

Part 1 Foundation Investigation

1.1 Introduction

This submission presents the results of a geotechnical investigation completed by Trow Associates Inc. (Trow) for the replacement of the Driver Creek Culvert (3350 mm by 2290 mm by approximately 26.0 m long Structural Plate Pipe Arch (SPPA), located on Highway 522 at Station 12+250 within Pringle Township. The culvert replacement is to consist of a pre-cast concrete box culvert 2400 mm wide by 2100 mm high and approximately 26.0 m long. Photographs of the site are included in Appendix A.

The purpose of this geotechnical investigation was to determine the existing soil conditions within the proposed construction limits by field investigation and laboratory testing.

The MTO's explanation of terms, abbreviations and symbols are included in Appendix C.

1.2 Site Description and Geological Setting

1.2.1 Site Description

The Driver Creek Culvert is located in the Pringle Township at Station 12+250 on Highway 522.

The site plan and cross section profile of the Driver Creek Culvert are as shown on Sheets No. 1 and 2 in Appendix B.

The overall terrain in the area consists of undifferentiated igneous and metamorphic rock, exposed at the surface or covered by a discontinuous layer of drift. The vegetation in the area consists mainly of deciduous trees, some coniferous trees and smaller low lying shrubs and grass. The drainage in the area generally consists of roadside ditches which drain into Driver Creek.

1.2.2 Geological Setting

According to the Ontario Geological Survey (OGS) Maps 2544 and 2556, the site is located in the Mesoproterozoic era within the central gneiss belt, which falls under the mafic rocks, amphibolite, gabbro, diorite and mafic gneisses. The topography in the area consists of undulating bedrock outcrops separated by intervening marshy zones and wooded areas. As such, the surface soils in the area consist of intervening shallow organic deposits (peat), with fluvial deposits consisting of gravel, sand, silt and clay.

1.3 Investigative Procedures

1.3.1 General

The fieldwork for this project was carried out from June 13th to June 14th, 2006. The investigation consisted of a total of 2 boreholes (BH-1 and BH-2). Borehole BH-1 was drilled at the culvert outlet (northwest end of culvert), to verify the soil conditions below the existing culvert. Borehole BH-2 was drilled at the southeast side of the existing culvert embankment to verify embankment fill materials and soil conditions below the existing culvert.

All boreholes were advanced with a Mobile CME-55 track mounted drill rig equipped with continuous flight hollow stem augers and standard soil sampling equipment. All boreholes were advanced by Landcore Drilling.

From the drilling program, soil samples were obtained using a 51 mm (2 inch) outside diameter split spoon sampler in conjunction with Standard Penetration Tests (ASTM D 1586), at 0.75 m intervals for the upper 3.0 m and at 1.5 m intervals thereafter. The Standard Penetration Test “N” values were recorded and used to provide an assessment of the in-situ relative density of the overburden soils. All boreholes were backfilled with auger cuttings and sealed with bentonite pellets.

All fieldwork was supervised by a member of Trow’s engineering staff who directed the drilling and sampling operations, logged the factual borehole data, and retrieved soil samples for subsequent laboratory testing and identification. All geodetic borehole elevations were determined in the field by Sutcliffe Rody Quesnel (SRQ). The location of the boreholes and geodetic elevations are shown on Sheet 1, with a cross-section of the boreholes on Sheet 2 in Appendix B.

1.4 Laboratory

The soil samples obtained in the field were carefully transported to our Sudbury laboratory and examined for further verification and classification. A laboratory testing program for the selected soil consisted of Natural Moisture Content Determination (LS 701), Particle Size Analyses (LS 702), Liquid Limit (LS 703) and Plastic Limit and Plasticity Index (LS 704).

The laboratory test results are summarized on the attached borehole logs in Appendix C, as well as in Appendix D.

1.5 Subsurface Conditions

1.5.1 General

The subsurface conditions encountered during the field investigation are summarized on the attached borehole logs in Appendix C. The following is a description of the subsurface conditions encountered during the field investigation.

1.5.2 Stratigraphy, Culvert Outlet

In general, the stratigraphy within borehole BH-1 at the culvert outlet consisted of interlayered silty clay, sand, and silt overlying sand and silt, silty sand till and bedrock.

The interlayered silty clay, sand, and silt were encountered from the ground surface to approximately 7.6 m below existing grade. The sand and silt extend from 7.6 m below grade to 10.7 m below grade and the silty sand till extended from 10.7 to 11.1 m below grade where bedrock was encountered. The silty clay was brown to grey in colour, moist, very soft to soft in consistency and of medium plasticity. The sand was brown to grey in colour, damp above 4.6 m depth and wet below, very loose to loose, well graded, fine to coarse grained and contained trace fine grained gravel, trace to with silt, and trace organics. The silt material was grey in colour, damp, very loose, and contained some fine to coarse grained sand, trace organics and trace fine grained gravel. The sand and silt was grey in colour, wet, very loose above 9.10 m depth and compact below, poorly graded, fine to coarse grained and contained trace fine gravel. Uncorrected Standard Penetration Tests (SPT) “N” values within the interlayered material and the sand and silt material ranged from 0 to 6 blows per 300 mm above approximately 9.1 m depth and were 16 blows per 300 mm below 9.1 m depth. The underlying silty sand till was grey in colour, wet, very dense, poorly graded, fine to coarse grained and contained some fine to coarse grained gravel. The uncorrected SPT “N” value within the silty sand till was 57 blows per 300 mm.

Bedrock was encountered between underlying the silty sand till at approximately 11.1 m below existing grade. A 3.3 m core was taken from the bedrock. The bedrock consisted of granite and gneiss. The bedrock was fresh, intact, light grey to pink in colour, fine to medium grained and very strong. The Rock Quality Designation (RQD) was 100%, indicating an excellent quality of rock. The RQD is also included in the attached borehole logs in Appendix C.

1.5.3 Stratigraphy, Southeast Embankment

In general, the stratigraphy within boreholes BH-2 at the southeast embankment consisted of sand fill overlying native sand and sand till. The sand fill was encountered from the ground surface to approximately 3.1 m below grade. The sand fill was brown in colour, dry above 2.3 m depth and damp below, loose to compact, well graded, fine to coarse grained and contained trace to with fine grained gravel and some to with silt. Uncorrected SPT “N” values within the sand fill ranged from 5 to 11 blows per 300 mm. Underlying the sand fill was a 4.5 m thick layer of native sand which extended to approximately 7.6 m below grade. The native sand material was brown to grey in colour, wet, very loose to loose, above 6.10 m depth and compact below well graded, fine to coarse grained and contained trace fine grained gravel and some silt. Uncorrected SPT “N” values within the native sand ranged from 0 to 19 blows per 300 mm. Underlying the native sand material was a 1.9 m thick layer of sand till which extended to approximately 9.5 m below grade. The sand till overlaid suspected bedrock, where SPT refusal was encountered (i.e. >100 blows per 300 mm). The sand till was grey in colour, compact to very dense, wet, poorly graded, fine to coarse grained, and contained trace to some fine to coarse grained gravel and trace to with silt. Uncorrected SPT “N” values within the sand till ranged from 24 to 100 blows per 300 mm.

1.6 Groundwater Conditions

The groundwater elevations observed within boreholes BH-1 and BH-2 were between Elevations 221.42 m and 222.31 m. The lower water levels within the boreholes could be due to disturbance in the holes at the time of drilling and that the boreholes had not stabilized prior to backfilling. As such, for design purposes the groundwater level should be assumed to be equal to the creek water elevation, which was 223.91 m at the time of the investigation.

Seasonal variations in the water table should be anticipated, with higher levels occurring during wetter periods of the year (such as spring thaw and late fall) and lower levels during drier periods.

Part 2 Engineering Discussions and Recommendations

2.1 Introduction

The following subsections address the geotechnical design and construction considerations for the proposed Driver Creek culvert (2400 mm wide by 2100 mm high pre-cast concrete box culvert) located on Highway 522 at Station 12+250 within Pringle Township. The new culvert is to be approximately 26.0 m long. Photographs are included in Appendix A.

2.2 Culvert Replacement at Driver Creek Highway 522

It is understood by Trow that the existing 3350 mm by 2290 mm by 26.0 m long Structural Plate Pipe Arch (SPPA) culvert is to be replaced with a pre-cast concrete box culvert 2400 mm wide by 2100 mm high. The proposed invert will be placed at approximately the same invert as the existing culvert between elevations 223.17 m (northwest-outlet) and 223.37 m (southeast-inlet).

It is recommended that the footings for the proposed culvert be founded near the existing culvert between Elevations 223.37 m (inlet) and 223.17 m (outlet). At these elevations the proposed culvert will be founded on the insitu native sand material.

For the proposed culvert founded on the native sand, a Factored Bearing Resistance at ULS of 250 kPa and a Factored Bearing Resistance at SLS of 100 kPa is recommended in accordance with the Canadian Highway Bridge Design Code Section 6.7, Shallow Foundations. Prior to the placement of the culvert, the exposed material must be cleared of any soft, loose or disturbed soil. Any loose areas are to be sub-excavated and replaced with Granular "A" or Granular "B" Type II (OPSS 1010) compacted to a minimum of 100% of the Standard Proctor Maximum Dry Density (SPMDD). The groundwater level needs to be controlled below excavation levels to avoid disturbance, and any surface or groundwater seepage should be removed from within the excavation prior to the culvert replacement to allow placement of granular backfill in dry conditions. A non-woven geotextile separator (Terrafix 270R or equivalent) is to be used between the subgrade soils and the Granular "A" to stabilize the native soils.

The anticipated maximum total settlements for the concrete box culvert are not expected to exceed 25 mm, for construction done in accordance with design parameters and assuming good construction practice.

2.2.1 Culvert Bedding

The culvert bedding should consist of Granular “A” (OPSS 1010) with a minimum thickness of 300 mm beneath the culvert and extend a minimum of 300 mm on either side of the culverts edge and slope down at 1H:1V. The granular material should be compacted to 100% of the SPMDD in 150 mm thick lifts and placed in dry conditions. If construction proceeds during the winter months, the base of the trench should not be allowed to freeze prior to placing the bedding material. In areas where the base of the trench experiences loose or soft material, the area may have to be sub-excavated and the Granular “A” thickness increased to stabilize the trench base.

Prior to the placement of any fill material, the native sand is to be relatively level and visually inspected by a qualified geotechnical engineer.

2.2.2 Culvert Backfill

All potential organics and other deleterious material should be excavated as outlined in OPSD 803.010, attached in Appendix E. The culvert backfill should consist of stone free Granular “B”, Type I or Granular “A” (OPSS 1010) placed in maximum 150 mm lifts kept at the same elevation on both sides of the culvert. The granular backfill should be compacted to 100% of SPMDD.

The culvert should be encased with a minimum of 300 mm of compacted material. Typical backfill diagrams are presented in Appendix E, OPSD 803.010. The minimum height of fill over the top of the culvert for heavy equipment during construction shall be 1000 mm, unless otherwise noted by the structural engineer. In addition the Contractor is to follow SP No. 902S01, regarding backfilling for structures.

2.2.3 Lateral Earth Pressure

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

$$p = K (\gamma h + q)$$

where p = Lateral earth pressure (kPa).

K = Coefficient of earth pressure.

γ = Unit weight of backfill (kN/m³).

h = Depth to point of interest (m).

q = Surcharge load acting adjacent to the wall at the ground surface (kPa).

The above expression does not take into account hydrostatic pressure, which must be included for the groundwater level at existing ground surface.

Table 1 below lists various earth pressure properties for given materials.

Table 1 - Material Types and Earth Pressure Properties

Material	Friction Angle ϕ (unfactored)	Coefficient of Active Earth Pressure (k_a)	Coefficient of Passive Earth Pressure (k_p)	Coefficient of Earth Pressure at Rest (k_0)	Unit Weight γ (kN/m ³)
Granular A	35°	0.27	3.7	0.43	22
Granular B Type I	32°	0.31	3.3	0.47	21
Granular B Type II	35°	0.27	3.7	0.43	21
Rock Fill	42°	0.2	5.0	0.33	21

Note: Values given for horizontal earth pressures are for horizontal backfill. For sloping backfill, the design requirements outlined in Sec C6.9.1(c) of the Canadian Highway Bridge Design Code should be used. A unit weight of $\gamma=21$ kN/m³ is based on well graded rockfill.

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

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

2.2.4 Design Parameters

The design of the culvert is based on the following soil parameters as outlined in Table 2.

Table 2 - Material Types and Strength Parameters

Material	Friction Angle ϕ	Cohesion c' (kPa)	Unit Weight γ (kN/m ³)
Granular A	35°	0	22
Granular B Type I	32°	0	21
Granular B Type II	35°	0	21
Sand	32°	0	21
Sand and Silt	30°	0	21
Silt	28°	1	20
Till: Silty Sand / Sand	38°	0	22

2.2.5 Sliding Resistance

A friction angle, θ' , of 30° can be used for sliding resistance along the Granular “A” and the pre-cast concrete culvert footings proposed for this site and 32 degrees for cast in place concrete culvert footings if applicable.

2.2.6 Erosion Protection Outlet

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

2.2.7 Erosion Protection Inlet

Rip-rap protection should be provided where the open creek enters the culvert. The rip rap should begin approximately 5 m before the culvert inlet and line the embankment slope to the spring line of the culvert. The size of the rip-rap is a function of the creeks hydrology. As a rule of thumb the thickness of the rip-rap should be a minimum of twice the median particle size, and 300 mm thick as a minimum. The rip-rap configuration at the creek bed should generally follow the OPSD 810.010, which is included in Appendix E of this report. Rip-rap placed at 1V:1H will be stable.

Where rip-rap is not present the embankment side slopes are to be vegetated with sodding, seeding or planting as necessary depending on the flow rate and volume. Should seeding be utilized, a 100 mm thick layer of topsoil should be placed along with a degradable erosion blanket to help minimize erosion until the vegetation begins to grow.

2.2.8 Clay Seal

A clay seal should be placed at the inlet of the proposed culvert, to prevent the migration of material along the face of the culvert, the formation of flow paths, and any potential internal erosion within the highway embankment. The following outlines the installation procedures and minimum material requirement of the clay seal:

- The clay seal should be placed against the constructed embankment, and subsequently protected by the inlet erosion protection extending a minimum of 1.0 m along the side of the culvert and extending out laterally 1.0 m from the culvert.
- The clay seal should be placed from the top of the culvert footings and extend along the side and the top of the culvert.
- The clay should have a Liquid Limit greater than 50% and a Plasticity Index greater than 17.5%.
- The clay seal is to be place in maximum 150 mm thick lifts and compacted to 95% SPMDD within 2% of the optimum moisture content.

2.2.9 Stream Bed Rip-Rap

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

2.2.10 Frost Protection

A frost penetration depth of up to 1.8 m can occur in open areas in the Pringle Township area without snow cover. The underlying sand, sand fill and silty clay material is considered to have low to moderate susceptibility to frost heaving, according to the MTO Guidelines for Soil Frost Susceptibility. Therefore there is potential for frost heave near the inlet and outlet of the pipe. To minimize potential movements, the frost protection treatment as outlined in OPSD 803.030 and 803.031 included in Appendix E of this report may be applied.

2.2.11 Excavations

All excavations must be conducted in accordance with the Occupational Health and Safety Act and Regulations for Construction (OHSARC). The sand fill, interlayered silty clay, sand, and silt material may be classified as Type 3 soils above the groundwater table and Type 4 soils below the groundwater table in conformance with the OHSARC. Excavations are expected to be below the groundwater levels as measured in the creek at the time of this investigation. To avoid disturbance of the founding materials and to allow placement of fill, groundwater must be controlled to below the proposed excavation levels.

Temporary excavation side slopes for Type 3 soils should not exceed 1H:1V. Temporary excavation side slopes for Type 4 soils should not exceed 3H:1V. There is a potential for sloughing to occur if the trench remains open for an extended period of time (i.e. 24-48 hours) or during a rainfall event. Therefore, it is recommended that excavations be supported by a trench box if they are to be open for an extended period of time or for rain events.

When excavations cannot be safely sloped to maintain stability during construction, temporary shoring suitably designed must be used. Steel sheet piles or steel “I” beam piles with timber lagging (soldier piles and lagging) should be employed for temporary excavations. It will be the Contractors responsibility to design a suitable temporary support system for the MTO review prior to installation. In addition the Contractor is to follow SP N. 902S01, regarding excavations for structures.

2.2.12 Dewatering

The soils encountered below the groundwater table and within potential excavation depths consist of sand, silt, sand and silt and silty clay. The estimated hydraulic conductivity, “k” of these materials is outlined below in Table 3.

Table 3 Estimated Hydraulic Conductivity

Materials	Hydraulic Conductivity “k” (m/s)
Sand	$10^{-3} - 10^{-5}$
Silt	$10^{-6} - 10^{-7}$
Sand and Silt	$10^{-4} - 10^{-6}$
Silty Clay	$10^{-7} - 10^{-9}$

Dewatering requirements will be governed by the water levels in the creek at the time of construction. It is the responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, groundwater levels and creek flow conditions for prior approval of the MTO. The method used should not undermine the existing road. The dewatering method is the responsibility of the Contractor and the Contractor should submit his proposal for review prior to construction.

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

2.2.13 Construction Recommendations

In order to minimize the disruption to traffic, it is recommended that the replacement of the culverts through Highway 522, be conducted in two construction stages. Each stage will consist of removing and replacing the culverts on one side of the Highway at a time as to provide a throughway lane at all times.

Although the excavations are expected to remain stable at a slope of 1H:1V above the groundwater table and 3H:1V below the groundwater table, sloughing will occur if the trench remains open for an extended period of time. Therefore it is recommended that during the removal and replacement of the culvert at Driver Creek that the excavated sidewalls immediately adjacent to the Highway or roadbed be supported by braced sheet piles, or some other form of bracing such as a soldier pile and lagging system.

3.0 CLOSURE

This report has been prepared by D. Muldowney, B.Eng., and reviewed by T. Crilly M.Sc., P.Eng. and S. Gonsalves, M.Eng., P.Eng. Designated MTO Foundation Contact. The field investigation was conducted by Craig St Amant.

We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

Yours truly,

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