



THURBER ENGINEERING LTD.

FINAL REV.1

**FOUNDATION INVESTIGATION AND DESIGN REPORT
SOUTH BRANCH LOUIS RIVER CULVERT REPLACEMENT
HIGHWAY 652 – 64.4KM NORTH OF HIGHWAY 579
SITE 39E-199/C**

GWP 5170-13-00

Geocres No: 42H-69

Report to:

McIntosh Perry Consulting Engineers Limited

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PART 1. FACTUAL INFORMATION

1 INTRODUCTION

This section of the report presents the factual findings obtained from a foundation investigation completed for the proposed culvert replacement at the Highway 652 crossing of the South Branch Louis River Culvert. The culvert is located approximately 64.4 km north of Highway 579 within Heightington Township. Thurber Engineering Limited (Thurber) carried out the current investigation as a sub-consultant to McIntosh Perry Consulting Engineers Ltd. (MPCE) under Agreement No. 5016-E-0007.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions influencing design and construction was developed in the course of the current investigation.

A previous foundation investigation report that was obtained from the online Geocres library and reviewed in preparation of this report is as follows:

Advanced Foundation Recommendation for the Proposed Structure, Replacements at Louis River (W.P. 7-81-09), Trib. Louis River (W.P. 7-81-08), Heightington Creek (W.P. 7-81-07), Existing Chin River, Detour Lake Road, Group W.P. 1301-80-00 (R), District #16 (Cochrane), dated March 3, 1981.

A review of that document indicated discrepancies with the current alignment, thus the information has not been included in the current report.

2 SITE DESCRIPTION

The existing culvert is a structural corrugated steel plate culvert built in 1983. The Culvert is reported to be 3.7 m in diameter and approximately 28 m long with a generally north to south alignment. The flow through the culvert is to the north.

At the location of the culvert (Linear Highway Referencing System Base Point: 70820, Offset: 7.9), Highway 652 is a two-lane highway with a rural cross-section and gravel shoulders. The Highway 652 fill height above the culvert is approximately 3.4 m with the road surface at approximate elevation 250.1 m. The existing embankment slopes are approximately inclined between 1.5H:1V to 2H:1V. Wooden posts with cable guiderails are present on both sides of the highway in the vicinity of the culvert. The land adjacent to the

highway is generally undeveloped and densely vegetated with shrubs and trees. Traffic volumes on Highway 652 are understood to be 70 AADT (2012).

The historical foundation report, referenced above, indicates that a timber trestle existed at this location.

Select photographs showing the existing conditions in the area of the culvert are included in Appendix D for reference.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing program was carried out between April 25th and May 12th, 2017. The field investigation consisted of advancing six boreholes identified as 17-01 through 17-06. The drilling was carried out using portable equipment for off-road boreholes 17-01, 17-02, 17-05 and 17-06 and a truck mounted CME 750 drill rig for the on-road boreholes 17-03 and 17-04. Prior to commencement of drilling, utility clearances were obtained in the vicinity of the borehole locations.

Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Borehole 17-01, 17-02, 17-05 and 17-06, which were drilled with portable equipment, also utilized a full-weight hammer for SPT testing. In-situ vane shear testing was completed in the cohesive soil deposits. One Thin Walled (Shelby) Tube sample of clay was retrieved from Borehole 17-03 to obtain a relatively undisturbed soil sample. The boreholes were sampled to depths ranging from 5.6 to 18.9 m (elev. 231.2 to 239.0 m) below the existing ground surface. Borehole 17-04 was extended below the base of the sampled borehole with a Dynamic Cone Penetration Test (DCPT) to a base elevation of 217.2 m.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The drilling supervisor logged the boreholes and processed the recovered soil samples for transport for further laboratory examination and testing.

A 32 mm diameter standpipe piezometer was installed in Borehole 17-06 to allow for measurements of the groundwater level after completion of drilling. The piezometer installation details are illustrated on the respective Record of Borehole sheet provided in Appendix B. Following completion of the field investigation and obtaining water level readings, the standpipe piezometer was decommissioned. The boreholes were backfilled in general accordance with MOEE requirements (O.Reg. 903). Boreholes 17-03 and 17-04 were capped with 150 mm of cold patch asphalt to reinstate the traveling surface.

The approximate borehole locations are shown on the Borehole Locations and Soil Strata Drawing included in Appendix A. The coordinates and elevation of the boreholes are provided on this drawing and on the individual Record of Borehole sheets.

4 LABORATORY TESTING

The recovered soil samples were subjected to visual identification and to natural moisture content determination. Selected samples were also subjected to gradation analysis (hydrometer and/or sieve) and Atterberg Limit testing. The results of these tests are summarized on the Record of Borehole sheets included in Appendix B. Two samples of

soil recovered from within Borehole 17-02 and 17-06 were selected and submitted for analytical testing of corrosivity parameters and sulphate content. All laboratory test results are provided in Appendix C.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and the Borehole Location and Soil Strata drawing included in Appendix A. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description for interpretation of the site conditions. It must be recognized that the soil and groundwater conditions may vary between and beyond borehole locations.

In general terms, the site was found to be underlain by a pavement structure and granular fill overlying native deposits of granular overburden and a deposit of native clay. An organic layer was present at the surface of most off-road boreholes, with the exception of Borehole 17-05. Bedrock was not encountered within the depth of investigation.

5.1 Embankment Materials

5.1.1 Prime Surface Treatment

Boreholes 17-03 and 17-04 were drilled through the existing Highway 652 embankment and encountered a layer of prime surface treatment with a thickness of 15 to 20 mm.

5.1.2 Fill: Sand with Gravel

Below the prime surface treatment in Boreholes 17-03 and 17-04 was a layer of fill consisting of sand with gravel. Frequent cobbles were inferred at Borehole 17-03. The upper 2.3 m of this fill material at Borehole 17-03 consisted of silty sand. The underside of the fill was 6.1 m (elev. 244.0 m) below the existing roadway surface in both boreholes.

The SPT tests conducted in this fill gave N-values typically ranging from 13 blows per 300 mm of penetration to 100 blows for 203 mm indicating a relative density of compact to very dense. The higher SPT tests results recorded near the surface may reflect frozen consistency or the presence of cobbles within the fill material.

Recorded moisture contents ranged from 7 to 18%. The results of grain size analyses conducted on two samples of the fill materials are summarized below and are illustrated on Figure C1 in Appendix C.

Soil Particle	Percentage (%)
Gravel	32
Sand	59-65
Fines	3-9

5.2 Peat and Organic Soil

The northern off-road Boreholes 17-01 and 17-02 encountered a layer of soft to very soft fine fibrous peat with a thickness of 0.6 m at ground surface with a base elevation of 244.0 to 244.3 m.

The organic content of the peat in Boreholes 17-01 and 17-02 was measured to be 10.2 %. Recorded moisture contents of the peat was measured to be 63 to 92%.

A layer of organic silt was encountered at the ground surface of borehole 17-06 with a thickness of 0.6 m with a base elevation of 244.3 m.

Recorded moisture content of the organic silt was measured to be 55%.

5.3 Sandy Clay

A layer of sandy clay (CL) was encountered directly below the organic silt layer in Borehole 17-06. It was observed to be 2.4 m thick with a base elevation of 241.9 m.

The SPT N-values ranged from 6 to 19 blows indicating a firm to very stiff material.

The moisture content ranged from 18 to 28%. The results of gradation testing indicated that the one tested sample contained 13% gravel, 36% sand, 31% silt and 20% clay sized particles. The grain size curve is presented on Figure C2 in Appendix C. Atterberg Limit testing on one sample yielded a Liquid Limit of 33% and a Plastic Limit of 19%; the results are included on Figure C6 in Appendix C.

Various amounts of sand and gravel were also encountered within a deeper clay deposit as indicated in boreholes 17-2, 17-3 and 17-4 (Section 5.5).

5.4 Sand and Gravel

A layer of silty sand with gravel to poorly graded sand to poorly graded gravel was encountered below the fill in Boreholes 17-03 and 17-04, below the peat in Boreholes 17-01 and 17-02, at ground surface at Borehole 17-05 and below the low plasticity clay in Borehole 17-06. The thickness of this non-cohesive layer ranged from 1.3 to 4.3 m with the base elevation at 240.4 to 242.6 m. Occasional clayey silt pockets were present throughout this layer. A poorly graded gravel was noted at the base of this unit in Borehole 17-04.

The SPT tests conducted in this non-cohesive layer gave N-values typically ranging from 2 to 33 blows indicating a relative density of very loose to compact.

Recorded moisture contents ranged from 6 to 30%. The results of grain size analyses conducted on five samples are summarized below and are illustrated on Figure C3 in Appendix C.

Soil Particle	Percentage (%)	
Gravel	3 - 59	
Sand	33 - 88	
Silt	24	2-14
Clay	15	

5.5 Clay

A native deposit of clay with trace to some of sand and trace gravel was encountered directly below the granular deposits in all boreholes. Occasional cobbles and boulders may be present within this layer, as they were encountered at Boreholes 17-01, 17-02 and 17-05. The clay across the site consists of interbedded low and intermediate plasticity clays. All boreholes were terminated within the clay layer at an elevation of 231.2 to 239.0 m.

The SPT N-values ranged from weight of hammer to 100 blows. However, the higher SPT tests results may reflect the presence of cobbles and boulders within the clay deposit. Field vane tests were performed within the deposit and recorded undrained shear strengths ranging from 26 to 144 kPa indicating a firm to very stiff consistency. Remolded field vane testing indicates that the clay shows some sensitivity.

The moisture content of the samples tested ranged from 16 to 53%. The results of grain size analyses conducted on ten samples of the native clay are summarized below and are illustrated on Figure C4 and C5 in Appendix C.

Soil Particle	Percentage (%)
Gravel	0 – 7
Sand	3 – 34
Silt	29 – 50
Clay	22 – 57

Atterberg Limit testing was completed on ten samples of the native clay deposit. The results are summarized on the Record of Borehole sheets in Appendix B and the Atterberg Limit graphs are included in Figure C7 and C8 of Appendix C. The laboratory results are summarized below and indicate that the clay varies from low to intermediate plasticity (CL to CI).

Parameter	Value
Liquid Limit	27 – 48
Plastic Limit	14 – 19
Plasticity Index	13 – 28

One Shelby Tube sample was recovered in the native clay from Borehole 17-03 at a depth of 9.4 m. The sample was submitted to Stantec's laboratory in Ottawa, Ontario for extraction. A photograph of the sample is presented in Appendix C. No silt layering or varves were noted.

5.6 Bedrock

Bedrock was not encountered within the depth of investigation, however, Borehole 17-04 was advanced below the sampled borehole with a DCPT to refusal at a depth of 32.9 m (elev. 217.2 m).

5.7 Groundwater

The groundwater level was measured on May 29, 2017 to be 0.8 m (elev. 244.1 m) below the ground surface within the standpipe piezometer installed in Borehole 17-06. The groundwater level was measure at 0.2 m (elev. 244.7 m) below the ground surface at the time of decommissioning the standpipe piezometer on June 12, 2017. The water level at the South Louis River culvert was above the banks of the river at the time of the field investigation (see photographs in Appendix D) and was recorded at elevation 245.2 on April 28th, 2017.

These observations are considered short term and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant and/or prolonged precipitation events.

5.8 Analytical Testing

Two samples of soil were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, resistivity and conductivity. The analysis results are summarized in the table below:

Borehole	Sample	Depth (m)	Sulphate (µg/g)	pH (-)	Resistivity (Ohm-cm)	Chloride (µg/g)
17-02	SS2	0.6 – 1.2	7	7.49	7930	25
17-06	SS2	0.6 – 1.2	9	7.42	6910	14

6 MISCELLANEOUS

Borehole locations were selected by Thurber relative to existing site features and the anticipated foundation locations. The as-drilled locations and ground surface elevation were measured by Thurber following completion of the field program.

George Downing Estate Drilling Ltd. of Hawksbury, Ontario supplied and operated the drilling equipment to conduct the drilling, soil sampling, in-situ testing and borehole decommissioning of the on-road boreholes. Ohlmann Geotechnical Services Inc. of Almonte, Ontario supplied and operated the portable drilling equipment to conduct the drilling, soil sampling, in-situ testing, standpipe piezometer installation and borehole decommissioning of the off-road holes. The field investigation was supervised on a full time basis by Mr. Chris Murray, E.I.T. of Thurber. Overall supervision of the investigation program was conducted by Mr. Stephen Peters, P.Eng.

Routine geotechnical laboratory testing was completed by Thurber's laboratory in Ottawa, Ontario, as well as by Stantec's laboratory in Ottawa, Ontario. Analytical testing was completed by Paracel Laboratories in Ottawa, Ontario. Interpretation of the factual data and preparation of this report were carried out by Miss Katya Edney, P.Eng. and Mr. Stephen Peters P.Eng. The report was reviewed by Dr. Fred Griffiths, P.Eng. and Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundation Projects.

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PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This section of the report provides an interpretation of the factual data from Part 1 of this report and presents geotechnical recommendations to assist the design team in designing a suitable foundation for the proposed replacement of the existing culvert crossing Highway 652 at South Louis River. The discussion and recommendations presented in this report are based on the information provided by McIntosh Perry Consulting Engineers Ltd. (MPCE) and on the factual data obtained during the course of the investigation.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The construction or design-build contractor must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The existing culvert is a structural corrugated steel round culvert reported to be 3.7 m in diameter and approximately 28 m long with a generally north to south alignment. The flow through the culvert is to the north. The invert of the existing culvert is reported to be at elevation 243.3 m. The Highway 652 fill height above the culvert is approximately 3.4 m with the road surface at approximate elevation 250.1 m. It is noted that water in the standpipe piezometer was nearly at ground surface at the time of the site visits and significant precipitation recently had occurred resulting of overtopping of the banks of the river.

The historical foundation report indicates that a timber trestle existed at this location. There exists the possibility that buried remnants remain in the fill and native soils and the Contractors construction methods must consider the possible presence of such obstruction.

7.1 Proposed Structure

The General Arrangement Drawing for this site presented as part of the 60% package indicated that the replacement culvert was to have an interior opening 3 m wide by 3 m high

and an invert elevation of 243.14 m at the outlet. Headwalls or wingwalls are not anticipated for this site.

7.2 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations and existing ground conditions and in accordance with the Canadian Highway Bridge Design Code (CHBDC), version CSA S6-14.

It is understood that a structural culvert replacement would have a consequence classification of *Typical Consequence*, in accordance with Section 6.5.1 of the CHBDC. The geotechnical resistance factor of 0.5 for bearing and 0.8 for settlement, both adopted for typical degree of understanding, were used to obtain the factored resistance values as per CHBDC 2014. If the consequence classification changes, the geotechnical assessment will need to be reviewed and revised.

8 SEISMIC CONSIDERATIONS

8.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth generation seismic model developed by the Geological Survey of Canada (GSC). The seismic hazard for this site has been obtained from the GSC calculator. The data includes a peak ground acceleration (PGA), peak ground velocity (PGV) and the 5% spectral response acceleration values (Sa(T)) for the *reference* ground condition (Site Class C) for a range of periods (T) and for a range of return periods including 475-year, 975-year and 2475-year events. The GSC seismic hazard calculation data sheet for this site is included in Appendix F.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the peak ground acceleration (PGA), which is 0.104g at this site.

8.2 CHBDC 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.

The soil profile at this site has been classified as a Site Class D in accordance with Table 4.1, Section 4.4.3.2 of the CHBDC (S6-14).

8.3 Seismic Liquefaction

Based on the subsurface condition encountered at the drilled locations through the embankment at this site the foundation soils are considered to have a low to moderate susceptibility to liquefaction and cyclic mobility during a seismic event. The consequence of liquefaction would likely be limited to surficial sloughing near the toes of the embankment, which could be readily repaired.

9 DESIGN OPTIONS

9.1 Culvert Type and Foundation Alternatives

Selection of the culvert type must consider the proposed construction procedures, staging requirement, geotechnical resistance available in the foundation soils, the depth to suitable bearing stratum and post-construction settlement criteria. From a geotechnical perspective, the following culvert types were considered:

- Circular Pipes (Concrete, HDPE, Steel)
From a foundation engineering perspective, a pipe culvert is a technically feasible alternative. An internal pipe diameter of equal or greater than that of the current pipe culvert may need to be provided for increased flow capacity and, hydraulic properties. Multiple pipes is another option.
- Open Bottom Culvert (Box, Arch)
An open bottom culvert is not recommended for this site from a foundation engineering perspective due to low available bearing resistance, the high water table and requirement for greater excavation depths to construct the culvert footings and satisfy frost depth requirements. The use of an open bottom culvert would require additional dewatering efforts and has the potential for settlement following construction.
- Closed Bottom Culvert (Box)
A precast segmental box culvert in an open cut excavation is considered a feasible option from a foundation engineering perspective. Precast sections, rather than cast-in-place construction, can be installed expediently with less potential for disturbance of the founding soils during installation.
- Steel Sheet Pile Walls with Precast Concrete Slab
A sheet pile wall supporting precast concrete slabs is feasible but not recommended at this site due to the anticipated depth to a suitable bearing stratum.

A comparison of these alternatives, based on their respective advantages and disadvantages, is included in Appendix E. It is not considered to be economical or practical to support a culvert on deep foundations at this site and therefore this option is not presented in this report.

9.2 Construction Methodology Alternative

For the proposed culvert replacement, the following construction methods were considered.

- Open Cut with Full Road Closure and Detour
Installation of a new culvert using open cut techniques and a full road closure would allow for an expedited construction schedule and could reduce costs associated with requiring roadway protection and water diversion. However, it is understood that an acceptable detour route is not available and therefore this option is not feasible.

- Open Cut with Temporary Modular Bridge

It is considered feasible from a geotechnical perspective to complete a culvert replacement at this site within a full width open cut excavation with a single lane temporary modular bridge spanning the excavation to allow for movement of traffic across the site. Consideration will have to be given to the clearance requirement to determine if this option is constructible. An additional borehole investigation may be required dependant on the location of the temporary abutments relative to the existing borehole locations if this option is pursued further.

- Open Cut with Staged Temporary Widening and/or Lowering

Widening of the existing highway and/or construction of a temporary detour embankment to accommodate a traffic passage during construction has been considered from a geotechnical perspective. Settlement of the foundation soils under the existing embankment and temporary detour embankment should be expected. A review of the requirement for property acquisition and highway geometry will need to be completed to assess this option.

Temporary grade lowering can be incorporated into the design to reduce the overall height of embankment above the base of the proposed excavation while maintaining traffic within the existing embankment footprint. However, the vertical road alignment and traffic speed constraints will need to be reviewed from a highway design perspective.

- Open Cut with Staged Temporary Protection System

The use of open cut techniques in conjunction with staged culvert replacement is a feasible construction option from a geotechnical perspective. This option will require roadway protection, as discussed further in Section 11.2, installed along the embankment centerline to maintain a single lane of traffic along the current highway alignment. Cobbles were inferred in the fill and the potential exists for obstruction within the embankment fill which need to be taken into consideration during the design and installation of roadway protection. Additionally, the potential for remnants of a historic buried trestle exists. To reduce lateral deflections of the protection system, the roadway protection may need to include a strutting or bracing system. The height of the TPS could be reduced if a temporary grade lowering was also included.

- Trenchless Techniques

Trenchless techniques would have the advantage of minimum disruption to traffic and would avoid an excavation through the existing highway embankment. However, the anticipated size of the replacement culvert will limit the available installation methods and the available cover above the culvert will need to be reviewed once the size of culvert and invert elevations have been determined. There exists a potential for loose cohesionless soils and the presence of cobbles and boulders within the embankment fill, which may also limit the available techniques to closed faced systems. Furthermore, free water was observed at the inlet and outlet and would require cofferdams around the entry and exit points. A trenchless culvert installation is not recommended at this site.

9.3 Recommended Approach for the Culvert Replacement

From a foundation engineering perspective, replacing the existing culvert with either a circular or a closed box culvert using open cut techniques is the recommended culvert replacement option. Temporary protection systems (TPS) would be needed to facilitate construction. Design of the TPS will need to account for the lateral capacity available in the foundation soils at this site. Grade lowering could be considered to reduce the height of the TPS.

10 FOUNDATION DESIGN RECOMMENDATIONS

Foundation design aspects for the replacement culvert include subgrade conditions, geotechnical resistance, settlement of the founding soils, imposed loading pressures, erosion control, protection system design, groundwater control and stability of stage construction. The culvert must be designed to resist loadings including lateral earth pressures, hydrostatic pressure, weight of embankment fill, traffic loading and any surcharge due to construction equipment and activities under static and seismic conditions.

10.1 Culvert Foundation Bearing Resistances

Provided the replacement culvert is constructed on the same alignment with a similar opening size as the existing culvert and the embankment is reconstructed with no grade raise or widening (temporary or permanent), it is anticipated that the subgrade soils within the culvert footprint will not be subjected to any significant additional loading.

10.1.1 Box Culvert

The recommended geotechnical resistances for a 3.0 m wide (interior) pre-cast box culvert installed on a bedding layer at or below the founding elevation of the current culvert (approximate elev. 243.3 m) on an undisturbed native sand or clay subgrade, with the appropriate subgrade preparations (Section 11.3) are as follows:

Culvert End/Embankment Toe:

- Factored Geotechnical Resistance at ULS of 150 kPa
- Factored Geotechnical Resistance at SLS of 90 kPa

Embankment Centerline:

- Factored Geotechnical Resistance at ULS of 195 kPa
- Factored Geotechnical Resistance at SLS of 145 kPa

The factored geotechnical resistances include the following factors:

- Consequence factor (Ψ) of 1.0 (as per CHBDC Table 6.1)
- Geotechnical resistance factors (as per CHBDC Table 6.2):
 - $\phi_{gu} = 0.5$ (static analysis; typical degree of understanding)
 - $\phi_{gs} = 0.8$ (static analysis; typical degree of understanding)

The bearing resistance values are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be reduced in accordance with CHBDC Clause 6.10.3 and Clause 6.10.4. Foundation settlement, based on the supplied SLS

resistance, is expected to be less than 25 mm. Organic soils consisting of peat and organic silt will be encountered in the area of the inlet and outlet. The bearing resistances provided above are based on the assumption that this organic material, where encountered at the subgrade layer, must be removed down to the competent inorganic soils and replaced with well compacted granular fill. **In addition, extension of the culvert length with embankment widening beyond the current toe of slope, either permanent or temporary, is not recommended as it would induce settlement at the culvert inlet and outlet.**

Resistance to lateral forces/sliding resistance between the precast concrete and the underlying Granular 'A' bedding (Section 10.2) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.45.

Surface water diversion and dewatering (Section 11.3) will be required to place the bedding material and install the culvert in the dry.

10.1.2 Pipe Culvert

If a pipe culvert is selected with an open cut installation technique it should be designed and constructed in accordance with OPSS 421, OPSD 802.010 (with Granular A used as bedding and embedment material) and OPSD 803.031 (with a frost depth as noted in Section 10.3). The recommendations of Sections 10.2, 10.5, 10.7, and 11 should be applied. Geotechnical resistance values are not required for pipe culverts. The culvert should be founded at or below elevation 243.3 m on the compact native undisturbed sand or clay with the appropriate subgrade preparations. A modulus of subgrade reaction of 15 MN/m³ can be used for a pipe culvert installed at this site.

10.2 Subgrade Preparation, Bedding and Backfilling

After excavation and removal of the existing culvert and exiting fill, all organics, peat, soft or loose deposits, disturbed soils, alluvial deposits and deleterious materials must be stripped from the culvert footprint to expose competent native subgrade material at or below the desired founding elevations. Given the potential loose to compact silty sand and firm consistency of clay materials anticipated at the founding level of the replacement culvert, construction equipment should not be permitted to travel on the exposed subgrade.

The exposed subgrade may vary across the site and must be inspected to confirm that the subgrade is suitable and uniformly competent. Any soft or organic materials at the subgrade level should be sub-excavated and backfilled and compacted as per OPSS.PROV 501 with granular fill consisting of OPSS.PROV 1010 Granular A material as soon as practical to protect the subgrade from disturbance during construction. In order to provide a more uniform foundation subgrade condition for the culvert on clay soils, a minimum 0.5 m thick layer of well compacted bedding material conforming to OPSS.PROV 1010 Granular A requirements must be provided under the base of the culvert as per OPSS 422 and OPSD 803.010 (box culvert) and OPSD 802.010 (pipe culvert) unless loose/soft or organic deposits are encountered at the founding elevation where sub-excavation will then be required as recommended, above.

The compaction of granular bedding directly above the subgrade may result in disturbance of the material with pumping of fines into the granular bedding and difficulty achieving the specified degree of compaction. Protection of the subgrade should include installation of

Class II a non-woven geotextile with a maximum FOS of 150 µm (OPSS 1860) installed beneath the 0.5 m Granular A bedding layer. The geotextile should be placed as soon as possible after reaching the subgrade level and following receipt of written notice to proceed in accordance with SP 109S12. An NSSP is provided in Appendix G to include in the contract documents to alert the Contractor of the sensitive nature of the foundation soils.

It is noted that construction will extend below the creek elevation. Water diversion and dewatering will be required to prepare the subgrade in the dry. Please refer to Section 11.3 for additional comments on groundwater and surface water control.

It is recommended that culvert cover be in accordance with OPSS 902 and consist of free-draining, non-frost susceptible granular materials such as Granular A or Granular B Type II material meeting the requirements of OPSS.PROV 1010.

Culvert backfill above the granular cover should be in accordance with OPSS 902 and consist of material meeting the requirements of OPSS Select Subgrade Material or better and should be compacted in regular lifts as per OPSS.PROV 501. Heavy compaction equipment, used adjacent to the culvert, must be restricted in accordance with OPSS.PROV 501. Care must be exercised when compacting the fill adjacent to and above the culvert in order not to damage the culvert.

10.3 Frost Depth

The depth of frost penetration at this site is 2.4 m. It is not necessary to found a closed box or pipe culvert at a depth below frost penetration. However, frost taper treatment should be as per OPSD 803.010 (box culvert) or OPSD 803.031 (pipe culvert) and as directed within the Pavement Design Report.

10.4 Lateral Earth Pressures

The lateral earth pressures parameters provided in Table 10-1 and Table 10-2 below are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in design.

For design purposes, the groundwater level should be assumed to be at elevation 244.7 m or creek level, whichever is higher.

10.4.1 Static Lateral Earth Pressures

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the following expression:

$$p_h = K * (\gamma h + q)$$

where:

p_h	=	horizontal pressure on the wall at depth h (kPa)
K	=	earth pressure coefficient (see table below)
γ	=	unit weight of retained soil (see table below, to be adjusted for groundwater depth)
h	=	depth below top of fill where pressure is computed (m)
q	=	value of any surcharge (kPa)

A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12.3 of the CHBDC. Typical earth pressure coefficients for backfill are shown in Table 10-1.

Table 10-1. Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		OPSS SSM and Existing Sand Fill $\phi = 30^\circ, \gamma = 21.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active, K_A (Yielding Wall)	0.27	0.40	0.31	0.48	0.33	0.54
At Rest, K_o (Non-Yielding Wall)	0.43	-	0.47	-	0.50	-
Passive, K_P (Movement towards Soil Mass)	3.7	-	3.3	-	3.0	-
Soil Group(*)	"medium dense sand"		"loose to medium dense sand"		"loose sand"	

Note: (*) Figure C6.16 of the Commentary to the CHBDC.

The use of a material with a high friction angle and low active pressure coefficient (Granular A or Granular B Type II) is preferred as it results in lower earth pressures acting on the culvert.

The parameters in the table correspond to full mobilization of active and passive earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. The values to be used in design can be assessed from Figure C6.16 of the Commentary to the CHBDC using the soil group designation as outlined in Table 10-1. Active pressures should be used for any head walls or unrestrained walls. Where ground surfaces are sloped behind the walls, the corresponding coefficients provided in the Table 10-1 should be used.

10.4.2 Combined Static and Seismic Lateral Earth Pressure Parameters

In accordance with Clause 4.6.5 of the CHBDC (S6-14), a structure should be designed using dynamic earth pressure coefficients that incorporate the effects of earthquake loading. The following recommendations are per Section C4.6.5 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using Mononobe-Okabe Method with:

- $k_h = \frac{1}{2} * F(PGA) * PGA$, for structures that allow 25 to 50 mm of movement, and
- $k_h = F(PGA) * PGA$, for non-yielding walls

The ratio of wall movement to wall height required to mobilize the active conditions would be approximately 0.002 for a yielding structure with respect to the assessment of seismically induced lateral earth pressures.

The coefficients of horizontal earth pressure for seismic loading presented in Table 10-2 may be used. The provided earth pressure coefficients are based on a Seismic Site Class D, PGA with a 2% probability of exceedance in 50 years of 0.102g (Geological Survey of Canada – Fifth Generation) and a $F(PGA)$ of 1.80 as per Table 4.8 of the CHBDC (S6-14 update No. 1, April 2016).

Table 10-2. Dynamic Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Slope Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Slope Surface Behind Wall (2H:1V)
Active, K_{AE} Yielding Wall	0.32	0.48	0.35	0.58
Active, K_{AE} Non-Yielding Wall	0.38	0.65	0.43	0.78

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soils profile.

$$\sigma_h = K * \gamma * d + (K_{AE} - K) * \gamma * (H - d)$$

where:

σ_h	=	lateral earth pressure at depth d (kPa)
d	=	depth below the top of the wall (m)
K	=	static earth pressure coefficient (K_a for yielding walls, K_o for non-yielding walls)
γ	=	unit weight of retained soil (to be adjusted for groundwater depth)
K_{AE}	=	combined static and seismic earth pressure coefficient
H	=	total height of the wall (m)

10.5 Embankment Design and Reinstatement

10.5.1 Embankment Reconstruction

Embankment reconstruction after culvert replacement should be carried out in accordance with OPSS.PROV 206. It is understood that the embankment will be reinstated to match the existing embankment with side slopes as steep as 1.5H:1V. The embankment must therefore be reinstated with Granular B Type II or rockfill. The fill should be placed and compacted in accordance with OPSS.PROV 501. Due to the steepness of the requested embankment slope, gravel sheeting or rock protection as per OPSS 511 will be required to reduce erosion of the slope face.

The existing sand fill material can be reused as backfill in the areas above the culvert cover/embedment provided the material is free of organics, asphalt pieces, timber pieces, unfrozen and there is sufficient space to stockpile adjacent to the embankment footprint and control the moisture within acceptable limits for compaction.

Where new embankment fill is placed against existing embankment slopes or on a sloping ground surface steeper than 3H:1V, benching of the existing slope should be carried out in accordance with OPSD 208.010.

10.5.2 Embankment Settlement and Stability

The condition of the existing embankment slopes was examined in the field during the field investigation and no evidence of instability (tension cracks etc.) was noted at that time.

Provided no grade raise or embankment widening is required and proper construction methods are used, no long term or global stability issues are anticipated for reinstated embankments at this site. Material stockpiling above the existing grades is a temporary construction measure and the stability implication should be reviewed by the Contractor. The selection and placement of construction equipment (such as cranes) is also the Contractor's responsibility.

It is understood that no grade raise is anticipated along the alignment of Highway 652 and therefore negligible foundation settlement is expected to occur.

The magnitude of the embankment compression constructed with granular materials is in the order of 0.5% of the embankment height and is expected to occur following fill placement.

10.6 Temporary Detour

A foundation investigation was not completed for a temporary detour embankment as part of the current assignment. If construction staging dictates that a temporary detour embankment is needed, additional field investigations with recommendations may be required.

10.7 Cement Type and Corrosion Potential

Analytical tests were completed to determine the potential for degradation of concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel. 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. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. The class of concrete selected should consider the effects of road de-icing salts.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The tests results provided in Section 5.8 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The corrosive effects of road de-icing salts should also be considered.

11 CONSTRUCTION CONSIDERATIONS

11.1 Excavation

Excavation for the culvert replacement must be carried out in accordance with OPSS 902 and will be carried out through the existing embankment fill and extend into the underlying native sand deposits. Excavation to deeper than underside of bedding will likely be required at the culvert ends to remove deposits of organics soils (see Sections 10.1 and 10.2).

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of OHSA, the fills above the water table may be classified as Type 3 soil. The fills below the water table and all native soils may be classified as Type 4 soil.

At locations where there are space restrictions or where a slope has to be retained, the excavations will need to be carried out within a protection system. Further discussion is presented in Section 11.2.

11.2 Temporary Protection Systems

Temporary Protection Systems will be required during various stages of construction and must be implemented in accordance with OPSS.PROV 539 and designed for Performance Level 2 (maximum 25 mm horizontal deflection). The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system. The protection system should be installed at a suitable distance away from the new culvert to limit the disturbance to subgrade associated with removal of the protection system following completing of construction. Alternatively, the protection system could be left in place and cut off at or below 1.2 m beneath the finished grades as per OPSS 903. To reduce the

potential for ground movements vibratory equipment should not be permitted at this site for installation or removal of the temporary protection systems. Suggested wording for an NSSP is provided in Appendix G.

Lateral earth pressure coefficients, under fully mobilized conditions, that can be used in design of the protection system installed through embankment fill and culvert backfill are provided in Table 10-1. The lateral earth pressure coefficients for the existing native clay soils are given below:

$$\begin{array}{lll} \gamma & = & 18 \text{ kN/m}^3 \quad (\text{must be adjusted for water table}) \\ K_A & = & 0.36 \\ K_P & = & 2.77 \end{array}$$

Temporary protection systems are the responsibility of the Contractor and should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. A suitable strutting or bracing system may need to be incorporated into the roadway protection design to resist the lateral loadings including traffic loading and surcharge loading due to construction equipment and operations. The Contractor must undertake an assessment of the foundation soils ability to support the weight of the crane used during installation of the protection system.

11.3 Surface and Groundwater Control

Culvert construction, subgrade preparation and placement and compaction of granular bedding must be carried out in the dry. The Contractor must be prepared to control the groundwater and surface water flow at this site to permit construction in a dry and stable excavation. Temporary groundwater and surface water control measures will be required to remain operational during construction until the culvert is installed and backfilled. Dewatering systems must be designed, operated and removed in accordance with OPSS.PROV 517 and Special Provision No. 517F01 with the following inputs for Table A: Note 1 = Yes and Note 2 = N/A.

The groundwater level will fluctuate and the minimum groundwater elevation for the site at the time of the proposed culvert replacement should be taken as the water level in the creek at the time of construction as defined by SP517F01. Excavation below the groundwater level to construct the culvert foundation will be required and excavation below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause base heave/boiling and sloughing of the loose granular foundation soil below the water level, making it difficult to maintain a dry, sound base on which to work. The Contractor should be prepared to lower the groundwater level prior to each stage of excavation. The groundwater level within the culvert footprint should be lowered by pumping from sumps prior to each stage of excavation to at least 500 mm below the underside of the planned base of excavation.

Construction of cofferdams will be required to divert flow away from the culvert subgrade area. A sheet piled cofferdam can be designed following the recommendation provided in Sections 11.1 and 11.2. It is recommended that excavation be enclosed within a water tight enclosure extending into the native clay.

Further assessment of dewatering requirements and the need for a PTTW should be carried out by specialists experienced in this field.

11.4 Scour Protection and Erosion Control

Based on the subsurface conditions encountered at the drilled locations through the embankment at this site the embankment materials soils are considered to have low susceptibility to erosion as per the Wischmeier Nomograph. The native soils at the inlet and outlet are considered to have low to moderate susceptibility to erosion.

Scour and erosion protection should be provided for the culvert inlet and outlet areas. Design of the scour and erosion protection measures must consider hydrologic and hydraulic concerns and should be carried out by specialists experienced in this field.

Typically, rock protection should be provided over all earth surfaces subjected to flowing water. Due to the steepness of the requested slope, the area requiring slope protection should be extended. Treatment at the outlet should be in accordance with OPSD 810.010. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

It is recommended that a concrete cut-off wall be used to minimize the potential for piping and erosion around the inlet of the culvert.

12 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- Disturbance of the soil subgrade. The water level may be above the banks of the river at the time of construction, resulting in moisture sensitive subgrade conditions and may become heavily disturbed when subjected to construction traffic. Site and subgrade drainage will be critical to maintain subgrade conditions. The Contractor must be aware of the issue so that he may adjust his operations to suit the subgrade conditions
- Cobbles were inferred while drilling and although not encountered during drilling, buried obstructions such as historical timber trestles may also be encountered during excavation in the embankment fill or interfere with driving of protection systems.
- Groundwater levels may fluctuate. Excavation will involve lowering the groundwater level below the excavation base to maintain a dry excavation and stable side slopes.
- The Contractor's selection of construction equipment and methodology must include assessment of the capability of the existing embankment to support the proposed construction equipment and any temporary structure fill (i.e., as a pad for crane support).

The successful performance of the culvert will depend largely upon good workmanship and quality control during construction. Subgrade examination should be carried out by qualified geotechnical personnel during construction to confirm that foundation recommendations are correctly implemented and material specifications are met.

13 CLOSURE

Engineering analysis and preparation of this report were carried out by Miss Katya Edney, P.Eng. and Mr. Stephen Peters, P.Eng. The report was reviewed by Dr Fred Griffiths, P.Eng a Designated Principal Contact for MTO Foundation Projects.

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