounted drill rig, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario.

At Boreholes 23-01 and 23-02, upon encountering auger refusal, approximately 5.0 and 3.2 m of bedrock was cored, respectively, to final depths of 9.8 m and 10.9 m, using rotary diamond drilling techniques while retrieving HQ sized bedrock core. Borehole 23-03 was terminated within the overburden at a depth of about 10.5 m below existing ground surface.

Samples of overburden in the boreholes were generally obtained at vertical intervals of about 0.76 m using a 35 mm inside diameter (50 mm outer diameter) split-spoon sampler in general accordance with ASTM D1586 - Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils.

Traffic control was provided throughout the duration of the field work in accordance with the Ontario Traffic Manual, Book 7, Temporary Conditions.

A monitoring well was installed in both boreholes 23-01 and 23-03 to measure the groundwater level at the site. The wells consist of a 50 mm diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within sand backfill and sealed by a section of bentonite backfill. The water levels in the monitoring wells were measured on October 5, 2023 (i.e., 2 to 3 days following installation). The monitoring wells were decommissioned on December 21, 2023, in accordance with Ontario Regulation 903, as amended.

Borehole 23-03 was backfilled with bentonite pellets mixed with native soils in the overburden. The site conditions were restored following completion of work which included the resurfacing of the pavement surface using cold-patch asphalt for boreholes advanced through the pavement.

The field work was supervised by a member of GEMTEC's technical and engineering staff, who located the boreholes and supervised the drilling, sampling, and in situ testing operations, logged the boreholes, and examined and cared for the soil samples retrieved. The soil samples were identified in the field, placed in appropriate containers, and transported to GEMTEC's laboratory in Ottawa for further examination. Index and classification tests consisting of water content determinations and grain size distribution were carried out on selected soil samples. Unconfined compression strength (UCS) testing was carried out on selected rock samples at GEMTEC's laboratory. The laboratory tests were carried out in accordance with MTO and/or ASTM Standards, as appropriate.

One selected sample of soil from Borehole 23-02 was sent to Paracel Laboratories Ltd. for basic chemical analysis related to potential corrosion of buried steel elements and sulfate attack on buried concrete elements (corrosion and sulphate attack).

Classification of the rock mass quality of the bedrock core samples with respect to the Rock Quality Designation (RQD) and Uniaxial Compressive Strength (UCS) are described based on Table 4.26 and Table 4.21, respectively, of the Canadian Foundation Engineering Manual (CFEM, 2023[2]). The degree of weathering of the bedrock samples (i.e., fresh to slightly weathered) and the strength classification of the intact rock mass, based on field identification (i.e., strong to very strong), are described in accordance with Table B.3 and Table B.6, respectively, of the International Society for Rock Mechanics (ISRM[3]) standard classification system.

Following drilling, the borehole locations were surveyed by GEMTEC personnel using a Spectra SP60 GPS unit that has a $\pm$2 cm horizontal and a $\pm$3 cm vertical accuracy. The borehole locations, including NAD83 MTM Zone 10 northing and easting coordinates and ground surface elevations referenced to Geodetic datum (CGVD28), are shown on Drawing 1 and are summarized in Table 1.

[2] Canadian Geotechnical Society, 2023. Canadian Foundation Engineering Manual, 5th Edition.

[3] International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

**Table 1 – Summary of Borehole Locations**

| Borehole No. | Location | NAD83 2010.0 MTM Zone 10 | | Ground Surface Elevation (m) | Borehole Depth (m) |
|---|---|---|---|---|---|
| | | Northing (m) | Easting (m) | | |
| 23-01 | SB Shoulder | 4964380.098 | 318723.965 | 231.37 | 9.80 |
| 23-02 | SB – median lane | 4964387.617 | 318733.118 | 231.23 | 10.90 |
| 23-03 | NB Shoulder | 4964389.074 | 318744.655 | 230.83 | 10.50 |

## 4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

### 4.1 General

The subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ testing from the current investigation are given on the Record of Borehole sheets presented in Appendix A. The results of the laboratory testing carried out during the current investigation are presented on the Record of Borehole sheets as well as on Figures B1 to B4 in Appendix B. The borehole locations and the interpreted stratigraphic profile projected along the proposed alignment of the planned culvert are provided on Drawing 1.

Photographs of bedrock core samples are provided in Appendix C. The results of basic chemical analysis completed on the selected soil sample are provided in Appendix D. Site photographs showing the general conditions at the site are presented in Appendix E.

The stratigraphic boundaries shown on the borehole sheets and on the interpreted stratigraphic section from Drawing 1 are inferred from observations of drilling progress and non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

### 4.2 Site Stratigraphy Summary

In general, the site stratigraphy consists of embankment fill over bedrock or native deposits of silty clay, silty sand and sandy silt, which in boreholes 23-01 and 23-02 are overlying relatively shallow bedrock. Relatively thick deposits of peat were encountered between the embankment fill and the native soils and bedrock at all the borehole locations. Boreholes 23-01 and 23-02 were terminated within the bedrock and borehole 23-03 was terminated in the silty sand/sandy silt deposits.

### 4.3 Surficial and Embankment Materials

An asphaltic concrete layer ranging in thickness from about 50 mm to 60 mm was encountered at ground surface in boreholes 23-01 and 23-02.

Below the asphaltic concrete at boreholes 23-01 and 23-02, and from ground surface at borehole 23-3, a layer of sand and gravel (base) ranging in thickness between about 130 to 150 mm was encountered.

Granular fill was observed in all three boreholes below the base layer and extends to depths ranging from about 2.8 m to 4.1 m below ground surface. The granular fill can generally be described as sand with varying amounts of gravel and silt. Organic material, including roots, was occasionally observed at lower depths within the layer, as well as cobbles and boulders.

Standard penetration tests carried out in the granular fill gave N values ranging between about 1 and 47 blows per 0.3 m of penetration, which reflect a very loose to dense compactness.

Blast rock was encountered about 1.7 m below ground surface at borehole 23-01, based on the drilling resistance, and it extends to a depth of at least 2.7 m.

The moisture content of the five samples of fill tested ranged between about 2 and 36 percent. The results of grain size analysis testing carried out the five samples of this material are provided on Figure B1 in Appendix B and are summarized in the table below.

**Table 2 – Summary of Grain Size Distribution Testing on Embankment Fill**

<table>
<tr><th>Test Hole</th><th>Sample Number</th><th>Sample Depth (m)</th><th>Gravel (%)</th><th>Sand (%)</th><th>Silt (%)</th><th>Clay (%)</th></tr>
<tr><td>23-01</td><td>A[1]</td><td>0.06 to 0.2</td><td>28</td><td>67</td><td colspan="2">5</td></tr>
<tr><td>23-01</td><td>5</td><td>3.1 to 3.7</td><td>29</td><td>57</td><td>12</td><td>2</td></tr>
<tr><td>23-02</td><td>1</td><td>0.3 to 0.5</td><td>2</td><td>89</td><td colspan="2">9</td></tr>
<tr><td>23-02</td><td>4</td><td>2.4 to 2.7</td><td>1</td><td>91</td><td>6</td><td>2</td></tr>
<tr><td>23-03</td><td>2</td><td>0.8 to 1.4</td><td>18</td><td>72</td><td colspan="2">10</td></tr>
</table>

1. Sample 'A' represents a grab sample that was collected from within the borehole by removing the augers from the borehole (upon reaching a depth of 1.5 m) to allow for a visual assessment of the interior walls of the borehole.

### 4.4 Peat

Deposits of peat, ranging from about 0.5 to 0.8 m in thickness, were encountered in all three boreholes below the embankment fill. The peat deposits extend to depths ranging from 3.3 m to 4.8 m below ground surface. The peat is mainly amorphous at all three locations and includes traces of sand with some silt and clay content, as well as traces of wood pieces and other organic material (e.g. roots/rootlets). At borehole 23-03, the upper portion of the deposit appears to be more fibrous.

### 4.5 Silty Clay

A deposit of silty clay was encountered below the peat at borehole 23-03 at a depth of about 3.3 metres and extends to a depth of about 4.6 m below ground surface, therefore having a thickness of about 1.3 metres. The deposit consists of grey clay with silty sand seams.

A standard penetration test carried out in the silty clay gave an N value of 13 blows per 0.3 m of penetration, which reflect a stiff to very stiff consistency.

The measured moisture content of one sample tested was 23 percent. The results of grain size analysis testing carried out on a sample of this material are provided on Figure B2 in Appendix B and are summarized in the table below.

**Table 3 – Summary of Grain Size Distribution Testing on Silty Clay**

| Test Hole | Sample Number | Sample Depth (m) | Gravel (%) | Sand (%) | Silt (%) | Clay (%) |
|---|---|---|---|---|---|---|
| 23-03 | 6 | 3.8 to 4.4 | 0 | 28 | 50 | 23 |

### 4.6 Sand and Silty Sand to Sandy Silt

Sands and silts with varying amounts of clay and traces of gravel were encountered below the peat in boreholes 23-02 and 23-03 at elevations 226.7 and 226.2 m, respectively. The thickness of the sand layer at borehole 23-02 is about 2.3 m. Borehole 23-03 was terminated in the sandy deposit at a depth of 10.5 m. The SPT N values ranged from 3 to 18 blows per 0.3 m of penetration, indicating a very loose to compact state of compaction.

The measured moisture content of the samples tested ranged from 22 to 37 percent. The results of grain size analysis testing carried out on four samples of this material are provided on Figure B3 in Appendix B and are summarized in the table below.

**Table 4 – Summary of Grain Size Distribution Testing on Sand to Silty Sand**

| Test Hole | Sample Number | Sample Depth (m) | Gravel (%) | Sand (%) | Silt (%) | Clay (%) |
|---|---|---|---|---|---|---|
| 23-02 | 7 | 4.6 to 5.2 | 0 | 45 | 43 | 12 |
| 23-02 | 9 | 6.1 to 6.7 | 0 | 85 | 9 | 6 |
| 23-03 | 9 | 6.1 to 6.7 | 0 | 21 | 77 | 2 |

| Test Hole | Sample Number | Sample Depth (m) | Gravel (%) | Sand (%) | Silt (%) | Clay (%) |
|---|---|---|---|---|---|---|
| 23-03 | 11 | 7.6 to 8.2 | 0 | 64 | 34 | 2 |

### 4.7 Gravelly Sand

A deposit of gravelly sand with trace to some silt was encountered beneath the sands and silts in borehole 23-02. The deposit was fully penetrated and has a thickness of about 0.8 m.

An SPT ‘N’ value obtained within the deposit was 27 blows per 0.3 m of penetration, indicating a compact state of compaction.

### 4.8 Bedrock

Precambrian bedrock was encountered below the peat deposit in Borehole 23-01 and below the gravelly sand at borehole 23-02, at depths of about 4.8 m and 7.7 m below existing ground surface, respectively (i.e., elevations 226.6 and 223.5, respectively) and was cored for lengths of about 5.0 and 3.2 m, respectively, using HQ sized drilling equipment. The retrieved bedrock core was described as fresh, black, grey, and pink Gneiss, as presented on the Record of Borehole sheets in Appendix A.

The following table summarizes the bedrock surface depths and elevations as encountered at the borehole locations during the current investigation.

**Table 5 – Summary of Bedrock Depths and Elevations**

| Borehole Number | Borehole Location | Existing Ground Surface Elevation(m) | Depth to Bedrock Surface (m) | Bedrock Surface Elevation (m) |
|---|---|---|---|---|
| 23-01 | Culvert Outlet | 231.4 | 4.8 | 226.6 |
| 23-02 | Culvert Middle | 231.2 | 7.7 | 223.5 |

The Rock Quality Designation (RQD) values generally ranged from about 82 to 100 percent, indicating a rock mass of good to excellent quality.

Laboratory Uniaxial Compression Strength (UCS) tests were carried out on selected bedrock core samples and the measured UCS values ranged from 52 to 98 MPa, indicating the bedrock is strong. The results of the UCS testing are provided on Figure B4 in Appendix B. Photographs of core samples of the bedrock are provided in Appendix C.

### 4.9 Groundwater Conditions

A monitoring well was installed within the overburden in each of Boreholes 23-01 and 23-03 to measure the stabilized groundwater level at the site. The groundwater levels measured in the monitoring wells are presented in the table below.

**Table 6 – Summary of Groundwater Level Depths and Elevations**

| Borehole Number | Existing Ground Surface Elevation (m) | Depth to Groundwater Level (m) | Groundwater Elevation (m) | Date of Reading |
|---|---|---|---|---|
| 23-01 | 231.4 | 2.9 | 228.5 | October 5, 2023 |
| 23-01 | 231.4 | 2.5 | 228.9 | December 21, 2023 |
| 23-03 | 230.8 | 2.1 | 228.7 | October 5, 2023 |
| 23-03 | 230.8 | 1.5 | 229.3 | December 21, 2023 |

It should be noted that the groundwater levels at this site are expected to fluctuate seasonally in response to changes in precipitation and snow melt and are expected to be higher during the spring and periods of precipitation.

### 4.10 Steel Corrosion and Sulphate Attack, Chemical Analysis

One soil sample was submitted to Paracel Laboratories Ltd. for chemical analysis related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The test results are provided in Appendix D and are summarized in the table below.

**Table 7 – Summary of Chemical Analysis Testing**

| BH No. | Sample No. | Sample Depth (m) | Sample Type | Chloride (%) | Sulphate (%) | Electrical Conductivity (mS/cm) | Resistivity (ohm-cm) | pH |
|---|---|---|---|---|---|---|---|---|
| 23-02 | 5 | 3.1 – 3.7 | Soil | 0.045 | 0.006 | 0.995 | 1000 | 6.72 |

## 5.0 CLOSURE

The field work for this assignment was supervised by Mr. Adrian North. This report was prepared by Matthew Rainville, C.E.T. and Serge Bourque, M.Sc.E., P.Eng., and reviewed by Mr. William Cavers, P.Eng., a Principal Geotechnical Engineer with GEMTEC and the Key MTO Foundations Personnel for this project.

**GEMTEC Consulting Engineers and Scientists Limited**

Matthew Rainville, C.E.T.
Senior Technologist


Serge Bourque, M.Sc.E., P.Eng.
Principal Geotechnical Engineer


William Cavers, P.Eng.
Principal Geotechnical Engineer

MR/SB/WC

# METRIC

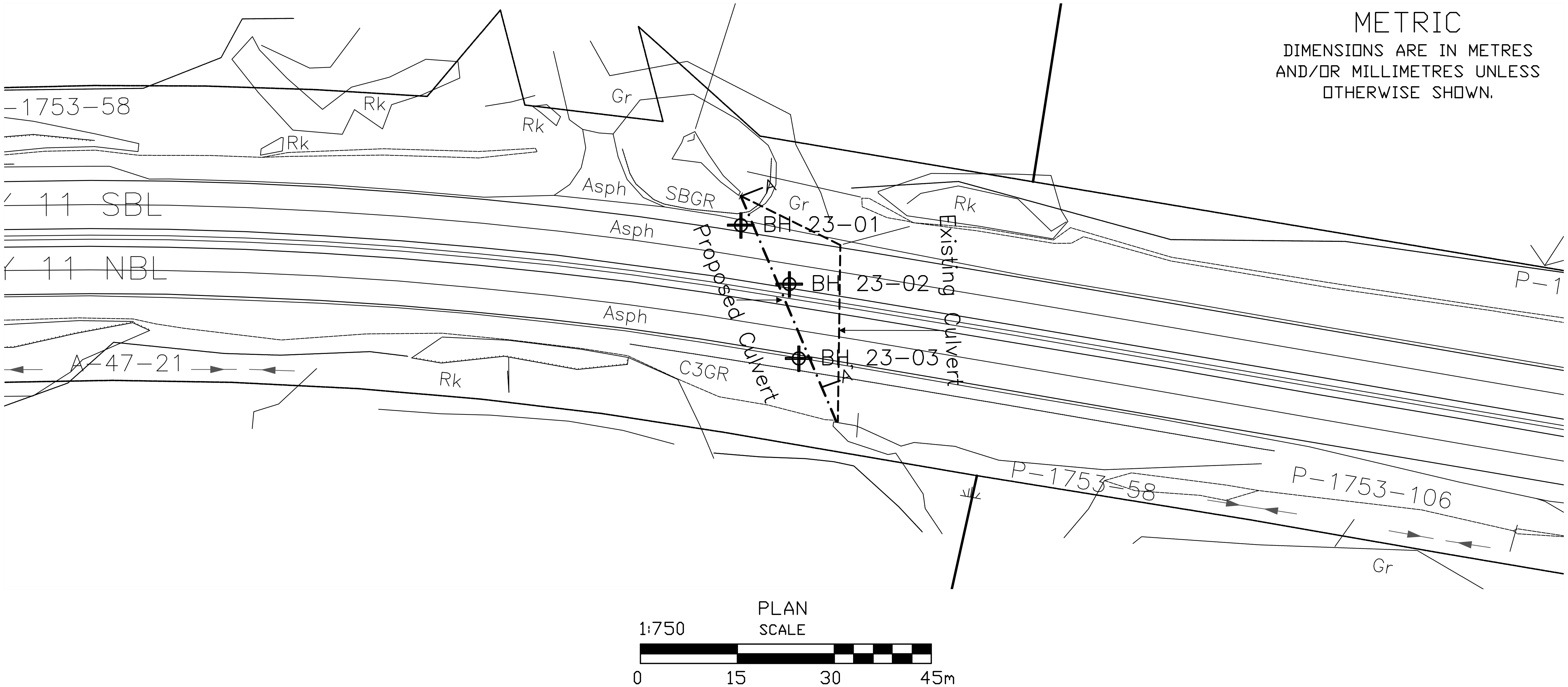
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN.

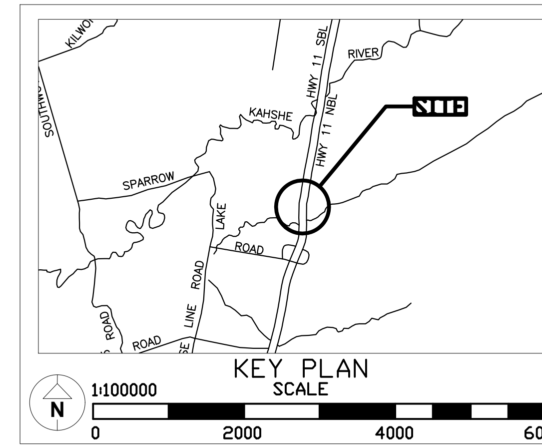
GWP No: 5461-09-00

Replacement of Culvert No. 8
Station 14+900
Borehole Locations and Soil Strata

SHEET 01

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

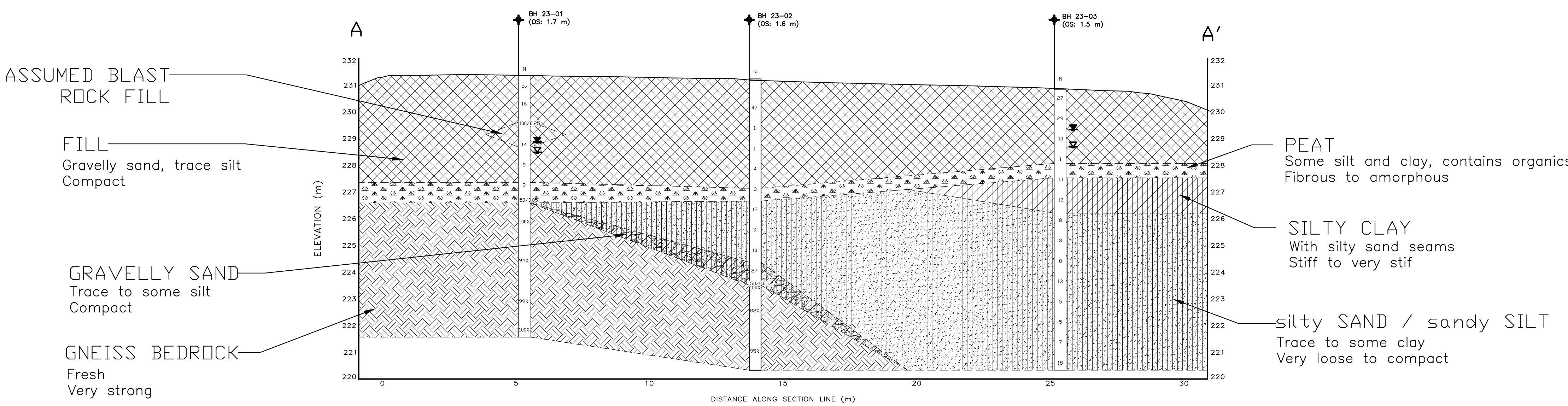



## LEGEND

| | |
|---|---|
| ⌖ | BOREHOLE – CURRENT INVESTIGATION |
| OS | OFFSET FROM SECTION LINE |
| N | STANDARD PENETRATION TEST VALUE |
| #X | BLOW COUNT |
| X% | ROCK QUALITY DESIGNATION (RQD) |
| ▽ | GROUNDWATER LEVEL (OCTOBER 5, 2023) |
| ▼ | GROUNDWATER LEVEL (DECEMBER 21, 2023) |

| BOREHOLE CO-ORDINATES: NAD83 (CSRS) / MTM ZONE 10 | | | |
|---|---|---|---|
| BH ID | ELEVATION | NORTHING | EASTING |
| BH 23-01 | 231.4 | 4964380.098 | 318723.965 |
| BH 23-02 | 231.2 | 4964387.617 | 318733.118 |
| BH 23-03 | 230.8 | 4964389.074 | 318744.655 |




CROSS SECTION A-A'
1:200 SCALE
0 4 8 12m

## NOTES

1. The Boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2. This drawing is for subsurface information only. the proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

## REFERENCES

Base plans provided in digital format by McIntosh Perry, drawing labeled: WP 5461-09-00 -Hwy 11 Severn-Kahshe WIP-2020-03-2.dwg

| REVISIONS | DATE | BY | DESCRIPTION |
|---|---|---|---|
| | | | |

GEOCRES No: 31D14-001

| | | | |
|---|---|---|---|
| HWY: 11 | | PROJECT No: 102944.001 | |
| SUBM'D: M.R. | CHKD: M.R. | DATE: 01/12/2024 | SITE: - |
| DRAWN: S.J. | CHKD: B.W. | APP'D: W.C. | DWG: 01 |

# PART B - FOUNDATION DESIGN REPORT

**Foundation Investigation and Design Report**
**Replacement of Culvert No. 8**
**Station 14+900, Highway 11**
District of Muskoka, Ontario
G.W.P.: 5461-09-00

## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

### 6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the existing Culvert No. 8 on Highway 11, located in the Township of Morrison, District of Muskoka, Ontario. The recommendations provided herein are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation and are provided in accordance with the current Canadian Highway Bridge Design Code CAN/CSAS6-19 (CHBDC).

The culvert is a non-structural culvert but recommendations have been provided for associated structures (e.g., retaining walls) if required.

The foundation investigation report, discussion, and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

### 6.2 Existing Conditions

At this location, Highway 11 is a divided highway with two travel lanes in each direction separated by a concrete barrier wall along the centreline.

No previous foundation investigation information was available for the subject culvert.

Preliminary information provided by McIntosh Perry for this assignment indicates the existing culvert is a non-rigid frame box culvert with an unknown date of construction. The existing culvert is installed in a non-linear arrangement (one bend located near the west shoulder of the highway) with a total alignment length of approximately 44 m, with a clear span of 0.9 m and a rise of 0.6 m. The invert elevation of the existing culvert is about 227.7 m at the east end (inlet) and 227.5 m at the west end (outlet) and the creek flows from the east to the west. The top of pavement elevation of Highway 11 at the N.B.L. and S.B.L. centrelines, in the vicinity of the existing culvert is about 231.0 and 231.5 m, respectively. The top of the existing culvert at the N.B.L. and S.B.L. centrelines is about 228.5 m and 228.2 m, respectively, corresponding to a cover of up to about 3.3 m.

The embankment sides are sloped at approximately 2.5H:1V and did not show any visible signs of distress at the time of the investigation, where visible given the notable vegetation.

No obvious signs of foundation settlement were noted at the time of the site investigation and the existing slopes appear to be performing satisfactorily.

## 6.3 Proposed Structure

At the time of reporting, limited information was available regarding the proposed culvert. It is understood the proposed culvert will not be installed within the alignment of the existing culvert, but rather will be installed at an approximate sixty (60) degree skew to the Highway 11 alignment, and directly link (linear installation) the current inlet and outlet locations of the existing culvert. Therefore, it is assumed that the proposed culvert will be installed at a similar elevation as the existing culvert. Based on the site plan details provided to GEMTEC, it is estimated that the length of the proposed culvert will be about 40 m. It is also understood that the replacement is currently planned to consist of a 1,600 mm diameter CSP culvert.

It is anticipated the creek flow will be maintained using a dam and pump or dam and divert system during construction of the new culvert. The existing culvert may either be removed as part of the construction program or, it may be abandoned in place and filled with concrete or grout after installation of the new culvert.

## 6.1 Culvert Type/Foundation Alternatives

Selection of the culvert type must consider the proposed construction procedures, staging requirements, geotechnical resistance available in the foundation soils, depth to suitable bearing stratum and post-construction settlement criteria.

Only shallow foundation options have been considered in sufficient detail for detailed design for the replacement culvert. It is not considered to be a practical or economical option to support the new culvert on deep foundations.

Common culvert and foundation types are listed below, along with a comparison of these alternatives, from a foundation perspective. Their respective advantages and disadvantages are outlined below and are summarized in Table A following the text of this report.

### 6.1.1 Closed Box Concrete Culvert

From a geotechnical perspective, the replacement could be achieved with a closed bottom culvert. Since the base of the closed box does not need to be founded below frost depth, the base of the excavation for a closed box would be at a higher elevation than required for the footings of an open bottom culvert. The shallower excavation would have the advantages of a shorter duration for dewatering, reduced dewatering effort, construction staging and reduced material and handling.

The peat will need to be removed from within the culvert footprint as it is not suitable for support of the culvert.

Based on the existing invert elevation ranging from approximately 227.5 to 227.7 m and allowing for an assumed 150 mm thick concrete base and a 300 mm thick layer of Granular A bedding, and for removal of the peat and placement of additional bedding as required, the founding elevation is expected to be around Elevation 226.6 to 227.0. Therefore, the anticipated founding subgrade level will be on or within the bedrock loose silty sand to sandy silt or stiff to very stiff silty clay.

### 6.1.2 Open Bottom Concrete Rigid Frame Culvert

With the design stream bed elevation ranging from approximately 227.5 to 227.7 m, an open bottom culvert founded at elevations ranging from 225.8 to 226.0 m (1.7 m below top of streambed elevation) is considered feasible at this site from a foundation standpoint. At this elevation, it is expected that the replacement culvert will be founded within the bedrock (at the outlet) or loose to very loose silty sand/sandy silt deposits. In order to meet frost protection requirements, the founding elevation for the open bottom structure will be lower than for the closed box or CSP options, thereby requiring deeper excavations. The deeper excavations would require bedrock removal, increased dewatering effort, longer construction duration and result in increased material and handling costs.

### 6.1.3 Corrugated Steel Pipe (CSP)

From a foundation perspective, the replacement can also be carried out with the installation of a new steel pipe with invert elevations ranging from 227.5 to 227.7 m. A bedding layer consisting of Granular A with a thickness of 300 mm is recommended. The peat will need to be removed from within the culvert footprint as it is not suitable for support of the culvert.

The base of excavation is expected to be at an approximate elevation between 226.6 to 227.4 m, after removal of the peat. This shallower excavation (than for an open footed culvert) would have the advantages of a shorter duration for dewatering, construction staging and reduced material and handling.

## 7.0 CONSTRUCTION ALTERNATIVES

This section presents discussions from a foundation perspective on alternative replacement methods for the proposed culvert.

### 7.1 Open Cut

Based on preliminary information provided to GETMEC, it is understood that the culvert could be replaced using open cut techniques in a 3-stage approach that would keep at least one lane of traffic in each direction open throughout the construction period.

The limited details that were available to GEMTEC at the time of reporting indicate the following general sequence details of the potential staging:

- Installation of approximately 10 m of the proposed culvert beginning at the outlet end (west end). This will require closure of the outside southbound lane;
- Installation of a temporary extension at the outlet of the newly installed culvert section, followed by the installation of a temporary widening of the west side embankment. The east side embankment will be widened at this time as well;
- Installation of temporary, single lanes on each of the temporary widened embankment sections;
- Installation of approximately 20 m of culvert within the centre portion of the alignment, requiring closure of all existing, permanent lanes (2 northbound and 2 southbound) and shifting the traffic onto the temporary lanes;
- Reinstatement of roadway with centre portion of alignment, and removal of temporary lanes and embankment extensions; and,
- Installation of approximately 10 m of the proposed culvert at the inlet end (east end). This will require closure of the outside northbound lane.

Based on GEMTEC's review of the preliminary section details for the temporary embankments (traffic bypass lanes), the side slopes of the excavations are proposed to be constructed at 1 Horizontal to 1 Vertical (1H:1V).

Due to the presence of the peat (which must be removed from below the culvert), unsupported excavation side slopes of 1H to 1V will likely not be achievable (even with dewatering of the fill and sandy deposits to at least 0.5 m below the excavation floor) and flatter excavation side slopes of at least 3 Horizontal to 1 Vertical will likely be required. Where flatter side slopes cannot be provided, temporary protection systems will be required.

## 7.2 Trenchless Construction

### 7.2.1 Trenchless Technologies

The Contractor should be responsible for choosing the method and equipment for the crossing installations, unless specific methods are otherwise prohibited. Ground behaviour will be, in part, dependent on the installation method adopted and this report provides guidance on the influence of ground behaviour on possible installation methods. It should not be construed that the Contractor is restricted to the particular methods considered herein, and in the event that alternative methods are considered, the Contractor must make their own interpretation of the anticipated ground behaviour, based on the factual information from the investigations undertaken at this site.

Common trenchless construction methods include horizontal directional drilling, pipe jacking and horizontal auger boring, pipe ramming, micro-tunneling, pilot tube micro-tunneling (PTMT), tunnel boring machine (TBM) and tunnel digging machine (TDM - i.e., open face shield tunnelling). A brief description of each method is included below.

- **Horizontal Directional Drilling (HDD):** HDD involves the drilling of a pilot hole using a steerable drill bit on a flexible string of drill rods while the bore is supported using a bentonite slurry. Once the pilot hole is complete, the bore would be reamed in one or more passes to a larger diameter, and then the pipe would be pulled through the bore (using the drill rods to pull the pipe into place). HDD equipment is available for drilling in both bedrock and overburden but is very challenging in bouldery ground. Deep entrance and exit pits are generally not required, however, larger laydown areas are required to install the product pipe, and the crossing typically needs to be longer to accommodate the shallow entry and exit angles for the drilling equipment. Bores are typically limited to less than 1200 mm in diameter.
- **Pipe Jacking and Horizontal Auger Boring (also referred to as Auger Jack and Bore):** A pipe jacking operation involves pushing an oversized liner pipe (casing) horizontally into the ground by jacking. The spoil is generally removed from within the casing using an auger boring machine. The cutting head is driven by, and is positioned at, the leading end of an auger string that is established within the casing pipe. The profile needs to be approximately horizontal. Jacking and receiving pits are required. There can be limited ability to steer the casing during jacking. This method is only applicable to construction in the overburden and may not be feasible in bouldery soils (e.g., glacial till). This method is also not feasible in flowing ground. If used in mixed face condition, this method can be adopted with a small boring unit (SBU) head which can advance through soil/bedrock mixed face condition and offers some steering ability.
- **Pipe Ramming:** Pipe ramming is a trenchless method that uses a pneumatic tool to hammer a steel pipe or casing into the ground. The pipe is almost always driven ‘open’ to thereby direct the soil into the pipe interior instead of compacting it outside the pipe. The leading edge of the pipe typically has a small overcut to reduce friction between the carrier pipe and soil and to improve the load conditions on the pipe. Soil/pipe friction reduction can also be achieved with lubrication, and different types of bentonite and/or polymers can be used for this purpose. Depending on the length of the installation, the soils inside the pipe can be removed either during or after the installation by augering, compressed air or water jetting. This method is not considered feasible in mixed face conditions including sound bedrock.
- **Micro-tunnelling:** Micro-tunnelling is a method of installing pipes in bores ranging from 0.6 to greater than 3 m in diameter behind a steerable remote-controlled shield that is pressurized with a bentonitic fluid to minimize ground losses. The process is essentially remote-controlled pipe jacking where all operations are controlled from the surface, cuttings are removed by the circulating slurry, and the necessity for personnel to enter the bore is eliminated. Micro-tunnelling equipment is generally more suited to drilling in overburden.
- **Pilot Tube Micro-tunneling (PTMT):** PTMT, also known as guided auger boring, employs augers for excavation and soil removal and a jacking system for advancing the drill pipes,

casings and product pipes. The guidance system comprises a target with LEDs mounted in the steering head of the equipment that is monitored through a TV monitor. The PTMT operation includes pilot boring and reaming and, since this technique is used for smaller size pipes, the equipment and space required for this operation is smaller than what is normally required for pipe jacking or microtunnelling. PTMT can obtain an accuracy of 10 mm per 100 m of pipe length; however, the accuracy depends on the ground conditions, the accuracy of the guidance system and the operator's skill. The "pilot tube" is advanced in a similar fashion to horizontal directional drilling with a guidance system used to control alignment and grade.

- **Tunnel Boring Machine (TBM):** TBM tunnelling operations involve the advance of a steerable machine with a rotating cutter head horizontally into the ground with successive sections of either an oversized liner pipe or the final product pipe advanced behind the TBM by pipe jacking. The spoil is removed from the tunnel as the TBM is advanced, using augers, conveyor belts or mucking carts. The cutting head is driven and steered by an operator inside the TBM and may be partially open to allow for access to the face. The tunnel profile needs to be approximately horizontal. Jacking and receiving pits are required. Locally, this method is generally used for construction in overburden, and open-faced machines have been used in bouldery soils (e.g., glacial till). Excavations through sandy soils below the water table typically require dewatering to maintain face stability when using open faced machines, specialized earth pressure balance or slurry shield TBMs, which pressurize the face of the excavation and improve face stability, or the use of micro-tunnelling.
- **Tunnel Digging Machine (TDM):** TDM tunnelling, also called open-face shield tunnelling, involves excavating the soils using a hydraulic excavator arm, working within a full-circumference tunnelling shield. Alternatively, hand mining (i.e., manual excavation) within the tunnelling shield could be carried out whereby the soil would be excavated using manual equipment with workers at the face. Typically, the liner (i.e., steel casing) or final pipe would be jacked in sections from the launching shaft. Unlike jack and bore, this method allows personnel to enter the tunnel to allow more control over the operations, such as for removal of obstructions. Similar to jack and bore, however, groundwater lowering is necessary to control cohesionless soils below the groundwater level. Manual or machine-assisted excavation generally requires a tunnel diameter of about 1.2 m or more.

### 7.2.2 Assessment of Feasible Trenchless Installation Methods

The following presents the feasibility of using trenchless installation methods, advantages, disadvantages and geotechnical concerns.

It is understood that a single pipe, with an assumed diameter of about 1,600 mm and invert elevations of between about 227.5 and 227.7 m would be installed if a trenchless installation is preferred. For the purposes of assessing the feasibility of trenchless installation methods, it is assumed that a casing with a diameter of up to about 2.2 m would be used to allow for installation of the culvert pipe on line and grade.

The ground conditions along the planned casing, within the tunnel vertical limits (i.e., invert and obvert of the pipe), are likely to consist of existing embankment fill, consisting generally of sand with varying silt and gravel content to peat and/or native silty sand or silty clay. The invert of the casing pipe would be at or within the peat. Blast rock may also be present within the embankment fill. Depending on the final invert elevation of the pipe, bedrock may be encountered within the depth of the tunnel. The groundwater is anticipated to be above the invert of the pipe, based on the measured groundwater levels. Groundwater levels may fluctuate both seasonally and with precipitation events and groundwater levels could rise above those measured during this investigation.

The behaviour of the anticipated subsurface materials can be classified using Terzaghi's Tunnelman's Ground Classification system as modified by Heuer (1974). The behaviour of the sandy embankment fill is anticipated to behave as 'Running' above the groundwater table and 'Flowing' below the groundwater table. The native silty sand to sandy silt anticipated to be present within the tunnel horizon is anticipated to be 'Slow Ravelling / Cohesive Running' above the groundwater table, and 'Fast Ravelling / Flowing below' the groundwater table. In the absence of dewatering, the sandy embankment fill and the sandy silt to silty sand deposits will flow in an unsupported excavation.

Based on the existing culvert invert elevation of 227.5 to 227.7 m and an assumed casing diameter of 2.2 m, the available cover over the casing will be about 0.9 m (or less than half the tunnel diameter) at the pavement surface. It is understood that the required cover above the crown of the tunnel/bore for trenchless installation should be 2.5 to 3 tunnel/bore diameter relative to the ground surface. Lesser amounts of cover (as in this case) could jeopardize the stability of the working face (depending on the method) or lead to excessive ground movements both during and after installation.

The trenchless construction methods described in Section 7.2.1 include various advantages and disadvantages depending on soil conditions, depth of cover, vertical and horizontal alignment, length of pipe installation, cost and availability of equipment, and carry varying levels of risk of successfully completing the installation.

The sandy flowing soils below the groundwater table are a challenge for a trenchless installation along this alignment, unless a MTBM is used or the groundwater level is lowered in advance of tunnelling (which will also be required for an open-cut installation). The peat soils at the casing invert may also result in face instability, regardless of the extent of dewatering, due to their low strength.

There are also potential obstructions along the trenchless path in the form of cobbles and boulders and blast rock. The Precambrian rock surface is also indicated to be within 1 m of the invert near the outlet. The surface of Precambrian rock can vary significantly over short distances, due to its igneous origins, and there is a risk the rock surface may be higher between the boreholes.

Considering the risks at this site due to the required dewatering due to the presence of flowing soils, potential for face instability due to the peat at the invert level, the low cover (less than half the casing diameter) and the high risk of obstructions (boulders, blast rock and the bedrock surface) that would halt most trenchless methods, a trenchless installation is not recommended at this location from a foundations perspective.

#### 7.2.3 Recommended Culvert Replacement Approach

From a foundation engineering perspective, replacement of the culvert with a closed box culvert via open cut techniques, and using the phased approach described in the sections above (thus avoiding the need for temporary roadway protection to maintain traffic flow), is the recommended construction methodology for this project.

Both a steel CSP culvert or a concrete open footed culvert are also considered feasible from a foundation perspective.

While the use of some of the trenchless technologies described herein is considered feasible, they include increased risks that could result in project delays and additional cost incurrence.

## 8.0 OPEN-CUT DESIGN RECOMMENDATIONS

### 8.1 Seismic Design

#### 8.1.1 Seismic Hazard and Importance Category

The CHBDC states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) by the Geological Survey of Canada (GSC).

The current seismic hazard maps (referred to as the 5th generation seismic hazard maps) were developed by the GSC and were made available for public use in December 2015.

In accordance with Section 4.4.2 of the CHBDC, it is understood that Highway 11 at this location has been given an importance category of “Major Route”.

#### 8.1.2 Seismic Site Classification

In accordance with the CHBDC, the selection of the seismic site classification is based on the soil conditions encountered in the upper 30 m of the stratigraphy below the founding elevation. Based on the soil conditions encountered below the anticipated culvert founding elevation, the site is classified as a Seismic Site Class D in accordance with Table 4.1 of the CHBDC.

### 8.1.3 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the CHBDC and based on the location of the culvert (latitude 44.8195 N longitude -79.3238 W), the values provided in Table 8 are the reference Site Class C peak seismic hazard values based on data obtained from Earthquakes Canada (www.earthquakescanada.nrcan.gc.ca).

**Table 8 – Site Class C Spectral Values for Structure Site**

| Parameter | 2% Probability of Exceedance in 50 Years – 2,475 Year (g) |
|---|---|
| PGA | 0.065 |
| T ≤ 0.2 s | 0.112 |
| T = 0.5 s | 0.081 |
| T = 1.0 s | 0.050 |
| T = 2.0 s | 0.026 |
| T = 5.0 s | 0.007 |
| T 10.0 s | 0.003 |

The values given above are for the reference ground condition Site Class C and must be modified to the site-specific seismic site classification given in Section 8.1.2 (Site Class D) in accordance with Section 4.4.3 of the CHBDC. As indicated in Section 4.4.3.3 of the CHBDC, the value of $PGA_{ref}$ for use with Tables 4.2 to 4.9 shall be taken as 80 percent of the PGA for Site Class C where Sa(0.2)/PGA is less than 2.0. Based on this requirement, a $PGA_{ref}$ value of 0.052 was used for the 2,475-year return period. The corresponding site-specific Site Class D seismic hazard values given in Table 9 can be used for design.

**Table 9 – Site Class D Spectral Values for Structure Site**

| Parameter | 2% Probability of Exceedance in 50 Years – 2,475 year (g) |
|---|---|
| PGA | 0.084 |
| T ≤ 0.2 s | 0.139 |
| T = 0.5 s | 0.119 |
| T = 1.0 s | 0.078 |

| Parameter | 2% Probability of Exceedance in 50 Years – 2,475 year (g) |
|---|---|
| T = 2.0 s | 0.041 |
| T = 5.0 s | 0.011 |
| T 10.0 s | 0.005 |

#### 8.1.4 Liquefaction Assessment

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e., leading to potentially large surface settlements) and under undrained conditions generate excess pore pressures. The excess pore pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e., analogous to a slope failure) often referred to as "lateral spreading" or under certain conditions even catastrophic failure of the slope often referred to as "flow slides". Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

Given the relatively low Peak Ground Acceleration value at the site, the potential for seismically-induced liquefaction of the subsurface soils is considered to be marginal.

### 8.2 Foundation Options

#### 8.2.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, Highway 11 may be classified as having large traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a "typical" consequence level associated with exceeding limits states design. Given the level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of CHBDC, the level of confidence for design is considered to be a "typical degree of site and prediction model understanding" for this site. Accordingly, the appropriate corresponding ULS and SLS consequence factor, $\psi$ of [1.0], and geotechnical resistance factors from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in the following sections.

For seismic design, the consequence factor, $\Psi$, and resistance factor, $\phi_{gu}$, should be taken as unity, as per Section 4.6.3 of the CHBDC.

#### 8.2.2 Frost Protection

As per Ontario Provincial Standard Drawing (OPSD) 3090.101 (Foundation Frost Penetration Depths for Southern Ontario), the frost penetration depth at the site is 1.7 m below the existing ground surface. Footings constructed at this site (i.e., for an open footing rigid frame concrete culvert) should have a minimum embedment depth of 1.7 m below the top of streambed elevation for frost protection purposes.

## 8.3 Culvert Foundation Bearing Resistances

### 8.3.1 Box Culvert/Open Footing Culvert

For the design of shallow foundations placed on the native clay soils and/or silty sand/sandy silt, or a pad of compacted granular fill overlying these materials or sound bedrock the following factored geotechnical resistances may be used:

- Ultimate Limit States (ULS) of 250 kPa; and,
- Serviceability Limit States (SLS) of 100 kPa.

For the design of shallow foundations placed directly on sound bedrock, a factored geotechnical resistance of 1 MPa may be used (Serviceability Limit State Design does not apply to footings bearing directly on sound bedrock).

The SLS resistance values provided above are based on a maximum of 25 mm of total settlement and footings up to 2.5 m wide. For these shallow foundations, differential settlement magnitudes of less than 15 mm are expected, provided that proper subgrade preparation is carried out.

### 8.3.2 Steel CSP

Should a circular CSP be considered for the culvert replacement, it will be founded on the native clayey soils and/or native silty sandy soils, or a pad of compacted granular fill overlying these materials. It is recommended that a minimum 350 mm thick layer of OPSS.PROV 1010 Granular A be placed below the circular culverts (as discussed in further detail below) to form a bedding layer for the culvert segments and to limit the degradation of the sensitive native soil subgrade.

#### 8.3.2.1 Culvert Bedding, Backfill and Erosion Protection

For a circular replacement, the bedding levelling pad and backfill requirements should be in accordance with OPSS 421 (Construction Specification for Pipe Culvert Installation in Open Cut). The culvert should be provided with at least 350 mm of OPSS.PROV 1010 Granular A material for bedding purposes shaped to the underside of the circular culvert as per Section 7.6.5.4 of the CHBDC.

A 200 mm thickness of the bedding layer that is in direct contact with the invert should be left uncompacted to allow proper embedment of the corrugation profile. The remaining portion of the bedding should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory equipment. In addition, as per Section 7.6.4.2 of the CHBDC, reinforcement of the haunches of pipe arches should be provided as shown in Figure 7.4 of the CHBDC. The material placed in the trench reinforcement should also consist of OPSS.PROV

1010 Granular A material compacted to at least 95 percent of its standard Proctor maximum dry density.

### 8.4 Wingwalls / Retaining Walls

All footings for cantilever walls should be provided with 1.7 m of earth cover or thermal equivalent for frost protection. Although it is not necessary to found armour stone wall at depths greater than that required for frost protection and to provide sufficient embedment for stability, it must be sufficiently buried to prevent undermining by scour. All retaining wall footings should also be adequately protected against scour as noted in Section 1.9.5 of the CHBDC.

At the time of reporting, details for headwalls and retaining walls were not available. These structures, if planned, may be founded within the bedrock and/or on the compact to very loose native silty sand to sandy silt and/or the native silty clay, at or below about elevation 229 m, or on a pad of compacted suitable granular fill overlying these materials. A factored geotechnical resistance at Ultimate Limit States (ULS) of 250 kilopascals (kPa) and a geotechnical reaction of 100 kPa at Serviceability Limit States (SLS) may be used for design where the structures are founded on the noted native soils and/or pad of compacted granular fill. The SLS value assumes a settlement of 25 mm. If required, a granular levelling course approximately 75 mm in thickness can be placed on the founding strata for the armour stone wall.  Foundations placed on or within sound bedrock may be designed using factored geotechnical resistances at ULS of 500 kPa.

### 8.5 Sliding Resistance

The parameter values in Table 10 may be used to calculate the lateral resistance to sliding/shearing at the foundation-soil interface:

**Table 10 – Summary of Sliding Resistance Design Parameters**

| Structure Interaction | Effective Friction Angle (degrees) | Effective Cohesion (kPa) |
|---|---|---|
| Cast in Place Concrete – Native Silty Clay | - | 75 |
| Cast in Place Concrete – Native Glacial Till | 30 | - |

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

The above values assume that the subgrade materials are not disturbed by construction activities or groundwater inflow.

## 8.6 Lateral Earth pressures for Design

The lateral pressures acting on the retaining or culvert walls will depend on the type and method of placement of the backfill materials, on the nature of the soil behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

The following recommendations are made concerning the design of the walls in accordance with the CHBDC (version S6.1:19). It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design in accordance with CHBDC Figure 6.6.

### 8.6.1 Static Lateral Earth Pressures

If the wall support does not allow lateral yielding (such is typically the case for a rigid concrete box culvert), at-rest earth pressures should be assumed for geotechnical design. The granular fill should be placed in a zone with a width equal to at least or greater than the frost depth (i.e., 1.7 m) behind the culvert walls (Case (a) from commentary on CHBDC, Figure C6.31).

For Case (a), the restrained case, the pressures are based on the existing embankment fill materials, assuming a Select Subgrade Material (SSM) is used, and the following parameters (unfactored) may be used:

- Soil unit weight: 19 kN/m$^3$;
- Coefficient of lateral earth pressure - 'At rest' or restrained, $K_o$: 0.47.

If the wall support allows lateral yielding (unrestrained structure, such as typically the case for retaining walls), active earth pressures may be used in the geotechnical design of the structure. The granular fill should be placed in a wedged shaped zone with a width equal to at least 1.4 m at the footing level against a cut slope which begins at the footing level and extends upwards at a maximum inclination of 1 horizontal to 1 vertical (Case (b) from commentary on CHBDC Figure C6.31).

For walls backfilled using granular materials in accordance with Case (b), the following parameters (unfactored) may be assumed:

**Table 11 – Summary of Static Lateral Earth Pressure Coefficients**

| Parameter | Material | | |
|---|---|---|---|
| | Granular A | Granular B Type II | Granular B Type I |
| Unit Weight | 21.5 | 22.5 | 19.5 |
| Coefficient of Lateral Earth Pressure – Active (unrestrained) $K_a$ | 0.27 | 0.27 | 0.31 |
| Coefficient of Lateral Earth Pressure – Passive (unrestrained) $K_p$ | 3.7 | 3.7 | 3.25 |

The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as follows:

- Rotation (i.e., of wall movement to wall height) of approximately 0.002 about the base of a vertical wall;
- Horizontal translation of 0.001 times the height of the wall; or,
- A combination of both.

### 8.6.2 Seismic Lateral Earth Pressures

Seismic loading will result in increased lateral earth pressures acting on the walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.

In accordance with Sections 6.14.7.2 and C.6.14.7.2 of the CHBDC and its Commentary, for structures which do not allow lateral yielding, the horizontal seismic coefficient, $k_h$, used in the calculation of the seismic active pressure coefficient is equal to the site-adjusted Peak Ground Acceleration (PGA). For structures which allow lateral yielding, $k_h$ is taken as 0.5 times the site-adjusted PGA. For both cases the value of the vertical seismic coefficient, $k_v$ is taken as zero.

The following seismic active pressure coefficients ($K_{AE}$) may be used in the design for each of the backfill conditions. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

**Table 12 – Seismic Active Earth pressure Coefficients. $K_{AE}$**

| Parameter | Site Adjusted PGA | Material Granular A | | |
|---|---|---|---|---|
| | | Granular A | Granular B Type II | Granular B Type I |
| Non-yielding Wall | 0.084 | 0.32 | 0.32 | 0.36 |
| Yielding Wall | | 0.29 | 0.29 | 0.33 |

The above $K_{AE}$ values for yielding walls are applicable provided that the wall can move up to 250 $k_h$ (mm), where kh is the site-adjusted PGA as given in the table above. This corresponds to displacement of up to approximately 21 mm for 2,475-year design earthquakes at this site.

The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ yielding walls;}$$

$$\sigma_h(d) = K_o \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ non-yielding walls;}$$

Where: $\sigma_h(d)$ = the (static plus seismic) lateral earth pressure at depth, d, (kPa);

$K_a$ = the static active earth pressure coefficient;

$K_o$ = the static at-rest earth pressure coefficient;

$K_{AE}$ = the seismic active earth pressure coefficient;

$\gamma$ = the unit weight of the backfill soil (kN/m$^3$), as given previously;

D = the depth below the top of the wall (m); and,

H = the total height of the wall (m).

### 8.7 Embankment Design and Reinstatement

The existing embankments have slopes that are flatter than 2H:1V. Embankment reinstatement, after culvert replacement, should be carried out in accordance with OPSS.PROV 206 (Construction Specification for Grading) and should match the adjacent slope geometry. The new embankment material should consist of imported OPSS.PROV 1010 Granular B Type I or II material. Excavated granular fill may also be reused as embankment fill, provided there is no organic or cohesive material in the excavated fill, there is sufficient space to stockpile on site, and the moisture content is controlled within acceptable limits for compaction. Excavated granular fill must not be used as culvert bedding or backfill.

Granular fill should be placed and compacted in accordance with OPSS.PROV 501 (Construction Specification for Compacting). Where new embankment fill is placed against existing embankment slopes the existing earth or fill slope must be benched in accordance with OPSD 208.010 (Benching of Earth Slopes).

Provided the subgrade is prepared as outlined and embankment fill is placed as recommended herein, an embankment slope inclined at 2H:1V or flatter will remain stable.

Settlements in excess of 25 mm are not anticipated due to placement of the new embankment fill.

### 8.8 Subgrade Preparation

All embankment fill, topsoil, organics (e.g. peat) and soft or loose soils should be removed from within the zone of influence of the proposed foundations and wasted or reused as landscaping fill, as required. Subgrade preparation should be performed and monitored in accordance with OPSS.PROV 902 (Construction Specification for Excavating and Backfilling – Structures) and MTO SP 109S12, Amendment to OPSS.PROV 902, August 2018.

The native subgrade for the culvert foundation is anticipated to be silty or clayey soils, which can be easily disturbed and should be protected promptly after excavation and inspection. Excavations should be carried out using smooth bladed buckets to minimize disturbance to the soils. An Operational Constraint or a NSSP should be included in the contract in this regard, which directs the contractor to not travel on the subgrade surface with equipment.

The exposed subgrade should be provided with a minimum 300 mm of OPSS.PROV 1010 Granular A or Granular B Type II bedding (as indicated in Sections 6.1.1 and 6.1.3 above). Alternatively, a 100 mm thick concrete working slab can be placed if a box culvert is selected. After the concrete for the working slab has set, the culvert could then be constructed directly on the working slab without the need for a granular bedding material.

A sample NSSP for subgrade protection has been provided in Appendix F.

## 9.0 CORROSION AND CEMENT TYPE

One soil sample was submitted to Paracel Laboratories Ltd for chemical analysis related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The test results are provided in Appendix D.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The sulphate results in

Table 7 of this report, were compared with Table 3 of Canadian Standards Association Standards A23.1-14 (CSA A23.1) and generally indicate a low degree of sulphate attack potential on concrete structures at this site. Accordingly, GU cement could be specified for concrete in below grade applications.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub- surface environment. Generally, the test results provided in Table 7 indicate a moderate to high potential for corrosion of exposed ferrous metal at the site which should be considered in the design.

## 10.0 CONSTRUCTION CONSIDERATIONS

### 10.1 Open Cut Excavations

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities.

Excavations to depths of up to about 5 m below the existing Highway 11 grade are anticipated for the installation of a box or CSP culvert. The excavations will be through embankment fill consisting generally of very loose to dense sand with varying amount of gravel and silt as well as cobbles and boulders including blast rock, peat, native silty clay, and native sandy soil with varying silt content.

According to the OHSA, the fill material and the native sandy soils, when above the groundwater level, can be classified as Type 3 and, accordingly, during excavation within these materials, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical, or flatter, extending upwards from the bottom of the excavation. The native silty clay can be classified as a Type 2 soil and therefore allowance should be made when excavating within the silty clay for excavation side slopes of 1 horizontal to 1 vertical, or flatter, beginning from a point vertically extending upwards to 1.2 metres or less from the bottom of the excavation (provided the bottom of the excavation remains within this material or bedrock). In the instances where excavations through the fill and sandy soils will extend below the groundwater level and within the peat, these soils are classified as Type 4 soils and therefore allowance should be made for excavation side slopes within the zone below the water table of 3 horizontal to 1 vertical, or flatter, extending upwards from the bottom of the excavation.

The groundwater level in the area of the culvert is expected to reflect the creek water level.

### 10.2 Temporary Protection Systems

If the required safe side slopes for the open cut excavations cannot be accommodated, then temporary roadway protection (i.e., excavation shoring) will be required to facilitate excavation to the foundation level for the replacement of the culvert, removal of the existing culvert or for the entry and exist pits.

The design of the shoring will be entirely the responsibility of the contractor. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems), and the lateral movement should meet Performance Level 2. Traffic loading should be included as a surcharge. Traffic loading does not account for construction equipment loadings which may be higher; the contractor's shoring designer should confirm those load requirements.

Increased difficulty with the installation of temporary protection systems should be anticipated due to the presence of boulders within the embankment fill materials and the relatively shallow depth to the bedrock surface over a portion of the alignment. For preliminary assessment purposes, the use of sheet piles is not considered feasible. One option is to use H-piles and timber lagging with the H-piles installed in pre-drilled holes into the bedrock, until the depth of overburden will allow the use of sheetpiles. Recommended wording for an NSSP alerting the Contractor to this condition and the requirement to use appropriate equipment and installation techniques is provided in Appendix F.

For design of the temporary protection systems, the estimated soil parameters given below are considered applicable. Any internal bracing or raker supports must be designed to accommodate the loads applied from earth pressures, water pressures and surcharge pressures from area, line or point loads as well as the effects of sloping ground behind the system. Passive toe restraint to any soldier piles may be determined using conventional passive earth pressure distribution acting over an equivalent width equal to three times the soldier pile socket diameter provided that the soldier piles are separated by more than three times the socket diameter.

**Table 13 – Shoring Design Parameters**

| Soil Type | Coefficient of Lateral Pressure | | | Internal Angle of Friction (degrees) | Unit Weight (kN/m$^3$) |
|---|---|---|---|---|---|
| | Active, $K_a$ | At-rest, $K_o$ | Passive, $K_p$ | | |
| Existing Embankment Fill | 0.31 | 0.47 | 3.25 | 32 | 19.5 |
| Peat | 1.0 | 1.0 | N/A | 5 | 15.0 |
| Native Sandy Soils | 0.27 | 0.43 | 3.69 | 35 | 20.0 |

The lateral earth pressure coefficients presented above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are expected, the coefficients shown need to be corrected accordingly.

Consideration could be given to either partial or full removal of the protection system upon completion of construction or each stage of construction (as required). Where possible, full removal of the protection system should be considered to mitigate potential impediments to future rehabilitation/reconstruction work in the area.

### 10.3 Groundwater and Surface Water Control

The Contractor must be prepared to control the groundwater and surface water flow at the site to permit the proposed culvert replacement to be constructed in a dry and stable excavation. The groundwater level for the site at the time of the proposed replacement should be taken as the water level in the creek or the levels shown on the borehole records, whichever are higher. It is recommended that the replacement be conducted during a drier season such as after the spring freshet or prior to the fall season.

A temporary flow passage system will be required to replace the culvert in the dry. It is anticipated that a dam and pump or dam and divert system will be used during construction of the new culvert. The existing culvert may either be removed as part of the construction program or may be abandoned in place by filling with concrete or grout.

Excavations below the groundwater level are anticipated for preparing the subgrade, installing the new culvert and the removal of the existing culvert (if required). All dewatering measures, including creek and surface water diversion, must always remain operational and effective during the construction period.

However, the selection and design of temporary unwatering/dewatering system is the responsibility of the Contractor. The Contract Documents must alert the Contractor to this responsibility and to design the system in accordance with MTO SP FOUN0003 (Dewatering Structure Excavations), dated January 2020, which amends OPSS 902. A copy the SP FOUN0003 (SP3) is provided in Appendix F along with the appropriate Designer Fill-ins.

### 10.4 Erosion and Scour Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site as per OPSS.PROV 805 (Temporary Erosion and Sediment Control Measures).

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS.PROV 804 (Seed and Cover).

Provision should be made for scour and erosion protection at the culvert inlet and outlet. To prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles) and through the bedding material, a concrete cut-off wall or collar should be provided at the inlet and outlet end of the culvert.

The detailed requirements for and design of erosion protection measures for the inlet and outlet of this culvert shall be assessed by the hydraulic design engineer.

## 11.0 CONSTRUCTION CONCERNS

The likely construction methodology includes open cut excavations for the installation of foundation elements of the new culvert. Potential construction concerns may include, but are not necessarily limited to:

- Construction will extend below the water level in the watercourse. An adequate and effective surface water management and dewatering plan must be implemented to construct the culvert and wingwall/retaining wall foundations in the dry.
- The native soil which will be exposed beneath culvert bedding layers or wing wall/retaining wall spread footings is readily disturbed.
- The Contractor's selection of construction equipment and methodology must include assessment of the capability of the existing soils to support the proposed construction equipment and supplies.
- Obstructions could be encountered in the existing embankment fill and may limit choice of equipment and methods.

The successful performance of the structure installation will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations will be required as per OPSS 902 during construction to confirm that the foundation recommendations are correctly implemented, and material specifications are met.

## 12.0 CLOSURE

This report was prepared by Matthew Rainville, C.E.T., and Serge Bourque, M.Sc.E., P.Eng., and reviewed by Mr. William Cavers, P.Eng., a Senior Geotechnical Engineer with GEMTEC and the Key MTO Foundations Personnel for this project.

**GEMTEC Consulting Engineers and Scientists Limited**

Matthew Rainville, C.E.T.
Senior Technologist

Serge Bourque, M.Sc.E., P.Eng.
Principal Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
S. BOURQUE
100194197
Jan 18, 2024
PROVINCE OF ONTARIO

William Cavers, P.Eng.
Principal Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
W. CAVERS
100077934
Jan18/24
PROVINCE OF ONTARIO

MR/SB/WC

**Table A – Comparison of Foundation Alternatives**

| Culvert Alternative | Feasibility | Advantages | Disadvantages | Relative Costs | Constructability/Risks |
|---|---|---|---|---|---|
| Closed Concrete Box Culvert | • Preferred option from a foundation perspective | • Wide base reduces bearing pressures<br>• Base of the closed box does not need to be founded below frost depth reducing excavation depths and dewatering requirements due to the shallower excavation<br>• Less prone to effects of scour and erosion | • Requires temporary roadway protection and cofferdam systems to be installed prior to carrying out excavation and to facilitate construction of the culvert. | • Low to moderate | • Potential for base disturbance if groundwater not controlled, leading to added cost and schedule delays |
| Corrugated Steel Pipe Culvert | • Also a preferred option from a foundation perspective | • Wide base reduces bearing pressures<br>• Base of the steel pipe culvert does not need to be founded below frost depth reducing excavation depths and dewatering requirements due to the shallower excavation | • Generally lower durability compared to a concrete option and therefore potentially has a shorter service life<br>• Requires temporary roadway protection and cofferdam systems to be installed prior to carrying out excavation and to facilitate construction of the culvert. | • Low to moderate | • Potential for base disturbance if groundwater not controlled leading to added cost and schedule delays |
| Open Footing Concrete Culvert | • Feasible | • More flexibility for installation of temporary flow passage system | • Founding elevation is deeper than with closed box culvert, requiring deeper excavations and increased dewatering requirements.<br>• Greater probability of encountering bedrock during footing excavation.<br>• More susceptible to effects of scour and erosion<br>• Requires deeper and more costly temporary roadway protection and cofferdam systems with minimal embedment above bedrock. | • Moderate | • Deeper excavation increases excavation volume and dewatering requirements leading to added cost and schedule delays<br>• Potential for base disturbance if groundwater not controlled leading to added cost and schedule delays |

# APPENDIX A

Record of Borehole Sheets
List of Abbreviations and Terminology

Ministry of Transportation
Ontario

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

# RECORD OF BOREHOLE No 23-01



**METRIC**

| | | | |
|---|---|---|---|
| G.W.P. 5461-09-00 | LOCATION N 4964380.1; E 318724.0 NAD 83 MTM ZONE 10 | ORIGINATED BY AN |
| DIST Eastern HWY 11 | BOREHOLE TYPE Power Auger, 200 mm Diameter (Hollow Stem)/Rotary Drill, HQ Core | COMPILED BY MR |
| DATUM CGVD28 | DATE 2023.10.03 - 2023.10.03 | CHECKED BY BC |

## SOIL PROFILE

| ELEV DEPTH | DESCRIPTION |
|---|---|
| 231.4 | |
| 0.0 / 0.1 | ASPHALTIC CONCRETE |
| 0.2 | Gravelly sand, trace silt (BASE)<br>Brown<br>Moist |
| | Sand, trace gravel, trace silt (FILL)<br>Compact<br>Light brown<br>Moist |
| 229.7<br>1.7 | Blast rock, some sand and gravel (FILL)<br>Moist |
| 228.6<br>2.7 | Gravelly sand, some silt, trace clay, to sand, some silt, contains organics, possible cobbles/boulders (FILL)<br>Loose<br>Reddish brown to dark brown<br>Wet |
| 227.4<br>4.0 | PEAT, some silt and clay, trace to some sand, contains wood pieces/roots<br>Amorphous<br>Dark brown<br>Wet |
| 226.6<br>4.8 | GNEISS BEDROCK<br>Fresh<br>Very strong<br>Black, grey, pink |
| 221.6<br>9.8 | End of Borehole |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | WATER CONTENT (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|
| 1 | SS | 24 | ~5 | 28 67 (5) |
| 2 | SS | 16 | | |
| 3 | SS | >100 for 250 mm | | |
| 4 | SS | 14 | | |
| 5 | SS | 9 | ~21 | 29 57 12 2 |
| 6 | SS | 3 | | |
| 7 | SS | >50 for 50 mm | | |
| 8 | RC | | | RC #8<br>TCR=100%<br>SCR=68%<br>RQD=100%<br>UCS=98MPa |
| 9 | RC | | | RC #9<br>TCR=100%<br>SCR=91%<br>RQD=94%<br>UCS=85MPa |
| 10 | RC | | | RC #10<br>TCR=100%<br>SCR=86%<br>RQD=99%<br>UCS=52MPa |
| 11 | RC | | | RC #11<br>TCR=100%<br>SCR=96%<br>RQD=100% |

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (231 to 222); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT $W_P$, NATURAL MOISTURE CONTENT w, LIQUID LIMIT $W_L$; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

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

MTO BOREHOLE LOG 102944.001_GINT_MTO_GEOTECH.GPJ GEMTEC MTO.GDT 1/10/24

Ministry of Transportation Ontario

GEMTEC Consulting Engineers and Scientists

# RECORD OF BOREHOLE No 23-02



**METRIC**

| | | | | | |
|---|---|---|---|---|---|
| G.W.P. | 5461-09-00 | LOCATION | N 4964387.6; E 318733.1 NAD 83 MTM ZONE 10 | ORIGINATED BY | AN |
| DIST | Eastern HWY 11 | BOREHOLE TYPE | Power Auger, 200 mm Diameter (Hollow Stem)/Rotary Drill, HQ Core | COMPILED BY | MR |
| DATUM | CGVD28 | DATE | 2023.10.04 - 2023.10.04 | CHECKED BY | BC |

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | W (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 231.2 | | | | | | |
| 0.0 / 0.1 | ASPHALTIC CONCRETE | | | | | |
| 0.2 | Sand and gravel, trace silt (BASE) Brown Moist | 1 | GS | | ~8 | 2 89 (9) |
| | Sand, trace to some gravel, trace silt (FILL) Dense Brown Moist | 2 | SS | 47 | | |
| 229.7 / 1.5 | Silty sand, trace gravel, to sand, trace gravel, trace silt, contains organics/roots (FILL) Very loose Brown to grey brown Moist to wet | 3 | SS | 1 | | |
| | | 4 | SS | 1 | ~27 | 1 91 6 2 |
| | | 5 | SS | 4 | | |
| 227.1 / 4.1 | PEAT, some silt and clay, some sand, contains wood pieces/roots Amorphous Dark brown to dark grey Wet | 6 | SS | 3 | | |
| 226.7 / 4.6 | Silty SAND/sandy SILT, some clay, to SAND, trace silt, trace clay, contains organics. Compact to loose Grey brown to grey Moist to wet | 7 | SS | 17 | ~20 | 0 45 43 12 |
| | | 8 | SS | 9 | | |
| | | 9 | SS | 10 | ~22 | 0 85 9 6 |
| 224.4 / 6.9 | GRAVELLY SAND, trace to some silt Compact Grey Wet | 10 | SS | 27 | | |
| 223.5 / 7.7 | GNEISS BEDROCK Fresh Very strong Black, grey, pink | 11 | SS | >50 | | |
| | | 12 | RC | | | RC #12 TCR=100% SCR=71% RQD=100% |
| | | 13 | RC | | | RC #13 TCR=100% SCR=57% RQD=82% UCS=62MPa |
| | | 14 | RC | | | RC #14 TCR=100% SCR=55% RQD=95% |
| 220.3 / 10.9 | End of Borehole | | | | | |

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (231 to 221); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT $W_P$, NATURAL MOISTURE CONTENT W, LIQUID LIMIT $W_L$; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

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

Ministry of Transportation
Ontario

GEMTEC
Consulting Engineers and Scientists

# RECORD OF BOREHOLE No 23-03



**METRIC**

G.W.P. 5461-09-00 | LOCATION N 4964389.1; E 318744.7 NAD 83 MTM ZONE 10 | ORIGINATED BY AN

DIST Eastern HWY 11 | BOREHOLE TYPE Power Auger, 200 mm Diameter (Hollow Stem) | COMPILED BY MR

DATUM CGVD28 | DATE 2023.10.02 - 2023.10.02 | CHECKED BY BC

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | WATER CONTENT (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 230.8 | | | | | | | |
| 230.6 / 0.2 | Sand and gravel, trace silt (BASE) Grey Moist | 1 | SS | 27 | | | |
| | Sand, trace to some gravel, trace silt, with cobbles/boulder (FILL) Compact Brown Moist | 2 | SS | 29 | 230 | ~12 | 18 72 (10) |
| 229.3 / 1.5 | Silty sand, trace to some gravel (FILL) Loose to very loose Brown Moist to wet | 3 | SS | 10 | 229 | | |
| | | 4 | SS | 1 | | | |
| 228.1 / 2.8 | PEAT to PEAT, some silt and clay, trace to some sand, contains organics Fibrous to amorphous Dark brown/black to grey brown/dark brown Wet | | | | 228 | | |
| 227.5 / 3.3 | SILTY CLAY, with silty sand seams Stiff to very stiff Grey Wet | 5 | SS | 10 | | | |
| | | 6 | SS | 13 | 227 | ~23 | 0 28 50 23 |
| 226.2 / 4.6 | Layered Silty SAND/Sandy SILT, trace to some clay Very loose to compact Grey Wet | 7 | SS | 8 | 226 | | |
| | | 8 | SS | 3 | 225 | | |
| | | 9 | SS | 8 | | ~36 | 0 21 77 2 |
| | | 10 | SS | 13 | 224 | | |
| | | 11 | SS | 5 | 223 | ~25 | 0 64 34 2 |
| | | 12 | SS | 5 | 222 | | |
| | | 13 | SS | 7 | 221 | | |
| | | 14 | SS | 18 | | | |
| 220.3 / 10.5 | End of Borehole | | | | | | |

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT $W_P$, NATURAL MOISTURE CONTENT W, LIQUID LIMIT $W_L$; WATER CONTENT (%) 10 20 30; UNIT WEIGHT $\gamma$ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

# ABBREVIATIONS AND TERMINOLOGY USED ON RECORDS OF BOREHOLES AND TEST PITS

| SAMPLE TYPES | |
|---|---|
| AS | Auger sample |
| CA | Casing sample |
| CS | Chunk sample |
| BS | Borros piston sample |
| GS | Grab sample |
| MS | Manual sample |
| RC | Rock core |
| SS | Split spoon sampler |
| ST | Slotted tube |
| TO | Thin-walled open shelby tube |
| TP | Thin-walled piston shelby tube |
| WS | Wash sample |

| SOIL TESTS | |
|---|---|
| w | Water content |
| PL, $w_p$ | Plastic limit |
| LL, $w_L$ | Liquid limit |
| C | Consolidation (oedometer) test |
| $D_R$ | Relative density |
| DS | Direct shear test |
| $G_S$ | Specific gravity |
| M | Sieve analysis for particle size |
| MH | Combined sieve and hydrometer (H) analysis |
| MPC | Modified Proctor compaction test |
| SPC | Standard Proctor compaction test |
| OC | Organic content test |
| UC | Unconfined compression test |
| γ | Unit weight |

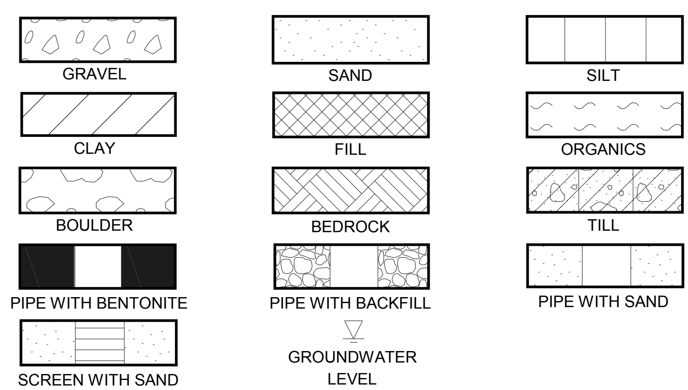
## PENETRATION RESISTANCE

**Standard Penetration Resistance, N**
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 millimetres (30 in.) required to drive a 50 mm split spoon sampler for a distance of 300 mm (12 in.). For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

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

| | |
|---|---|
| WH | Sampler advanced by static weight of hammer and drill rods |
| WR | Sampler advanced by static weight of drill rods |
| PH | Sampler advanced by hydraulic pressure from drill rig |
| PM | Sampler advanced by manual pressure |

| COHESIONLESS SOIL Compactness | | COHESIVE SOIL Consistency | |
|---|---|---|---|
| **SPT N-Values** | **Description** | **Cu, kPa** | **Description** |
| 0-4 | Very Loose | 0-12 | Very Soft |
| 4-10 | Loose | 12-25 | Soft |
| 10-30 | Compact | 25-50 | Firm |
| 30-50 | Dense | 50-100 | Stiff |
| >50 | Very Dense | 100-200 | Very Stiff |
| | | >200 | Hard |




## GRAIN SIZE

Scale: 0.01 | 0.1 | 1.0 | 10 | 100 | 1000mm

| SILT CLAY | SAND | | | GRAVEL | COBBLE | BOULDER |
|---|---|---|---|---|---|---|
| | Fine | Medium | Coarse | | | |
| < 0.08 | 0.08 – 0.4 | 0.4 – 2 | 2 – 5 | 5 – 80 | 80 – 200 | > 200 |

## DESCRIPTIVE TERMINOLOGY

(Based on the CANFEM 4th Edition)

| 0 – 10 | 10 – 20 | 20 – 35 | 35 – |
|---|---|---|---|
| TRACE | SOME | ADJECTIVE | noun > 35% and main fraction |
| trace clay, etc | some gravel, etc. | silty, etc. | sand and gravel, etc. |

# LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

| WEATHERING STATE | |
|---|---|
| Fresh | No visible sign of rock material weathering |
| Faintly weathered | Weathering limited to the surface of major discontinuities |
| Slightly weathered | Penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material |
| Moderately weathered | Weathering extends throughout the rock mass but the rock material is not friable |
| Completely weathered | Rock is wholly decomposed and in a friable condition but the rock and structure are preserved |

| CORE CONDITION |
|---|
| **Total Core Recovery (TCR)**<br>The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run |
| **Solid Core Recovery (SCR)**<br>The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run. |
| **Rock Quality Designation (RQD)**<br>The percentage of solid drill core, greater than 100 mm length, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completed broken core to 100% for core in solid segments. |

| BEDDING THICKNESS | |
|---|---|
| **Description** | **Thickness** |
| Thinly laminated | < 6 mm |
| Laminated | 6 - 20 mm |
| Very thinly bedded | 20 - 60 mm |
| Thinly bedded | 60 - 200 mm |
| Medium bedded | 200 - 600 mm |
| Thickly bedded | 600 - 2000 mm |
| Very thickly bedded | 2000 - 6000 mm |

| DISCONTINUITY SPACING | |
|---|---|
| **Description** | **Spacing** |
| Very close | 20 - 60 mm |
| Close | 60 - 200 mm |
| Moderate | 200 - 600 mm |
| Wide | 600 -2000 mm |
| Very wide | 2000 - 6000 mm |

| ROCK QUALITY | |
|---|---|
| **RQD** | **Overall Quality** |
| 0 - 25 | Very poor |
| 25 - 50 | Poor |
| 50 - 75 | Fair |
| 75 - 90 | Good |
| 90 - 100 | Excellent |

| ROCK COMPRESSIVE STRENGTH | |
|---|---|
| **Comp. Strength, MPa** | **Description** |
| 1 - 5 | Very weak |
| 5 - 25 | Weak |
| 25 - 50 | Moderate |
| 50 - 100 | Strong |
| 100 - 250 | Very strong |

# APPENDIX B

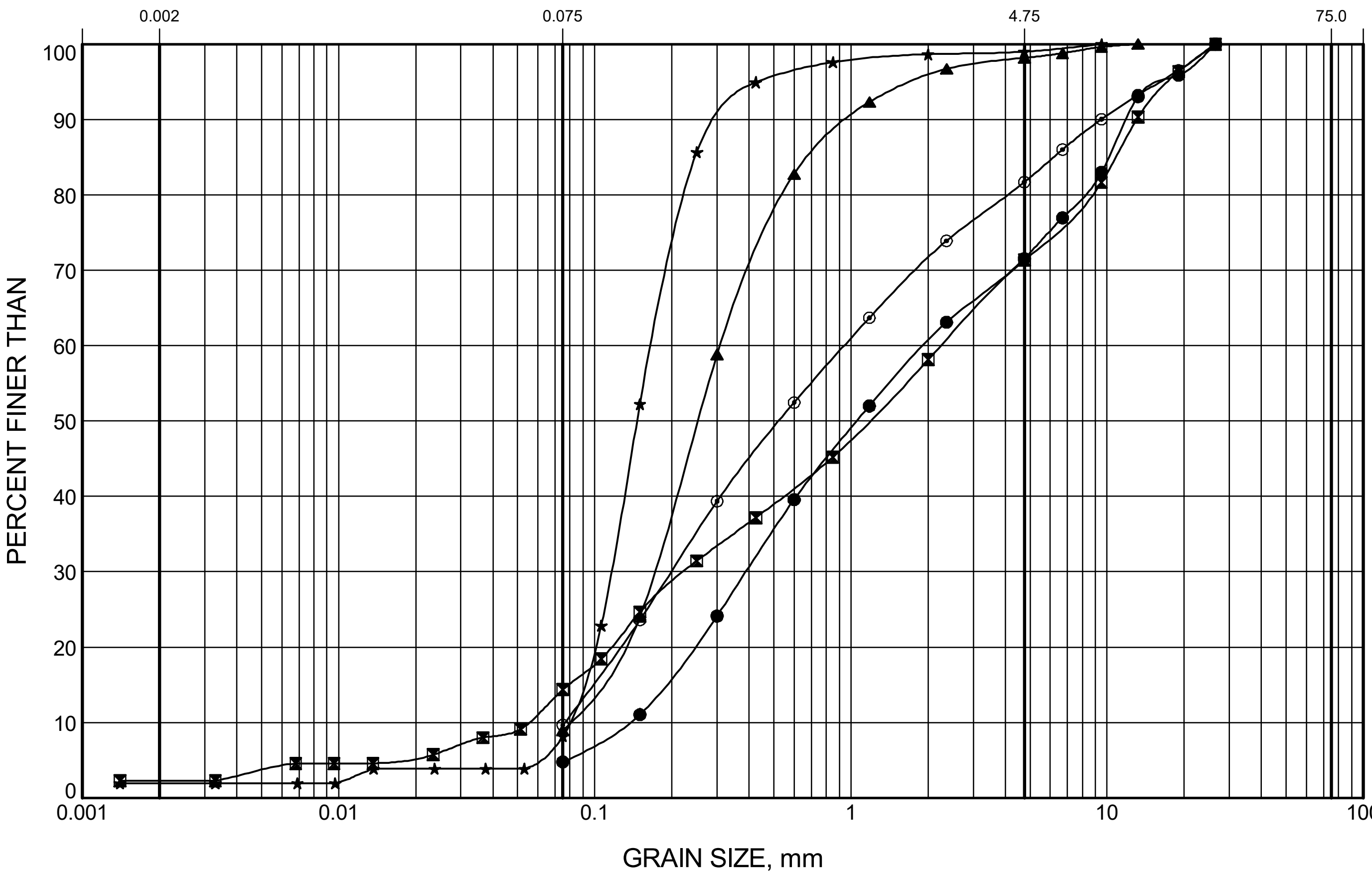
Laboratory Test Results

# GRAIN SIZE DISTRIBUTION

**Hwy 11, Replacement of Culvert #8**

**FIGURE B1**

## Embankment Fill




| CLAY | SILT | SAND | | | GRAVEL | | COBBLES |
|---|---|---|---|---|---|---|---|
| | | FINE | MEDIUM | COARSE | FINE | COARSE | |

| Legend | Borehole | Sample | Depth (m) | % Gravel | % Sand | % Silt | % Clay |
|---|---|---|---|---|---|---|---|
| ● | 23-01 | A | 0.06 - 0.2 | 28 | 67 | 5 | |
| ⧗ | 23-01 | 5 | 3.1 - 3.7 | 29 | 57 | 12 | 2 |
| ▲ | 23-02 | 1 | 0.3 - 0.5 | 2 | 89 | 9 | |
| ★ | 23-02 | 4 | 2.4 - 2.7 | 1 | 91 | 6 | 2 |
| ⊙ | 23-03 | 2 | 0.8 - 1.4 | 18 | 72 | 10 | |

SOILS GRAIN SIZE GRAPH (COMBINED) 102944.001_GINT_MTO_LABORATORY.GPJ

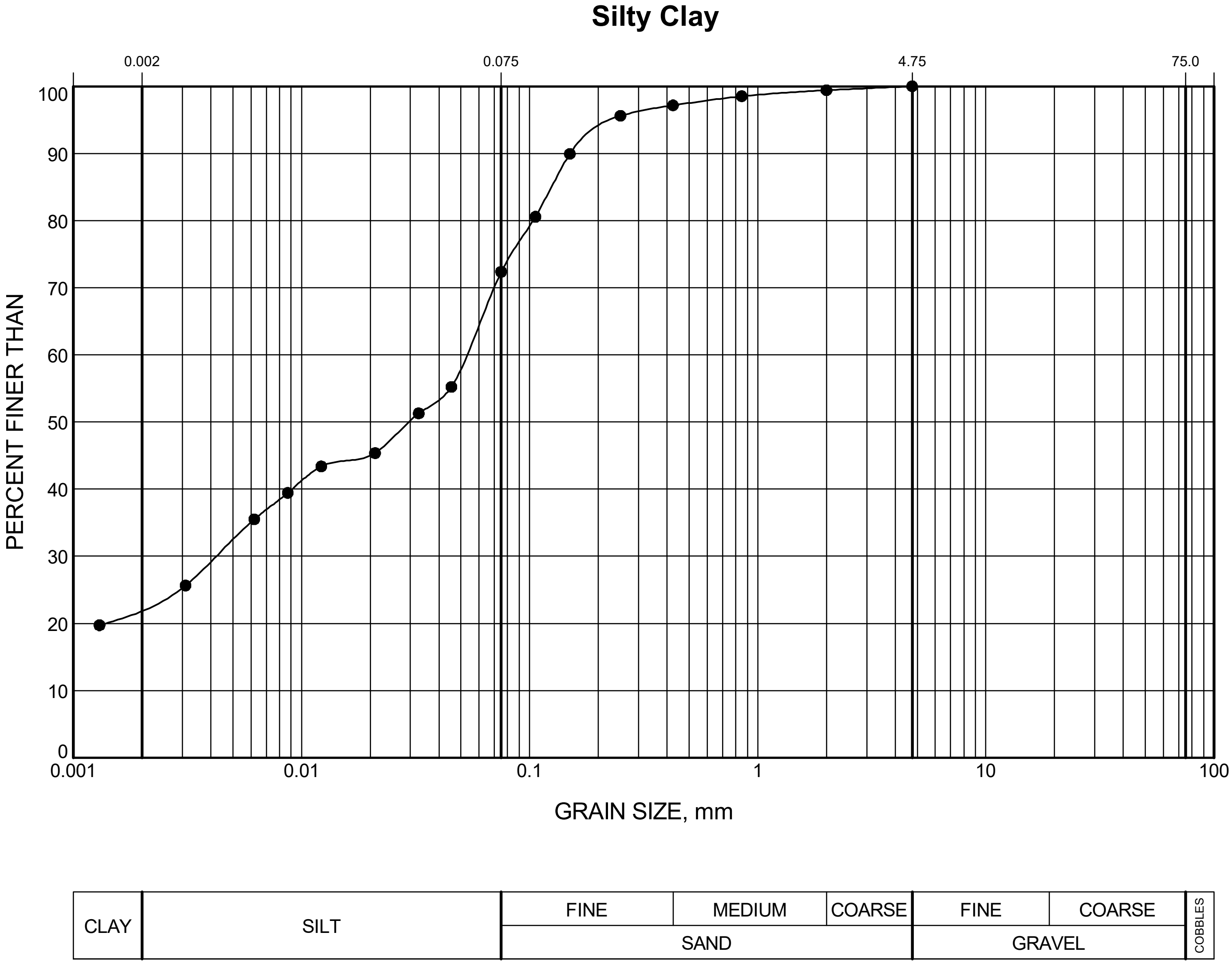
GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

Date: October 2023 Prepared by: KS
G.W.P.: 5461-09-00 Checked by: BC

# GRAIN SIZE DISTRIBUTION

**Hwy 11, Replacement of Culvert #8**

**FIGURE B2**




| Legend | Borehole | Sample | Depth (m) | % Gravel | % Sand | % Silt | % Clay |
|---|---|---|---|---|---|---|---|
| ● | 23-03 | 6 | 3.8 - 4.4 | 0 | 28 | 50 | 23 |

SOILS GRAIN SIZE GRAPH (COMBINED) 102944.001_GINT_MTO_LABORATORY.GPJ

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

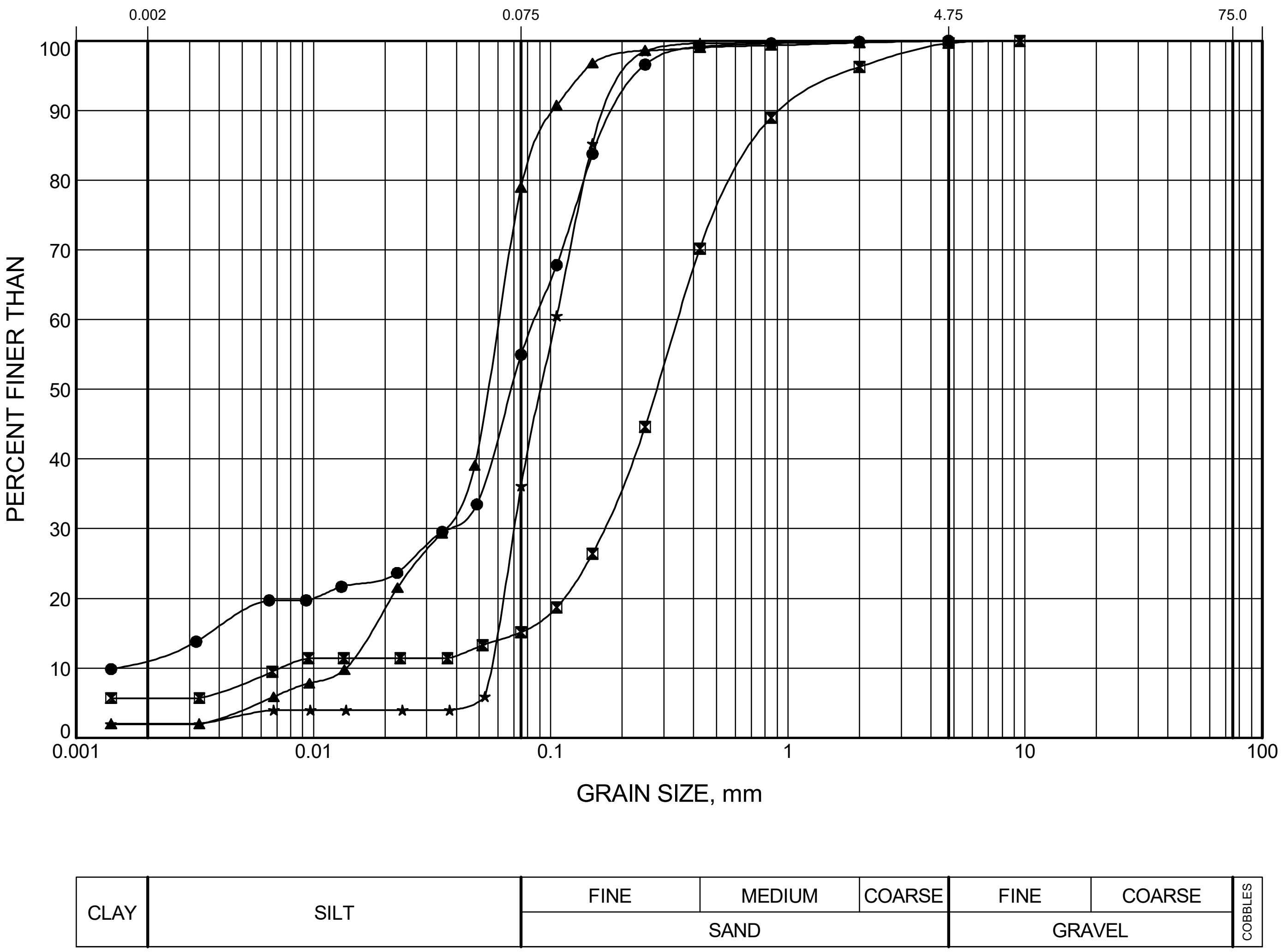
| | | | |
|---|---|---|---|
| Date: | October 2023 | Prepared by: | KS |
| G.W.P.: | 5461-09-00 | Checked by: | BC |

# GRAIN SIZE DISTRIBUTION

**Hwy 11, Replacement of Culvert #8**

**FIGURE B3**

## Sand, Silty Sand, Sandy Silt




| Legend | Borehole | Sample | Depth (m) | % Gravel | % Sand | % Silt | % Clay |
|---|---|---|---|---|---|---|---|
| ● | 23-02 | 7 | 4.6 - 5.2 | 0 | 45 | 43 | 12 |
| ⊠ | 23-02 | 9 | 6.1 - 6.7 | 0 | 85 | 9 | 6 |
| ▲ | 23-03 | 9 | 6.1 - 6.7 | 0 | 21 | 77 | 2 |
| ★ | 23-03 | 11 | 7.6 - 8.2 | 0 | 64 | 34 | 2 |

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

| | | | |
|---|---|---|---|
| Date: | October 2023 | Prepared by: | KS |
| G.W.P.: | 5461-09-00 | Checked by: | BC |

| GEMTEC Consulting Engineers and Scientists | Client: McIntosh Perry | FIGURE B4 |
|---|---|---|
| | Project: GWP 5461-09-00 - Hwy 11 - Kahshe - Culvert Investigation | Rock Core Compressive Strength |
| | Project #: 102944001 | |

| Date/Time Sampled: 23/10/17 12:21:00 PM | Date/Time Tested: 23/10/17 12:22:05 PM |
|---|---|

| BH | Sample No | Depth | Description | Diameter, mm | Area, mm² | Length After Capping, mm | L/D | Load, kN | Comp. Str., MPa |
|---|---|---|---|---|---|---|---|---|---|
| 23-01 | 08 | 5.02-5.18 | Bedrock | 63.0 | 3122 | 125 | 1.98 | 305.090 | 97.7 |
| 23-01 | 09 | 7.01-7.11 | Bedrock | 63.6 | 3174 | 126 | 1.98 | 271.150 | 85.4 |
| 23-01 | 10 | 8.36-8.68 | Bedrock | 63.2 | 3140 | 125 | 1.98 | 163.380 | 52.0 |
| 23-02 | 13 | 8.53-8.96 | Bedrock | 63.3 | 3143 | 125 | 1.98 | 195.620 | 62.2 |

# APPENDIX C

Photographs of Bedrock Core Samples

**BOREHOLE: 23-01**
**BORING DATE: OCTOBER 3, 2023**
**DEPTH: 4.78 to 9.80 mbgs**



GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS
32 Steacie Drive, Ottawa, ON K2K 2A9
T: (613) 836-1422 | www.gemtec.ca | ottawa@gemtec.ca


| Project | | |
|---|---|---|
| FOUNDATION INVESTIGATION AND DESIGN REPORT<br>REPLACEMENT OF CULVERT NO. 8<br>STATION 14+900, HIGHWAY 11<br>DISTRICT OF MUSKOKA, ONTARIO | **FIGURE C1**<br>File No.<br>102944.001 | ROCKCORE PHOTOGRAPH<br>BOREHOLE 23-01 |

**BOREHOLE: 23-02**
**BORING DATE: OCTOBER 4, 2023**
**DEPTH: 7.72 to 10.90 mbgs**


GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS
32 Steacie Drive, Ottawa, ON K2K 2A9
T: (613) 836-1422 | www.gemtec.ca | ottawa@gemtec.ca

Project
FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF CULVERT NO. 8
STATION 14+900, HIGHWAY 11
DISTRICT OF MUSKOKA, ONTARIO

**FIGURE C2**

File No.
102944.001

ROCKCORE PHOTOGRAPH
BOREHOLE 23-02

# APPENDIX D

Chemical Analysis of Soil Sample Relating to Corrosion

300 - 2319 St. Laurent Blvd
Ottawa, ON, K1G 4J8
1-800-749-1947
www.paracellabs.com

# Certificate of Analysis

**GEMTEC Consulting Engineers and Scientists Limited**
32 Steacie Drive
Kanata, ON K2K 2A9
Attn: Matt Rainville

Client PO:
Project: 102944.001
Custody:

Report Date: 19-Oct-2023
Order Date: 12-Oct-2023

**Order #: 2341205**

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

| Paracel ID | Client ID |
|---|---|
| 2341205-01 | BH23-02 SA5 10'-12' |

Approved By:

Dale Robertson, BSc
Laboratory Director

Certificate of Analysis

Report Date: 19-Oct-2023

**Client:** **GEMTEC Consulting Engineers and Scientists Limited**

Order Date: 12-Oct-2023

**Client PO:**

**Project Description: 102944.001**

## Analysis Summary Table

| Analysis | Method Reference/Description | Extraction Date | Analysis Date |
|---|---|---|---|
| Anions | EPA 300.1 - IC, water extraction | 13-Oct-23 | 13-Oct-23 |
| Conductivity | MOE E3138 - probe @25 °C, water ext | 13-Oct-23 | 13-Oct-23 |
| pH, soil | EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext. | 12-Oct-23 | 12-Oct-23 |
| Resistivity | EPA 120.1 - probe, water extraction | 13-Oct-23 | 19-Oct-23 |
| Solids, % | CWS Tier 1 - Gravimetric | 13-Oct-23 | 13-Oct-23 |

PARACEL
LABORATORIES LTD.

**Order #: 2341205**

Certificate of Analysis

Report Date: 19-Oct-2023

**Client: GEMTEC Consulting Engineers and Scientists Limited**

Order Date: 12-Oct-2023

**Client PO:**

**Project Description: 102944.001**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| | **Client ID:** | BH23-02 SA5 10'-12' | - | - | - | - | - |
| | **Sample Date:** | 04-Oct-23 10:30 | - | - | - | | |
| | **Sample ID:** | 2341205-01 | - | - | - | | |
| | **Matrix:** | Soil | - | - | - | | |
| | **MDL/Units** | | | | | | |
| **Physical Characteristics** | | | | | | | |
| % Solids | 0.1 % by Wt. | 82.8 | - | - | - | - | - |
| **General Inorganics** | | | | | | | |
| Conductivity | 5 uS/cm | 995 | - | - | - | - | - |
| pH | 0.05 pH Units | 6.72 | - | - | - | - | - |
| Resistivity | 0.1 Ohm.m | 10.0 | - | - | - | - | - |
| **Anions** | | | | | | | |
| Chloride | 10 ug/g | 448 | - | - | - | - | - |
| Sulphate | 10 ug/g | 64 | - | - | - | - | - |

## Method Quality Control: Blank

| Analyte | Result | Reporting Limit | Units | %REC | %REC Limit | RPD | RPD Limit | Notes |
|---|---|---|---|---|---|---|---|---|
| **Anions** | | | | | | | | |
| Chloride | ND | 10 | ug/g | | | | | |
| Sulphate | ND | 10 | ug/g | | | | | |
| **General Inorganics** | | | | | | | | |
| Conductivity | ND | 5 | uS/cm | | | | | |
| Resistivity | ND | 0.1 | Ohm.m | | | | | |

## Method Quality Control: Duplicate

| Analyte | Result | Reporting Limit | Units | Source Result | %REC | %REC Limit | RPD | RPD Limit | Notes |
|---|---|---|---|---|---|---|---|---|---|
| **Anions** | | | | | | | | | |
| Chloride | 208 | 10 | ug/g | 192 | | | 8.0 | 35 | |
| Sulphate | 122 | 10 | ug/g | 122 | | | 0.1 | 35 | |
| **General Inorganics** | | | | | | | | | |
| Conductivity | 313 | 5 | uS/cm | 278 | | | 11.9 | 5 | QR-04 |
| pH | 7.94 | 0.05 | pH Units | 7.97 | | | 0.4 | 2.3 | |
| Resistivity | 31.9 | 0.1 | Ohm.m | 36.0 | | | 11.9 | 20 | |
| **Physical Characteristics** | | | | | | | | | |
| % Solids | 92.2 | 0.1 | % by Wt. | 92.1 | | | 0.2 | 25 | |

## Method Quality Control: Spike

| Analyte | Result | Reporting Limit | Units | Source Result | %REC | %REC Limit | RPD | RPD Limit | Notes |
|---|---|---|---|---|---|---|---|---|---|
| **Anions** | | | | | | | | | |
| Chloride | 284 | 10 | ug/g | 192 | 92.1 | 82-118 | | | |
| Sulphate | 219 | 10 | ug/g | 122 | 96.4 | 80-120 | | | |

**Qualifier Notes:**

## QC Qualifiers:

QR-04 Duplicate results exceeds RPD limits due to non-homogeneous matrix.

**Sample Data Revisions:**

None

**Work Order Revisions / Comments:**

None

**Other Report Notes:**

n/a: not applicable

ND: Not Detected

MDL: Method Detection Limit

Source Result: Data used as source for matrix and duplicate samples

%REC: Percent recovery.

RPD: Relative percent difference.

NC: Not Calculated

Soil results are reported on a dry weight basis unlesss otherwise noted.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

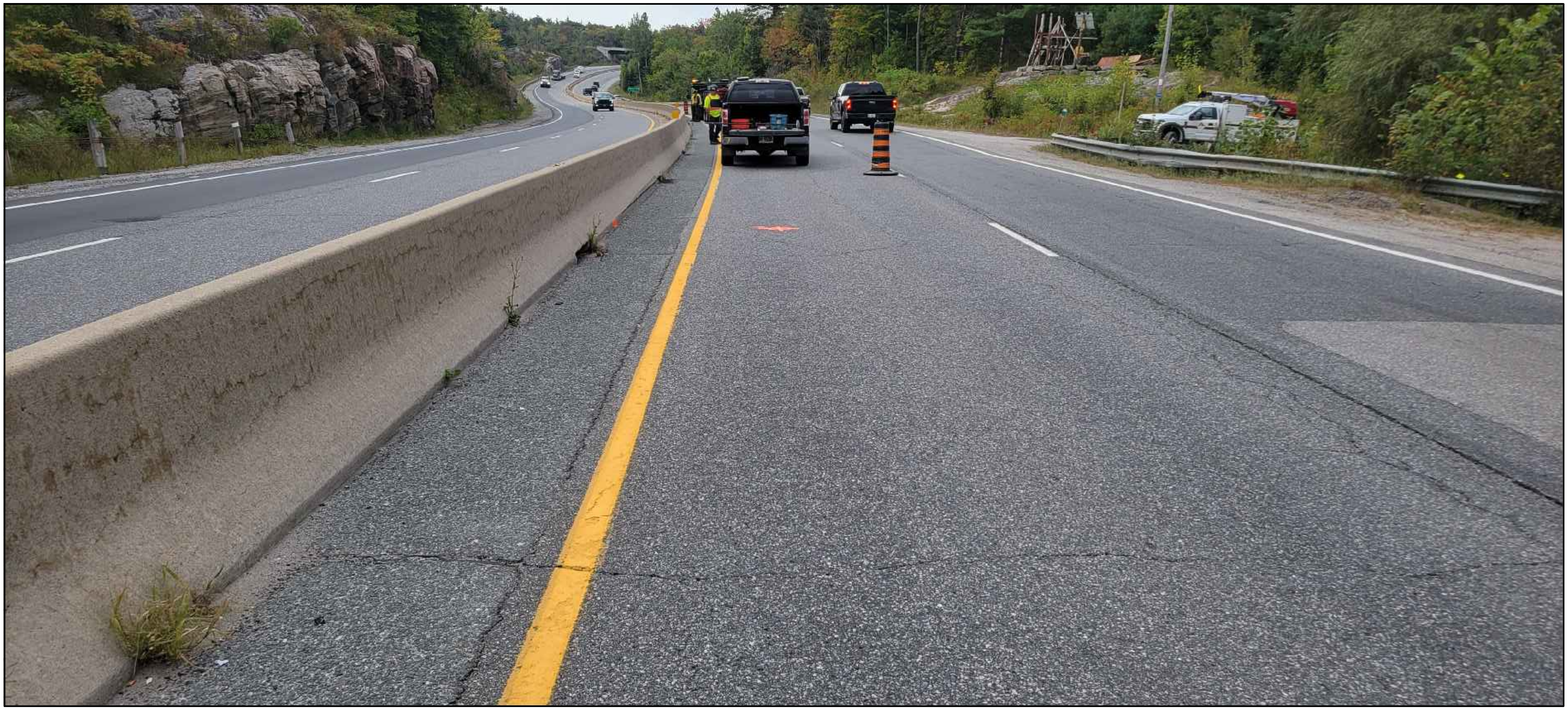
Any use of these results implies your agreement that our total liabilty in connection with this work, however arising, shall be limited to the amount paid by you for this work, and that our employees or agents shall not under any circumstances be liable to you in connection with this work.

# APPENDIX E

Photographs of Project Site

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS
32 Steacie Drive, Ottawa, ON K2K 2A9
T: (613) 836-1422 | www.gemtec.ca | ottawa@gemtec.ca

| Project | FIGURE E1 | |
|---|---|---|
| FOUNDATION INVESTIGATION AND DESIGN REPORT<br>REPLACEMENT OF CULVERT NO. 8<br>STATION 14+900, HIGHWAY 11<br>DISTRICT OF MUSKOKA, ONTARIO | File No.<br>102944.001 | PHOTOGRAPH<br>BOREHOLE 23-01<br>LOOKING NORTH |



GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS
32 Steacie Drive, Ottawa, ON K2K 2A9
T: (613) 836-1422 | www.gemtec.ca | ottawa@gemtec.ca

Project
FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF CULVERT NO. 8
STATION 14+900, HIGHWAY 11
DISTRICT OF MUSKOKA, ONTARIO

**FIGURE E2**

File No.
102944.001

PHOTOGRAPH
BOREHOLE 23-02
LOOKING SOUTH

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS
32 Steacie Drive, Ottawa, ON K2K 2A9
T: (613) 836-1422 | www.gemtec.ca | ottawa@gemtec.ca

Project
FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF CULVERT NO. 8
STATION 14+900, HIGHWAY 11
DISTRICT OF MUSKOKA, ONTARIO

**FIGURE E3**

File No.
102944.001

PHOTOGRAPH
BOREHOLE 23-03
LOOKING NORTH

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

32 Steacie Drive, Ottawa, ON K2K 2A9
T: (613) 836-1422 | www.gemtec.ca | ottawa@gemtec.ca

Project
FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF CULVERT NO. 8
STATION 14+900, HIGHWAY 11
DISTRICT OF MUSKOKA, ONTARIO

**FIGURE E4**

File No.
102944.001

PHOTOGRAPH
BOREHOLE 23-03
LOOKING SOUTH

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS
32 Steacie Drive, Ottawa, ON K2K 2A9
T: (613) 836-1422 | www.gemtec.ca | ottawa@gemtec.ca

Project
FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF CULVERT NO. 8
STATION 14+900, HIGHWAY 11
DISTRICT OF MUSKOKA, ONTARIO

**FIGURE E5**

File No.
102944.001

PHOTOGRAPH
LOOKING AT PROPOSED ALIGNMENT - WEST TO EAST

# APPENDIX F

Non-Standard and Standard Special Provisions

RECOMMENDED WORDING FOR: NSSP – PROTECTION OF SENSITIVE FOUNDATION SOILS

The Contractor is advised that the soil that will be exposed at the subgrade during the construction of the foundation of the culvert is moisture sensitive and may become disturbed or otherwise negatively impacted when subjected to construction or personal traffic, freeze thaw actions, ingress or ponding water. The Contractor shall be responsible for implementing adequate groundwater control measures and to minimize construction and personnel traffic on the founding subgrade and the engineered fill for bedding should be placed and compacted as soon as practical after excavation to subgrade level.

RECOMMENDED WORDING FOR: NSSP – OBSTRUCTION AND SHALLOW BEDROCK

The proposed works will be carried out in ground conditions that include obstructions (cobbles, boulders, blast rock, etc.) and relatively shallow/varying depth to bedrock along the culvert alignment which may affect the efficiency and feasibility of undertaking certain construction techniques.

For example, obstructions may lead to increased difficulty with the installation of temporary protection systems and may impede installation to the required designed depths. The Contractor's installation method and temporary protection system design shall take into account the existing soil and bedrock conditions. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions for installation of the planned system.

**DEWATERING STRUCTURE EXCAVATIONS - Item No.**

Special Provision No. FOUN0003 January 23, 2020

**Amendment to OPSS 902, November 2019**

**902.02 REFERENCES**

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

**Ontario Provincial Standard Specifications, Construction**

OPSS 517Dewatering
OPSS 805Temporary Erosion and Sediment Control Measures

**902.03 DEFINITIONS**

Section 902.03 of OPSS 902 is amended by the addition of the following:

**Automatic Transfer Switch** means as defined in OPSS 517.

**Cofferdam** means as defined in OPSS 539.

**Cut-Off Wall** means as defined in OPSS 517.

**Design Storm Return Period** means as defined in OPSS 517.

**Groundwater Control System** means as defined in OPSS 517.

**Plug** means as defined in OPSS 517.

**Sediment** means as defined in OPSS 517.

**Sediment Control Measure** means as defined in OPSS 517.

**Temporary Flow Passage System** means as defined in OPSS 517.

**Unwatering** means as defined in OPSS 517.

**Vegetated Discharge Area** means as defined in OPSS 517.

**Waterbody** means as defined in OPSS 517.

**Watercourse** means as defined in OPSS 517.

**902.04 DESIGN AND SUBMISSION REQUIREMENTS**

**902.04.01 Design Requirements**

**902.04.01.01 Dewatering**

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

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

The dewatering system shall be according to the design requirements specified in OPSS 517.

**902.04.02 Submission Requirements**

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

**902.04.02.01 Preconstruction Survey**

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of **250** metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

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

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

**902.04.02.02 Working Drawings**

Working Drawings for the dewatering system shall be according to OPSS 517.

**902.07 CONSTRUCTION**

**902.07.04 Dewatering Structure Excavation**

Subsection 902.07.04 of OPSS 902 is amended by the addition of the following clauses:

**902.07.04.01 General**

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

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

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

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

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

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

Unwatering shall be carried out as necessary.

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

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

**902.07.04.02 Discharge of Water**

The discharge of water shall be according to OPSS 517.

**3** **Monitoring** Monitoring shall be according to

OPSS 517. **902.07.04.04System Amendments**

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

**902.07.04.05 Removal**

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

NOTES TO DESIGNER:

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

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

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

CUSTODIAN:Tony Sangiuliano, MERO - Foundation Group.

experience • knowledge • integrity

| | |
|---|---|
| civil | civil |
| geotechnical | géotechnique |
| environmental | environnement |
| structural | structures |
| field services | surveillance de chantier |
| materials testing | service de laboratoire des matériaux |

expérience • connaissance • intégrité