Peto MacCallum Ltd.
CONSULTING ENGINEERS

**FOUNDATION INVESTIGATION AND DESIGN REPORT**
**for**
**22 CULVERT REPLACEMENTS**
**RESURFACING FROM 0.8 KM EAST OF**
**HIGHWAY 17/ REGIONAL ROAD 55 INTERCHANGE AT**
**SUDBURY WESTERLY 21.8 KM**
**GEOGRAPHIC TOWNSHIPS OF DENISON, GRAHAM AND WATERS**
**GREATER SUDBURY AREA, ONTARIO**
**G.W.P. 5146-09-00**


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PML Ref.: 12KF053
Index No.: 167FIR and 168FDR
GEOCRES No: 41I-299
April 16, 2014

# FOUNDATION INVESTIGATION REPORT
**for**
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**Peto MacCallum Ltd.**
CONSULTING ENGINEERS

**FOUNDATION INVESTIGATION REPORT**
for
22 Culvert Replacements
Resurfacing from 0.8 km East of
Highway 17/Regional Road 55 Interchange
At Sudbury Westerly 21.8 km
Greater Sudbury Area, Ontario
GWP 5146-09-00

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## 1. INTRODUCTION

This report summarizes the results of the foundation investigations carried out for the proposed culvert replacements on Highway 17 centreline as part of the detail design for the resurfacing project which extends from 0.8 km east of the Highway 17/Regional Road 55 interchange at Sudbury westerly 21.8 km. The investigation was carried out by Peto MacCallum Ltd. (PML) for AECOM Canada Ltd (AECOM) on behalf of the Ministry of Transportation of Ontario (MTO).

A total of 22 culverts are proposed to be replaced within the project limits of which 6 culvert sites are located within the Geographic Township of Denison, 11 culvert sites are located within the Geographic Township of Graham and 5 culvert sites are located within the Geographic Township of Waters. The following table lists the culverts including the Geographic Township, station, existing culvert type and location of the culvert on Highway 17 with respect to east bound lane (EBL), centreline (C/L) and west bound lane (WBL):

| PML FOUNDATION CULVERT ID | MTO CULVERT ID | GEOGRAPHIC TOWNSHIP | STATION | EXISTING CULVERT TYPE | HIGHWAY 17 SECTION |
|---|---|---|---|---|---|
| A | D4 | Denison | 13+410 | 760x38.6 CSP | EBL |
| B | D7 | | 15+506 | 760x79 CSP | C/L |
| C1 | D9 | | 16+110 | 1070x28.4 CSP | WBL |
| C2 | D9 | | 16+110 | 1070x28.4 CSP | EBL |
| D | D11 | | 16+740 | 910x162.4 CSP | C/L |
| E | D12 | | 16+958 | 760x79.7 CSP | C/L |
| F | G3 | Graham | 10+525 | 1070x21.2 CSP | EBL |
| G | G4 | | 10+685 | 910x33.9 CSP | EBL |
| H | G5 | | 10+910 | 910x36.5 CSP | EBL |

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| PML FOUNDATION CULVERT ID | MTO CULVERT ID | GEOGRAPHIC TOWNSHIP | STATION | EXISTING CULVERT TYPE | HIGHWAY 17 SECTION |
|---|---|---|---|---|---|
| I | G7 | Graham | 12+273 | 760x83.0 CSP | C/L |
| J1 | G8 | | 12+620 | 1070x33.0 CSP | WBL |
| J2 | G9 | | 12+630 | 1070x30 CPS | EBL |
| K | G10 | | 12+850 | 780x23 CSP | WBL |
| L | G25 | | 17+894 | 1830x95 CSP | C/L |
| M | G26 | | 18+882 | 1520x81 CSP | C/L |
| N1 | G30 | | 19+820 | 1220x35.6 CSP | WBL |
| N2 | G31 | | 19+850 | 1220x37 CSP | EBL |
| O | W4 | Waters | 11+753 | 910x31.7 CSP | WBL |
| P | W14 | | 13+598 | 910x26.9 CSP | EBL |
| Q | W17 | | 14+943 | 910x36.7 CSP | WBL |
| R | W25 | | 15+687 | 1220x108.2 CSP | C/L |
| S | W26 | | 16+125 | 910x48.2 CSP | EBL |

General Arrangement drawings for the culverts were not available during the preparation of this report.

## 2. SITE DESCRIPTION AND GEOLOGY

All of the culverts are located within the greater Regional Municipality of Greater Sudbury of which 6 culvert sites are located within the Geographic Township of Denison, 11 culvert sites are located within the Geographic Township of Graham and 5 culvert sites are located within the Geographic Township of Waters.

The Highway 17 corridor, within the project limits, is generally flanked by open water bodies, marsh areas and rock outcrops. There are interchanges, underpasses, overpasses and at-grade-crossings within the Highway 17 project limits. The Culvert A is situated 100 m east of Den Lou Road at-grade-crossing. Within that stretch of the Highway 17, residential homes flank the north and south of the highway. The Culvert B is approximately 545 m east of

Bay Street Overpass. Culverts N1 and N2 are approximately 500 m west of Highway 144 Interchange. Within the vicinity of the interchange, dense stands of trees and rock outcrops on undulating ground surfaces are visible. Culvert P is situated about 280 m east from Highway 24 Underpass. Business and residential areas are situated south and north of the underpass at Highway 17. Culvert Q is located about 280 m west of Highway 55 Interchange from which Culvert R is located about 500 m east. Industrial areas are established north of the interchange with residential areas at the south of the interchange.

The project site is located within the Huronian Supergroup of the Canadian Shield. The typical rock types in the project area are argillite, siltstone and greywacke of the McKim Formation. The soil/bedrock interface is encountered at variable depths.

## 3. INVESTIGATION PROCEDURE

The condition in the terms of reference required locating boreholes to minimize interruptions to traffic and requirements for traffic protection so that an acceptable alternative to advancing a borehole at the midpoint of the culvert or the centre-line of the highway would be to advance a borehole at the shoulder. The terms of reference also required a maximum spacing between boreholes of 15.0 m.

A constraint in locating the boreholes during the field investigation was that the boreholes were advanced during winter conditions to facilitate access through wet and swampy ground. The boreholes were located to minimize disruption to traffic and safety concerns while providing the information needed for the design through interpolation between boreholes.

Other limiting factors included an inability to identify certain culvert end locations (particularly at culvert K) because they were not visible. Hence, borehole locations had to be selected based on the observed water flows and to accommodate drilling in areas with topographical challenges. Also, because of the proximity of the boreholes to the highway, tree clearing had to be kept to a minimum (for safety reasons – i.e. trees falling near the highway), which further limited possible drilling access routes. In addition, at some culvert ends, soft ground and open water conditions were encountered which prevented drilling at the end of the culverts.

The boreholes that were generally located as follows:

- for EBL or WBL culverts, one at the outlet, one at the inlet and one at the shoulder for a total of 3 boreholes
- for C/L culverts, in addition to the inlet and outlet boreholes; one borehole at each EBL and WBL shoulder and one in the median of Highway 17 for a total of 5 boreholes.

Finally, auger probe holes were drilled (using hand equipment) directly adjacent to the culverts to augment the information provided in the boreholes. These auger probes did not encounter the limitations imposed by the light to heavy weight equipment and topography and were considered to supplement the information provided in the other boreholes. The report has also been supplemented with the inclusion of relevant pavement boreholes.

The following table summarizes the subsurface investigation program at each culvert location.

**SUMMARY OF INVESTIGATION PROGRAM**

| PML (MTO) FOUNDATION CULVERT ID AND STATION | INVESTIGATION PERIOD | NO. OF BOREHOLES | DEPTH (m) | NO. OF DCPT | DEPTH (m) | NO. OF AUGER PROBES |
|---|---|---|---|---|---|---|
| **GEOGRAPHIC TOWNSHIP OF DENISON** | | | | | | |
| A (D4)<br>13+410 EBL | November 8, 29 and December 3, 2012 | 3 | 6.7-14.3 | 3 | 11.3 to 17.5 | - |
| B (D7)<br>15+506 C/L | May 1, 27, June 3 and 11, 2013 | 6 | 2.0-9.8 | 3 | 1.4 to 3.9 | - |
| C1 (D9)<br>16+110 WBL | November 9, 30 and December 14, 18, 2012 | 3 | 6.1-9.1 | 3 | 6.7 to 13.7 | 1 |
| C2 (D9)<br>16+110 EBL | November 9 and December 4, 2012 | 2 | 6.8-10.2 | 1 | 6.8 | - |
| D (D11)<br>16+740 C/L | February 26, 28, April 30, May 27 and 31, 2013 | 5 | 3.0-8.2 | 3 | 2.6 to 9.1 | 2 |

**SUMMARY OF INVESTIGATION PROGRAM**

| PML (MTO) FOUNDATION CULVERT ID AND STATION | INVESTIGATION PERIOD | NO. OF BOREHOLES | DEPTH (m) | NO. OF DCPT | DEPTH (m) | NO. OF AUGER PROBES |
|---|---|---|---|---|---|---|
| E (D12) 16+958 C/L | March 4, 25, May 2, 28 and June 3, 2013 | 5 | 6.7-15.7 | 3 | 9.1 to 10.7 | 3 |
| **GEOGRAPHIC TOWNSHIP OF GRAHAM** | | | | | | |
| F (G3) 10+525 EBL | November 27, December 3 and 11, 2012 | 3 | 3.1-9.0 | 1 | 3.1 | 3 |
| G (G4) 10+685 EBL | November 22 and 23, December 3 to 5 and December 19, 2012 | 3 | 0.6-10.1 | - | - | 2 |
| H (G5) 10+910 EBL | November 26 and December 5 to 7, 2012 | 3 | 2.2-7.0 | 2 | 2.4 | 2 |
| I (G7) 12+273 C/L | November 19 to 21, 2012<br>May 3, 6, 28 and June 5 and 6, 2013 | 5 | 2.1-17.2 | 2 | 2.2 and 8.4 | 1 |
| J1 (G8) 12+620 WBL | November 15, 19 and 20, 2012 | 3 | 7.8-8.7 | 2 | 8.2 and 11.3 | 2 |
| J2 (G9) 12+630 EBL | November 20, 29 and<br>December 5, 2012 | 2 | 6.7-7.9 | 1 | 7.8 | 8 |
| K (G10) 12+850 WBL | November 14, 16, 22 and 30, 2012 | 3 | 7.2-10.5 | 2 | 8.1 and 9.6 | 2 |
| L (G25) 17+894 C/L | December 13, 2012<br>April 29, May 6, 7, 29, 30 and June 5, 2013 | 6 | 1.5-18.0 | 2 | 9.1 and 10.0 | 2 |
| M (G26) 18+882 C/L | November 28 and December 7, 2012<br>May 29 and 30, June 4 and 10, 2013 | 6 | 2.6-18.7 | 1 | 5.0 | - |
| N1 (G30) 19+820 WBL | December 13 and 18, 2012<br>April 29, 2013. | 3 | 1.1-6.2 | 1 | 5.1 | 2 |
| N2 (G31) 19+850 EBL | December 6 and 18, 2012 and April 29, 2013 | 2 | 1.4-8.7 | 1 | 1.1 | 3 |

**SUMMARY OF INVESTIGATION PROGRAM**

| PML (MTO) FOUNDATION CULVERT ID AND STATION | INVESTIGATION PERIOD | NO. OF BOREHOLES | DEPTH (m) | NO. OF DCPT | DEPTH (m) | NO. OF AUGER PROBES |
|---|---|---|---|---|---|---|
| **GEOGRAPHIC TOWNSHIP OF WATERS** | | | | | | |
| O (W4)<br>11+753 WBL | December 7 and 11, 2012 March 5 and October 31, 2013 | 3 | 1.5-8.5 | 1 | 6.0 | 3 |
| P (W14)<br>13+598 EBL | December 12 and 13, 2012<br>March 6 and June 12, 2013 | 3 | 7.7-8.5 | 2 | 8.5 and 9.1 | 4 |
| Q (W17)<br>14+943 WBL | April 11, 25 and June 19, 2013 | 3 | 0.8-14.6 | 1 | 8.4 | 2 |
| R (W25)<br>15+687 C/L | April 10, 11, 24, May 8 and 9 2013 | 5 | 1.2-15.8 | 2 | 10.0 and 15.0 | - |
| S (W26)<br>16+125 EBL | April 23, June 13 and 14, 2013 | 4 | 9.7-14.3 | 2 | 10.4 and 10.7 | - |

The positions of the boreholes relative to the culverts were selected by PML allowing for appropriate drilling equipment accessibility and buried utilities. Some of the boreholes were positioned more than 15.0 m away from the culvert locations which are summarized in the following table.

| PML (MTO) FOUNDATION CULVERT ID | GEOGRAPHIC TOWNSHIP | BOREHOLE ID | OFFSET | REASON |
|---|---|---|---|---|
| B (D7)<br>15+506 C/L | Denison | B-5 | 20.1 m south | Wet soft ground. Unable to mobilize any drill equipment at the end of the culvert. |
| I (G7)<br>12+273 C/L | Graham | I-1 | 22.7 m west | Wet soft ground. Unable to mobilize any drill equipment at the end of the culvert. |
| J1 (G8)<br>12+620 WBL | Graham | J1-3 | 21.6 m west | Pooled water at the end of the culvert. Unable to mobilize any drill equipment at the end of the culvert. |
| K (G10)<br>12+850 WBL | Graham | K-1 | 18.3 | Unable to locate the culvert at the site. The end boreholes were drilled near where water flow was visible. |
| | | K-3 | 37.9 | |

The boreholes were advanced using various methods including continuous flight solid and hollow stem augers, washboring techniques with track-mounted CME-850, sonic drilling, tripod mounted cathead set up or power auger, supplied and operated by specialist drilling contractors, working under the full-time supervision of a PML field supervisor.

It is considered that the boreholes generally represent the subsoil conditions at the culverts. The presence of undetected obstructions and large boulders must be considered for this project within the existing rockfill embankments.

The ground surface elevations at the borehole locations, except for the three boreholes, Q-1 to Q-3, investigated for culvert Q located at Station 14+943 WBL in the Geographic Township of Waters, were established by exp Geomatics. The elevations of the boreholes Q-1 to Q-3 were approximated from the profile drawing 14+953.474.dwg by exp Geomatics received via email dated February 26, 2014.

Soil samples were recovered from the boreholes at regular 0.75 and 1.5 m intervals of depth using the standard penetration test method. Standard penetration tests were conducted to assess the strength characteristics of the substrata. Soils were identified in accordance with the MTO soil classification manual procedures.

The groundwater conditions in the boreholes were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, where encountered, by measuring the groundwater level in the open holes. No groundwater observations could be made in the borehole completed using the tripod , where casing was advanced and washboring techniques as the drilling method continuously introduced outside water into the borehole.

The boreholes were backfilled with a bentonite/cement mixture where required in accordance with the MTO guidelines and MOE Reg. 903 for borehole abandonment.

The recovered soil samples were returned to our laboratory in Toronto for detailed visual examination, laboratory testing and classification. A summary of the laboratory testing program at each culvert location is presented in the table below:

**SUMMARY OF LABORATORY PROGRAM**

| PML (MTO) FOUNDATION CULVERT ID | TOWNSHIP | NATURAL MOISTURE CONTENT DETERMINATIONS | ATTERBERG PLASTICITY LIMITS | GRAIN SIZE DISTRIBUTION |
|---|---|---|---|---|
| A (D4) | Denison | 34 | - | 8 |
| B (D7) | | 12 | 5 | 5 |
| C1 (D9) | | 15 | 4 | 2 |
| C2 (D9) | | 22 | 2 | 5 |
| D (D11) | | 16 | 3 | 6 |
| E (D12) | | 36 | 7 | 14 |
| F (G3) | Graham | 4 | - | 1 |
| G (G4) | | 5 | - | 2 |
| H (G5) | | 10 | 1 | 3 |
| I (G7) | | 27 | 8 | 8 |
| J1 (G8) | | 14 | 2 | 2 |
| J2 (G9) | | 14 | 1 | 4 |
| K (G10) | | 22 | 5 | 6 |
| L (G25) | | 22 | 5 | 9 |
| M (G26) | | 21 | 6 | 8 |
| N1 (G30) | | 7 | 2 | 3 |
| N2 (G31) | | 5 | - | 2 |
| O (W4) | Waters | 10 | - | 2 |
| P (W14) | | 20 | 3 | 6 |
| Q (W17) | | 12 | 3 | 4 |
| R (W25) | | 45 | 5 | 11 |
| S (W26) | | 47 | 2 | 14 |

All elevations in this report are expressed in metres.

## 4. SUMMARIZED SUBSURFACE CONDITIONS - GEOGRAPHIC TOWNSHIP OF DENISON

In summarizing the subsurface at each culvert location, reference is made to the appended Record of Borehole Sheets for details of the subsurface conditions including soil classifications, probable bedrock depth, inferred stratigraphy, standard penetration test results, dynamic cone penetration test results, pocket penetrometer test results, in-situ vane test results, and groundwater observations during and upon completion of augering. The results of laboratory Atterberg plasticity limits, grain size distributions and moisture content determinations are also shown on the Record of Borehole Sheets. The laboratory Atterberg plasticity limits results and grain size distribution results are also presented in plasticity charts and grain size distribution charts.

### 4.1 Culvert A (D4) – Station 13+410 EBL

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing A-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out during the period of November 8, 29 and December 3, 2012. A total of three boreholes (A-1 to A-3) were drilled to 6.7 to 14.3 m. Three dynamic cone penetration tests (DCPTs) were conducted directly adjacent to the location of borehole A-1 to 11.3 m and from the auger termination depth of 13.7 and 14.3 m in boreholes A-2 and A-3 to 14.3 and 17.5 m, respectively. Dynamic cone penetration test data was compiled on the appended Record of Borehole sheets.

The laboratory grain size distribution charts are presented in Figures A-GS-1 to A-GS-5. All of the test results are summarized on the Record of Borehole sheets.

In summary, the subsurface stratigraphy revealed in the boreholes generally comprised a topsoil or fill underlain by cohesionless deposits of sandy silt to silty sand, locally interrupted by a layer of silt, over sand/gravelly sand. Organic inclusions were noted to 3.0 and 8.7 m in boreholes A-1 and A-2, respectively. Probable bedrock was inferred by dynamic cone penetration test refusal at 11.3 to 17.5 m.

#### 4.1.1 Topsoil

A 200 mm thick topsoil layer was encountered surficially in borehole A-3 extending to elevation 250.7.

#### 4.1.2 Pavement Structure

A 140 mm of asphaltic concrete pavement with approximately 560 mm of sand and gravel base course was encountered in borehole A-2 that extended to 0.7 m (elevation 258.3).

#### 4.1.3 Fill

A 1.4 to 2.0 m thick fill unit was encountered surficially in boreholes A-1, beneath the pavement structure in borehole A-2 at 0.7 m (elevation 258.3) and beneath the surficial topsoil at 0.2 m (elevation 250.7) in borehole A-3, that extended to 1.4 to 2.2 m (elevation 248.7 to 256.8). The material comprised sand in borehole A-1 and sand to silty sand in borehole A-2. In borehole A-3, the fill consisted of silty clay with sand seams.

Organic material and wood debris were noted within the fill in boreholes A-1 and A-3. N values varied from the WH (weight of the hammer and rods) to 29 indicating very loose to compact and very soft to firm compactness/consistency. Moisture content determinations of 3 and 17% were noted in the cohesionless sandy soils in boreholes A-1 and A-2 and moisture contents of 25 to 31% were noted in the cohesive silty clay in borehole A-3.

#### 4.1.4 Sandy Silt to Silty Sand

A 2.1 to 8.3 m thick deposit of cohesionless sandy silt to silty sand was encountered below the fill layer in boreholes A-1 to A-3 at 1.4 to 2.2 m (elevation 248.7 to 256.8). This deposit extended to 4.3 to 10.5 m (elevation 240.4 to 254.7). A lower deposit of silt and sand was also encountered beneath the silt in borehole A-2 at 8.7 m (elevation 250.3), extended to the 13.7 m sampling depth (elevation 245.3) and likely to the probable bedrock encountered at 14.3 m (elevation 244.7) in the dynamic cone penetration test.

N values ranged from 3 to 23 within the upper deposit and typically from 6 to 12 in the lower deposit, indicating very loose to compact relative density. An N value of 1 was noted in the lower deposit in borehole A-2, but is suspected to be due to hydraulic disturbance.

The results of grain size distribution analyses for 6 sandy silt to silty sand samples are included in Figures A-GS-1 to A-GS-3. The moisture content determinations ranged from 7 to 26%.

#### 4.1.5 Silt

A cohesionless silt deposit was encountered below the silty sand layer in borehole A-2 at 4.3 m (elevation 254.7). The deposit was 4.4 m thick and extended to the silt and sand layer at 8.7 m (elevation 250.3). Organic inclusions were noted throughout the entire deposit. The material was loose to compact and moist to wet, with N values ranging 8 to 28 and moisture contents of 10 to 25%.

The result of a grain size distribution analysis for a sample of silt is included in Figures A-GS-4.

#### 4.1.6 Sand

A cohesionless sand deposit with varying silt and gravel content was encountered below the silty sand to sandy silt at 5.0 and 10.5 m (elevation 251.5 and 240.4) in boreholes A-1 and A-3, respectively. The unit extended to the sampling termination depth of 6.7 and 14.3 m (elevation 249.8 and 236.6) in boreholes A-1 and A-3, respectively. It is likely that the sand deposit is 6.3 to 7.0 m thick and extended to the probable bedrock at the 11.3 and 17.5 m (elevation 245.2 and 233.4, respectively) dynamic cone penetration test refusal depth. The stratum was found to have a typically loose to compact relative density with N values of 6 to 24.

The result of grain size distribution analysis for a sand sample is included in Figure A-GS-5. The moisture content determinations ranged from 19 to 34%.

#### 4.1.7 Probable Bedrock

The probable bedrock was inferred by dynamic cone penetration test refusal at 11.3 to 17.5 m (elevation 233.4 to 245.2).

#### 4.1.8 Groundwater

At the time of the investigation, there was no surface water noted at either culvert end. During augering, groundwater was observed at 3.7 and 7.0 m (elevation 252.8 and 252.0) in boreholes A-1 and A-2, respectively. Upon completion of drilling, groundwater was measured at 6.1 and 13.1 m (elevation 250.4 and 246.9) in boreholes A-1 and A-2 respectively. No groundwater observations could be made in borehole A-3 as outside water was continuously introduced into the borehole as a result of the washboring techniques. The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 4.2 Culvert B (D7) – Station 13+506 C/L

The borehole and pavement hole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing B-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out in December 19, 2012, May 1, 27 and June 3, 11, 2013. A total of six boreholes (B-1 to B-5 and B2-A) were drilled to 2.0 to 9.8 m. A total of three dynamic cone penetration tests (DCPTs) were conducted at 1.0 m west of borehole B-1, 2.0 m west of borehole B-3 and 1.0 m west of borehole B-5. The DCPTs were advanced 1.4 to 3.9 m. The DCPT data was compiled in Record of Borehole Sheets B-1, B-3 and B-5.

A total of eight pavement holes, of which four of them were on EBL and four of there on WBL, were conducted at the culvert location. The subsurface encountered in the EBL pavement holes included 150 to 390 mm thick asphaltic concrete (except in pavement hole 15+520 6.0 Lt C/L) over 140 to 970 mm granular material. A 0.3 m of sand was encountered below the granular material in pavement hole 15+520 3.5 Rt C/L. All the pavement holes met refusal on rockfill at 0.50 to 0.97 m depth. The subsurface encountered in the WBL pavement holes included 120 to 380 mm thick asphaltic concrete over 360 to 680 mm granular material. All the pavement holes met refusal on rockfill at 0.78 to 0.81 m depth.

The laboratory grain size distribution charts are presented in Figures B-GS-1 to B-GS-4 and Atterberg limits results are presented in Figures B-PC-1 to B-PC-4.

In summary, surficial 200 and 300 mm thick peat was encountered in borehole B-1 and B-5. Pavement structure was encountered in boreholes B-2, B-2A, B-4 underlain by fill materials. Borehole B-2 was terminated in fill unit at 4.9 m. Surficial fill was encountered in borehole B-3 which extended to termination depth of 5.8 m on probable bedrock. In boreholes B-1 and B-2A, 1.0 to 1.2 m thick organic deposits were encountered below peat and fill. Cohesionless silt and sandy silt were encountered below the organic soils in boreholes B-1 and B-2A, below fill in borehole B-4 and below clayey silt in borehole B-5 extending to borehole termination depths of 2.0 to 9.8 m where further auger refusal was met on probable bedrock.

#### 4.2.1 Peat

A 200 and 300 mm thick surficial peat layer was encountered in boreholes B-1 and B-5 extending to elevation 260.0 and 259.1, respectively.

#### 4.2.2 Pavement Structure

A 100 mm thick asphaltic concrete pavement over 400 mm thick sand and gravel base course was encountered in boreholes B-2, B-2A and B-4. The sand and gravel pavement fill extended to 0.5 m (elevation 264.6 to 267.9).

### 4.2.3 Fill

Variable fill materials were encountered below pavement structure in boreholes B-2, B-2A and B-4 at 0.5 m (elevation 264.6 to 267.9) and surficially (elevation 263.7) in borehole B-3. Borehole B-2 was terminated in the rockfill at 4.9 m (elevation 263.2) where auger refusal was met. A 5.8 m thick rockfill encountered below the pavement structure was penetrated through to 6.3 m (elevation 262.1) in borehole B-2A. About 5.8 m thick fill layer, which included compact to loose sand and firm to hard silty clay/clayey silt, was encountered in borehole B-3 extending to auger refusal on probable bedrock at 5.8 m (elevation 257.9). In borehole B-4, a 1.5 m thick cohesive very stiff clayey silt fill layer was encountered below a 3.5 m thick rockfill extending to 5.5 m (elevation 259.6).

N values ranged from 5 to 32 in borehole B-3 and 23 and 38 in borehole B-4. One N value of 8 was obtained in borehole B-2A. In situ vane testing conducted in borehole B-3 within the cohesive deposit indicated a shear strength value of 24 kPa, with a sensitivity of 1.

The results of grain size distribution analysis for clayey silt fill sample are included in Figure B-GS-1. The plasticity charts are presented in Figure B-PC-1. The liquid and plastic limits of clayey silt fill samples were 28 and 18, respectively with plasticity index value of 10. The moisture content determinations ranged from 20 and 22%.

### 4.2.4 Organic Soils

A 1.0 m thick very loose to compact organic silt was encountered below peat at 0.2 m (elevation 260.0) in borehole B-1 and extended to 1.2 m (elevation 259.0). Two N values recorded were 1 and 14. Moisture content determinations were 42 and 51%.

Below rockfill in borehole B-2A, a 1.0 m thick very stiff organic silty clay over 1.2 m thick very stiff organic clayey silt deposits were encountered extending to 8.5 m (elevation 259.9). Two N values recorded were 16 and 21.

A grain size distribution chart of an organic clayey silt sample is presented in Figure B-GS-2 and the corresponding Atterberg limits are presented in Figure B-PC-2. The liquid and plastic limits are 33 and 25, with a plasticity index of 8. Two moisture content determinations were 18 and 38%.

#### 4.2.5 Clayey Silt

A 1.5 and 1.1 m thick very soft to very stiff cohesive clayey silt deposit was encountered below fill in borehole B-4 at 5.5 m (elevation 259.6) and below peat at 0.3 m (elevation 259.1) in borehole B-5, respectively. The stratum in borehole B-4 extended to 7.0 m (elevation 258.1) where refusal was met on probable bedrock and in borehole B-5 the deposit extended to underlying cohesionless silt at 1.4 m (elevation 258.0). N values recorded were 24 in borehole B-4 and WH (weight of hammer and rods) in borehole B-5.

The grain size distribution of a clayey silt sample is presented in Figure B-GS-3 and the corresponding Atterberg limits are presented in Figure B-PC-3. The liquid and plastic limits are 27 and 18, with plasticity index of 9. Two moisture content determinations were 14 and 47%.

#### 4.2.6 Sandy Silt

A local compact sandy silt deposit was encountered in borehole B-2A at 8.5 m (elevation 259.9), which extended to termination depth 9.8 m (elevation 258.6) where auger refusal was met on probable bedrock.

#### 4.2.7 Silt

Compact to dense silt was encountered in borehole B-1 below organic silt and in B-5 underlying clayey silt at 1.2 and 1.4 m (elevation 259.0 and 258.0), respectively and extended to termination depth of 2.0 and 3.1 m (elevation 258.2 and 256.3) where auger refusal was met on probable bedrock. Three N values recorded were 10, 11 and 36.

The grain size distribution analyses results of selected samples are presented in Figure B-GS-4. The Atterberg limits are presented in Figure B-PC-4. The liquid and plastic limits of a silt sample were 26 and 22, respectively, with a plasticity index of 4 indicating low plasticity. The second silt sample tested was non-plastic. Two moisture content determinations were 26 and 29%.

#### 4.2.8 Probable Bedrock

The probable bedrock was inferred by auger refusal or DCPTs in boreholes all boreholes, except in borehole B-2, at 2.0 to 9.8 m (elevation 256.2 to 258.6).

#### 4.2.9 Groundwater

Open water was encountered in boreholes B-1 and B-5 during and upon completion of augering at ground surface (elevation 259.4 and 260.2). The remaining boreholes were charged with water during drilling. The groundwater level is subjected to seasonal fluctuations and rainfall patterns.

### 4.3 Culvert C1 (D9)– Station 16+110 WBL

The borehole and pavement hole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing C1-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out in November 9 and 30, December 14 and 18, 2012. A total of three boreholes (C1-1 to C1-3) were drilled to 6.1 to 9.1 m. A total of three dynamic cone penetration tests (DCPTs) were conducted at 1.0 m east of borehole C1-1 to 6.7 m, in borehole C1-2 from 6.1 to 13.7 m and 5.0 m west of borehole C1-3 to 7.2 m. The DCPTs were advanced to termination depths of 6.7 to 13.7 m. The DCPT data was compiled in Record of Borehole Sheets C1-1 to C1-3. An auger probe was conducted at the north end of the culvert near borehole C1-1 that penetrated through 0.3 m thick sand and 0.9 m thick silty clay and met refusal on probable rockfill at 1.2 m. The data was summarized in the Record of Borehole C1-1 sheet.

One pavement hole was conducted at the culvert location and the subsurface encountered included 430 mm thick asphaltic concrete over 270 mm thick granular material. The pavement hole met refusal on rockfill at 0.70 m.

The laboratory grain size distribution charts are presented in Figures C1-GS-1 and C1-GS-2 and Atterberg Limits results are presented in Figure C1-PC-1. All of the test results are summarized on the Record of Borehole sheets.

In summary, 200 and 300 mm thick surficial topsoil and peat was encountered in boreholes C1-1 and C1-3, respectively. Surficial fill encountered in borehole C1-2 extended to 3.2 m. A 0.5 and 1.1 m thick silty sand was encountered below the fill and peat in boreholes C1-2 and C1-3 at 3.2 and 0.3 m, respectively, which extended to underlying silty clay at 3.7 and 1.4 m. Very soft to very stiff silty clay was encountered in all three boreholes at 0.2 to 3.7 m and extended to 2.2 to 6.1 m. Borehole C1-2 was terminated in the silty clay layer at 6.1 m. Based on DCPT, probable silty clay in borehole C1-2 extended to 7.0 m under which probable silt/sandy silt/sand soils were encountered and extended to 13.7 m, mantling probable bedrock. Generally, very loose to compact cohesionless soils were encountered below the silty clay at 2.2 and 3.4 to borehole termination depths of 6.2 and 9.1 in boreholes C1-1 and C1-3. The DCPT conducted near borehole C1-1 was advanced through probable sandy silt to the termination depth of 6.7 m where further penetration refusal was met on probable bedrock. The auger probe near borehole C1-1 was advanced through 0.3 m thick sand over 0.9 m thick silty clay to a termination depth of 1.2 m where refusal was not met.

#### 4.3.1 Topsoil / Peat

A 200 mm thick surficial topsoil was encountered in borehole C1-1, which extended to elevation 271.7. In borehole C1-2, a 300 mm thick peat unit was encountered extending to elevation 272.2.

#### 4.3.2 Fill

A 3.2 m thick fill unit was encountered in borehole C1-2. The fill unit has two distinct layers, which includes a 0.6 m thick sand and gravel fill extending to elevation 275.8, overlying a 2.6 m thick rockfill to a depth of 3.2 m (elevation 273.2).

#### 4.3.3 Silty Sand

A 0.5 and 1.1 m thick cohesionless very loose to compact silty sand was encountered in boreholes C1-2 and C1-3 at 3.2 m (elevation 273.2) below fill and at 0.3 m (elevation 272.2) below peat, respectively, and extended to 3.7 and 1.4 m (elevation 272.7 and 271.1). Two N values recorded were WH (weight of hammer and rods) and 20. One moisture content determination was 23%.

#### 4.3.4 Silty Clay

Cohesive silty clay was encountered below the topsoil at 0.2 m (elevation 271.7) in borehole C1-1, below silty sand at 3.7 m (elevation 272.7) in borehole C1-2 and at 1.4 m (elevation 271.1) in borehole C1-3. The silty clay extended to 2.2 m (elevation 269.7) and 3.4 m (elevation 269.1) in borehole C1-1 and C1-3, respectively. Borehole C1-2 was terminated in the silty clay layer at 6.1 m (elevation 270.3). Based on DCPT in borehole C1-2, the silty clay probably extended to 7.0 m. N values for the silty clay ranged from WH to 15. One penetrometer test value obtained was 38 kPa. In-situ vane test conducted within the cohesive deposit obtained a shear strength value of 52 kPa, with a sensitivity of 4. The silty clay consistency ranged from very soft to very stiff.

Grain size distribution analyses results are presented in Figure C1-GS-1 and the corresponding Atterberg limits results are presented in Figure C1-PC-1. The liquid limits were 36 and 37 and the plastic limits were 19 and 21, with plasticity indices of 15 and 18. Moisture content determinations ranged from 32 to 45%.

### 4.3.5 Silt / Sandy Silt

A discontinuous 2.3 and 3.6 m thick silt layer was encountered in boreholes C1-1 and C1-3 at 2.2 and 3.4 m (elevation 269.7 and 269.1) and extended to 4.5 and 7.0 m (elevation 267.4 and 265.5). N values recorded were from 4 to 12. The silt was compact to loose in borehole C1-1 and loose to very loose in borehole C1-3. Grain size distribution analyses results are presented in Figure C1-GS-2. Moisture content ranged from 19 to 27%.

Underlying the silt in borehole C1-1, a 1.7 m thick loose sandy silt was encountered at 4.5 m (elevation 267.4), which extended to the termination depth of 6.2 m (elevation 265.7). Two N values recorded were 5 and 10 blows for 15 cm penetration where refusal on probable bedrock was met. The DCPT conducted near borehole C1-1 extended through probable sandy silt from 6.2 m (elevation 265.7) to 6.7 m (elevation 265.2), where refusal was met on probable bedrock.

Based on the DCPT carried in borehole C1-2, probable loose to compact silt / sandy silt was encountered in borehole C1-2 at approximately 7.0 m (elevation 269.4) and extended to 12.0 m (elevation 264.4),

### 4.3.6 Sand

A 2.1 m thick compact sand was encountered in borehole C1-3 at 7.0 m (elevation 265.5), which extended to the termination depth of 9.1 m (elevation 263.4). Two N values recorded were 28 and 50 blows for 0 cm penetration indicating refusal on probable bedrock. One moisture content determination was 19%.

Based on DCPT in borehole C1-2, probable sand was encountered at approximately 12.0 m (elevation 264.4) and extended to the refusal depth at 13.7 m (elevation 262.7) on probable bedrock.

#### 4.3.7 Probable Bedrock

Probable bedrock was encountered in boreholes C1-1 to C1-3 and the DCPTs at 6.2 to 13.7 m (elevation 262.7 to 265.7).

#### 4.3.8 Groundwater

During augering, surficial water (elevation 271.9) was observed in borehole C1-1 and at 0.3 m (elevation 272.2) in borehole C1-3. Upon completion of augering, water was observed at ground surface (elevation 272.5) in borehole C1-3. Groundwater was not observed in borehole C1-2 before or upon completion of augering.

### 4.4 Culvert C2 (D9) – Station 16+110 EBL

Data from borehole C1-3 (common borehole for culverts C1 and C2) was used to assess the subsurface ground condition at the C2 culvert location.

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing C2-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out during the period of November 9 to December 4, 2012. A total of three boreholes (C2-1, C2-2 and C1-3) were drilled to 6.7 to 10.2 m. Two dynamic cone penetration tests (DCPTs) were conducted at 5.0 m west of borehole C1-3 and 1.0 m south of borehole C2-2. The DCPTs were advanced to termination depths of 6.8 and 7.2 m. The DCPT data was compiled in Record of Borehole Sheets C1-3 and C2-2.

One pavement hole was conducted at the culvert location and the subsurface encountered included 210 mm thick asphaltic concrete over 290 mm thick granular material which in turn was underlain by 210 mm thick sand. The pavement hole met refusal on rockfill at 0.71 m.

The laboratory grain size distribution charts are presented in Figures C2-GS-1 to C2-GS-3 and Atterberg Limits results are presented in Figures C2-PC-1 and C2-PC-2. All of the test results are summarized on the Record of Borehole sheets.

In summary, a 300 mm thick peat was encountered surficially in boreholes C1-3 and C2-2. In borehole C2-1, a 1.2 m thick pavement structure over a 1.8 m thick rockfill was encountered. A 1.1 m thick silty sand was locally encountered below the peat in borehole C1-3. Cohesive very soft to very stiff 1.3 to 2.3 m thick silty clay/clayey silt was encountered below peat in borehole C2-2 and below rockfill in borehole C2-1. Below the cohesive layers, very loose to compact 2.1 to 4.4 m thick silt and sand deposits were encountered. The DCPT conducted near borehole C2-2 extended through probable silt to the refusal depth of 6.8 m. Probable bedrock was inferred by auger or dynamic cone refusal in boreholes C2-2 and C1-3 at 6.8 and 9.1 m.

### 4.4.1 Peat

A 300 mm thick peat layer was encountered surficially in boreholes C2-2 and C1-3 extending to 0.3 m (elevation 272.1 and 272.2).

### 4.4.2 Pavement Structure

A 250 mm asphaltic concrete pavement over sand and gravel base course extending to 1.2 m (elevation 275.3) was encountered in boreholes C2-1. One N value of 32 was recorded in the sand and gravel base course. The moisture content determination on a sand and gravel fill sample was 5%.

### 4.4.3 Fill

A 1.8 m thick rockfill unit was encountered in borehole C2-1 below pavement structure at 1.2 m (elevation 275.3) and extended to 3.0 m (elevation 273.5).

#### 4.4.4 Silty Sand

A cohesionless silty sand deposit was locally encountered below the peat at 0.3 m (elevation 272.2) in borehole C1-3. The deposit was 1.1 m thick extending to silty clay at 1.4 m (elevation 271.1). N values were WH (weight of hammer and rods) and 20 indicating very loose to compact. The moisture content determination was 23%.

#### 4.4.5 Clayey Silt

A 2.3 m thick cohesive clayey silt was encountered in borehole C2-1 at 3.0 m (elevation 273.5) and extended to 5.3 m (elevation 271.2). N values ranged from 10 to 18 decreasing with depth indicating very stiff to stiff consistency.

The results of grain size distribution analyses for clayey silt samples are included in Figure C2-GS-1. A plasticity chart is presented in Figure C2-PC-1. The liquid and plastic limits of clayey silt sample were 22 and 16, respectively with the corresponding plasticity index value of 6. The moisture content determinations on cohesive clayey silt samples ranged from 15 to 33%.

#### 4.4.6 Silty Clay

A cohesive deposit of silty clay was encountered below the fill at 5.3 m (elevation 271.2) in borehole C2-1 and below the peat at 0.3 m (elevation 272.1) in borehole C2-2 and below the cohesionless silty sand deposit at 1.4 m (elevation 271.1) in borehole C1-3. The stratum was 0.7 to 2.2 m thick extending to 2.5 to 6.0 m (elevation 269.1 to 270.5).

N values ranged from WH (weight of hammer and rods) to 15. In situ vane testing conducted in boreholes within the cohesive deposit indicated shear strength value of 52 kPa, with sensitivity value of 4. Penetrometer test results of 100 and 150 kPa were recorded. The stratum was considered to be typically firm to stiff consistency with local very soft layers.

It is inferred that the existing culvert was placed on firm to stiff cohesive silty clay.

The results of grain size distribution analysis for silty clay sample are included in Figure C2-GS-2. The plasticity charts are presented in Figure C2-PC-2. The liquid and plastic limits of silty clay sample were 36 and 21, respectively with the corresponding plasticity index value of 15. The moisture content determinations ranged from 31 to 45%.

#### 4.4.7 Silt

A cohesionless silt deposit was encountered below the silty clay at 2.5 to 6.0 m (elevation 269.1 to 270.5) in all of the boreholes. The unit was 3.6 to 4.2 m thick, probably 4.3 m thick in borehole C2-2 extending to borehole termination depth of 10.2 m (elevation 266.3) in borehole C2-1 and to probable bedrock at 6.8 m (elevation 265.6) in borehole C2-2 and to underlying sand at 7.0 m (elevation 265.5) in borehole C1-3. N values ranged from 4 to 13. The stratum was found to have a typically loose to compact.

The results of grain size distribution analysis for silt samples are included in Figure C2-GS-3. The moisture content determinations ranged from 22 to 31% indicating wet conditions.

#### 4.4.8 Sand

A cohesionless compact sand deposit was encountered below the silt at 7.0 m (elevation 265.5) in borehole C1-3. The unit was 2.1 m thick extending to underlying probable bedrock at 9.1 m (elevation 263.4). Two N values recorded were 28 and 50 blows for 0 cm penetration, where refusal on probable bedrock was met. The moisture content determination was 19% indicating wet conditions.

#### 4.4.9 Probable Bedrock

The probable bedrock was inferred by auger and dynamic cone refusal in boreholes C1-3 and C2-2 at 9.1 and 6.8 m (elevation 263.4 and 265.6). The remaining borehole C2-1 was terminated at 10.2 m (elevation 266.3).

#### 4.4.10 Groundwater

At the time of the investigation, there was no surface water noted either culvert end.

Groundwater was encountered in boreholes C2-2 and C1-3 at 2.5 and 0.3 m (elevation 269.9 and 272.2), respectively. Upon completion of drilling, groundwater was measured at ground surface (elevation 272.5) in borehole C1-3. The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 4.5 Culvert D (D11) – Station 16+740 C/L

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawings D-1 and D-2. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on February 26, 28, April 30, May 27 and 31, 2013. A total of five boreholes (D-1 to D-5) were drilled to 3.0 to 8.2 m. A total of three dynamic cone penetration tests (DCPT) were conducted at 2.0 m north of borehole D-1, 2.0 m west of borehole D-3 and 2.0 m north of borehole D-5. The DCPTs were advanced to termination depths of 2.6 to 9.1 m. The DCPT data was compiled in Record of Borehole sheets D-1, D-3 and D-5. In addition, two auger probes were carried out at the north end of the culvert to assess the subsoil condition and data was summarized in Record of Borehole D-1 sheet. One auger probe met refusal on probable rockfill at 1.1 m and the other auger probe penetrated through silty sand containing organics to 1.5 m depth.

A total of three pavement holes were conducted, two at EBL and one at WBL, at the culvert location. The subsurface encountered in the EBL pavement holes included 355 and 390 mm thick asphaltic concrete over 325 and 210 mm thick granular material which was underlain by 220 mm thick sand and gravel/gravelly sand layer, respectively. The pavement holes met refusal on rockfill at 0.82 and 0.90 m. The subsurface encountered in the WBL pavement hole included 360 mm thick asphaltic concrete over 160 mm thick granular material which was underlain by 220 mm thick sand. Refusal was met on probable rockfill at 0.74 m.

The laboratory grain size distribution charts are presented in Figures D-GS-1 and D-GS-4 and Atterberg Limits results are presented in Figure D-PC-1. All of the test results are summarized on the Record of Borehole sheets.

The subsurface stratigraphy revealed in the boreholes generally comprised a 0.2 and 0.3 m thick peat/topsoil or surficial pavement structure over 3.8 and 4.0 m thick fill underlain by organic soils and/or cohesive soils over cohesionless sandy silt/silt. Auger or dynamic cone refusal on probable bedrock was inferred by in boreholes D-1 to D-4 at 3.1 to 8.0 m (elevation 258.8 to 261.8). In borehole D-5, the bedrock is at a greater depth than 9.1 m (<elevation 248.9).

#### 4.5.1 Topsoil

A 200 mm thick surficial topsoil layer was encountered in borehole D-3 and extended to elevation 263.1.

#### 4.5.2 Peat

A 300 mm thick peat was encountered below surficial 0.3 m snow and ice cover in borehole D-5 at 0.3 m (elevation 257.7 and extended to 0.6 m (elevation 257.4) in borehole D-5, respectively.

#### 4.5.3 Pavement Structure

A 200 mm thick asphaltic concrete pavement over sand and gravel base course was encountered in boreholes D-2 and D-4 and extended to 0.5 m (elevation 268.6 and 267.1).

#### 4.5.4 Fill

A 3.8 and 4.0 m thick rockfill unit was encountered in boreholes D-2 and D-4 drilled on the existing embankment shoulders at 0.5 m (elevation 268.6 and 267.1) extending to 4.3 and 4.5 m (elevation 264.8 and 263.1).

#### 4.5.5 Organic Soils

A 500 mm thick organic clayey silt was locally encountered in borehole D-5 below the peat at 0.6 m (elevation 257.4). The deposit extended to 1.1 m (elevation 256.9). A composite N value of 7 was obtained indicating firm consistency. The moisture content determination was about 22%.

#### 4.5.6 Sand

A 500 mm thick of sand containing organics was locally encountered below surficial ice and snow cover in borehole D-1 at 0.6 m (elevation 261.3). The deposit extended to 1.1 m (elevation 260.8). One N value of 11 was obtained indicating compact state of compactness. The moisture content determination was 24%.

#### 4.5.7 Silty Clay

A cohesive deposit of silty clay containing organics was encountered below the topsoil and fill at 0.2 m (elevation 263.1) and 4.5 m (elevation 263.1) in boreholes D-3 and D-4, respectively and below the organic clayey silt at 1.1 m (elevation 256.9) in borehole D-5. The stratum was 1.3 to 3.3 m thick extending to probable bedrock in boreholes D-3 and D-4 at 3.5 and 5.8 m (elevation 259.8 and 261.8) and to underlying cohesionless soils at 8.2 m (elevation 249.8) in borehole D-5. N values ranged from 2 to 12. Penetrometer test results ranged from 25 to 138 kPa. In situ testing conducted within the cohesive deposit indicated shear strength values of 48 and greater than 100 kPa with sensitivity of 6. The silty clay layer was found to be very stiff to stiff with local firm layers.

The results of grain size distribution analysis for silty clay samples are included in Figure D-GS-1. The plasticity charts are presented in Figure D-PC-1. The liquid and plastic limits of silty clay samples ranged from 36 to 45 and 19 to 22, respectively with plasticity index values of 17 to 23. The moisture content determinations ranged from 27 to 43%.

#### 4.5.8 Silt / Sandy Silt/Sand and Silt

A cohesionless silt/sandy silt/sand and silt deposit was encountered in boreholes D-1, D-2 and D-5 at 1.1 to 4.3 m (elevation 254.2 to 264.8). The unit was 1.9 to 4.4 m thick, probably 5.3 m thick in borehole D-5. The deposit extended to probable bedrock at 3.1 and 8.0 m (elevation 258.8 to 261.1) and to borehole termination depth of 8.2 m (elevation 249.8), probably dynamic cone termination depth of 9.1 m (elevation 248.9).

N values ranged from 7 to 55 and 34 for 23 cm sampler penetration in borehole D-2. The stratum was found to have a loose to very dense state of compactness.

The results of grain size distribution analysis for silt/sandy silt/sand and silt samples are included in Figures D-GS-2 to D-GS-4. The moisture content determinations ranged from 9 to 32%.

#### 4.5.9 Probable Bedrock

The probable bedrock was inferred by auger refusal or dynamic cone in boreholes D-1 to D-4 at 3.1 to 8.0 m (elevation 258.8 to 261.8). Bedrock was not contacted in borehole D-5 at the termination depth 9.1 m (elevation 248.9)

#### 4.5.10 Groundwater

Groundwater was encountered in boreholes D-1 and D-5. During augering, groundwater was observed at 0.6 and 0.3 m (elevation 261.3 and 257.7) in boreholes D-1 and D-5, respectively. Upon completion of drilling, groundwater was measured at 0.6 m (elevation 261.3) in borehole D-1. The remaining boreholes were charged with water during drilling. The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 4.6 Culvert E (D12) – Station 16+958 C/L

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing E-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out during the period of March 4, 25, May 2, 28 and June 3, 2013. A total of five boreholes (E-1 to E-5) were drilled to 6.7 to 15.7 m. A total of three dynamic cone penetration tests (DCPT) were also conducted at 2.0 m west of borehole E-1, 2.0 m south of borehole E-3 and 3.0 m east of borehole E-5. The DCPTs were advanced to termination depths of 9.1 to 10.7 m. The DCPT data was compiled in Record of Borehole sheets E-1, E-3 and E-5. In addition, a total of three auger probes were advanced to assess the subsoil condition at the culvert ends and data was summarized in the Record of Borehole E-1 and E-5 sheets.. Two auger probes were carried out near borehole E-1 of which one met refusal on frozen ground and the other auger probe penetrated through silty clay to 1.5 m depth. One auger probe was conducted near borehole E-5 that penetrated through to 0.5 m where refusal was met on probable rockfill.

Two pavement holes were conducted, each at EBL and WBL, at the culvert location. The subsurface encountered in the EBL pavement hole included 340 mm thick asphaltic concrete over 260 mm thick granular material which was underlain by 700 mm thick sand. The pavement hole met refusal on rockfill at 1.3 m. The subsurface encountered in the WBL pavement hole included 330 mm thick asphaltic concrete over 480 mm thick granular material. Refusal was met on probable rockfill at 0.81 m.

The laboratory grain size distribution charts are presented in Figures E-GS-1 to E-GS-5 and Atterberg Limits results are presented in Figures E-PC-1 and E-PC-2.

In summary, a 300 mm thick peat was encountered in borehole E-1 and a 300 mm thick organic silty clay was encountered in borehole E-5 below ice and snow cover at 0.3 and 0.6 m. Surficial 200 mm thick topsoil was encountered in borehole E-2. Pavement structure was encountered in boreholes E-2 and E-4 to 0.5 m. A 0.6 to 4.2 m thick fill was encountered in boreholes E1 to E-4. Cohesive very soft to very stiff clayey soils were encountered below the fill which in turn was overlying typically compact to dense cohesionless soils.

#### 4.6.1 Peat and Topsoil

A 300 mm thick peat was encountered below the ice and snow cover in borehole E-1 at 0.3 m (elevation 256.9) and extended to 0.6 m (elevation 256.6).

A 200 mm thick surficial topsoil layer was encountered in borehole E-3 extending to elevation 258.2.

#### 4.6.2 Pavement Structure

A 100 mm thick asphaltic concrete pavement over approximately 400 mm thick sand and gravel base course was encountered in boreholes E-2 and E-4 to 0.5 m (elevation 263.3 and 262.8), respectively.

#### 4.6.3 Fill

A 0.6 and 1.3 m thick silty clay fill was encountered in borehole E-1 below the peat layer and in boreholes E-3 below the surficial topsoil at 0.6 m (elevation 256.6) and 0.2 m (elevation 258.2), respectively and extended to 1.2 m (elevation 256.0) and 1.5 m (elevation 256.9). Below the pavement structure, a 3.8 m thick rockfill over silty clay fill was encountered at 0.5 m (elevation 263.3) extending to 4.7 m in borehole E-2 and a 3.8 m thick rockfill was encountered in borehole E-4 below the pavement structure at 0.5 m (elevation 262.8) which extended to 4.3 m (elevation 259.0). N values recorded in the silty clay fill were from 1 to 5. Moisture content determinations were between 30 and 38%.

### 4.6.4 Organic Silty Clay

A 0.3 m organic silty clay was encountered in borehole E-5 below snow and ice cover at 0.6 m (elevation 256.4) and extended to 0.9 m (elevation 256.1).

### 4.6.5 Clayey Silt

A 2.0 and 1.3 m thick very soft to firm clayey silt deposits were encountered in borehole E-1 at 1.2 m (elevation 256.0) below fill and in borehole E-4 at 10.4 m (elevation 252.9) below silty clay and extended to 3.2 m and 11.7 m (elevation 254.0 and 251.6), respectively. N values recorded were WH (weight of hammer and rods), 3 and 4.

A grain size distribution chart is presented in Figure E-GS-1 and the corresponding Atterberg limits are presented in Figure E-PC-1. The liquid and plastic limits were 34 and 21 with a plasticity index of 13. Moisture content determinations were between 30 and 36%.

### 4.6.6 Silty Clay

A 2.2 to 6.6 m thick cohesive soft to very stiff silty clay was encountered in boreholes E-2, E-3 and E-4 below fill and below topsoil in borehole E-5 at 0.9 to 4.7 m (elevation 256.1 to 259.1) and extended to 3.7 to 10.4 m (elevation 249.5 to 255.1). N values ranged from WH to 26. In-situ vane test conducted within the cohesive soil obtained 22 to more than 100 kPa, with sensitivity values of 5 to 24. Penetrometer test values ranged between 25 and 87 kPa.

The results of grain size distribution analysis for silty clay samples are included in Figure E-GS-2. The Atterberg limits are presented in Figure E-PC-2. The liquid and plastic limits of silty clay samples ranged from 43 to 48 and 18 to 24, respectively with plasticity index values of 21 to 26. The moisture content determinations ranged from 20 to 50%.

#### 4.6.7 Silt

A 0.7 to greater than 4.0 m thick loose to dense silt was encountered in all boreholes at 3.2 to 11.7 m (elevation 249.5 to 255.1) and extended to 5.8 to 15.7 m (elevation 247.6 to 252.6). N values recorded typically ranged between 5 and 32. A low N value of 2 recorded in borehole E-2 was possibly due to hydraulic disturbance during sampling.

The results of grain size distribution analyses are presented is Figure E-GS-3. Moisture content determinations ranged between 16 and 34%.

#### 4.6.8 Silt and Sand

Compact silt and sand was encountered below silt in borehole E-1 at 5.8 m (elevation 251.4) and extended to the termination depth of 8.2 m (elevation 249.0). Two N values recorded were 16 and 18. The result of a grain size distribution analysis of a selected sample is presented in Figure E-GS-4. Two moisture content determinations were 19 and 24%.

#### 4.6.9 Sand and Silt

A cohesionless compact sand and silt was encountered in borehole E-3 at 5.8 m (elevation 252.6) and extended to the termination depth of the borehole at 6.7 m (elevation 251.7). One N value recorded was 12. The result of a grain size distribution analysis is presented in Figure E-GS-5. One moisture content determination was 18%.

#### 4.6.10 Sand

A compact sand was encountered below sandy silt in borehole E-2 at 11.6 m (elevation 252.2) extending to the termination depth of 14.3 m (elevation 249.5). Two N values recorded were 16 and 25 blows for 15 cm penetration. Two moisture content determinations were 22 and 24%.

#### 4.6.11 Groundwater

Water was observed below ice and snow cover in borehole E-1 at 0.3 m (elevation 256.9) and in borehole E-5 at 0.6 m (elevation 256.4) during and after completion of augering. Groundwater was observed in boreholes E-2 and E-3 at 8.7 m (elevation 255.1) and 3.7 m (elevation 254.7), respectively, during augering only. Borehole E-4 was charged with water during drilling. The groundwater level is subject to seasonal fluctuations and rainfall patterns.

## 5. SUMMARIZED SUBSURFACE CONDITIONS - GEOGRAPHIC TOWNSHIP OF GRAHAM

In summarizing the subsurface at each culvert location, reference is made to the appended Record of Borehole Sheets for details of the subsurface conditions including soil classifications, probable bedrock depth, inferred stratigraphy, standard penetration test results, dynamic cone penetration test results, pocket penetrometer test results, in-situ vane test results, and groundwater observations during and upon completion of augering. The results of laboratory Atterberg plasticity limits, grain size distributions and moisture content determinations are also shown on the Record of Borehole Sheets. The laboratory Atterberg plasticity limits results and grain size distribution results are also presented in plasticity charts and grain size distribution charts.

### 5.1 Culvert F (G3) – Station 10+525 EBL

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing F-1. The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on November 27, December 3 and 11, 2012. A total of three boreholes (F-1, F-2 and F-3) were drilled to 3.1 to 9.0 m. A dynamic cone penetration test was carried out 0.5 m east of borehole F-3 to 3.1 m. An auger probe was advanced at the culvert end near borehole F-1 which encountered possible fill consisting of sand and gravel with cobbles and boulders to the auger refusal depth of 0.5 m on probable boulder. In addition, two auger probes were advanced near borehole F-3, at the end of the culvert. Both probes encountered refusal at 0.8 m, likely on boulders. The data was summarized in the Record of Borehole F-1 and F-3 sheets.

One pavement hole was conducted at the culvert location and the subsurface encountered included 450 mm thick asphaltic concrete over 430 mm thick granular material. The pavement hole met refusal on rockfill at 0.73 m.

The laboratory grain size distribution chart is presented in Figure F-GS-1.

The subsurface stratigraphy revealed in the boreholes generally comprised of fill underlain by cohesionless deposits overlying bedrock. Cobbles and boulders were encountered within the fill materials. Probable bedrock was inferred by refusal at 3.1 to 9.0 m.

#### 5.1.1 Pavement Structure

A pavement structure was contacted at the ground surface in borehole F-2 consisting 450 mm of asphaltic concrete pavement over sand and gravel base course which extended to 0.9 m (elevation 251.0).

#### 5.1.2 Topsoil

A 200 mm thick topsoil was encountered surficially in borehole F-3 extending to 0.2 m, elevation 248.2.

#### 5.1.3 Fill

Below the pavement structure in borehole F-2 and topsoil in borehole F-3 and from the ground surface in borehole F-1, a 2.7 to 4.4 m thick fill stratum was encountered and penetrated at 2.7 to 5.3 m (elevation 244.9 to 246.6). The fill consisted of sand and gravel with cobbles and boulders in borehole F-1 and F-2, and sand with cobbles in borehole F-3. Peat layers were contacted within the fill in borehole F-3.

N values ranging from 3 to 20 were recorded within borehole F-3. Borehole F-3 encountered refusal at 3.1 m (elevation 245.3) on probable bedrock.

#### 5.1.4 Sand and Gravel

Below the fill, a sand and gravel stratum with cobbles was encountered at 2.7 m (elevation 244.9) in borehole F-1. This stratum extended to the refusal depth of 4.0 m (elevation 243.6). One N value of 36 was recorded in this stratum.

#### 5.1.5 Silt

A layer of silt was contacted below the fill at 5.3 m (elevation 246.6) in borehole F-2. The unit was 2.0 m thick extending to sand at 7.3 m (elevation 244.6). One N value of 31 was recorded indicating dense relative density.

The results of grain size distribution analysis for a representative sample recovered from this stratum are included in Figure F-GS-1.

#### 5.1.6 Sand

A sand stratum was encountered at 7.3 m (elevation 244.6) below the silt stratum in borehole F-2. This stratum extended to probable bedrock encountered at 8.7 m (elevation 243.2)

One N value of 88 was encountered within this stratum indicating very dense relative density.

#### 5.1.7 Probable Bedrock/ Bedrock

Probable bedrock was inferred by auger and/or dynamic cone refusal in boreholes F-1 and F-3 at 4.0 and 3.1 m (elevation 243.6 and 245.3), respectively. In borehole F-2, the bedrock was contacted at 8.7 m (elevation 243.2) and was drilled to auger refusal at 9.0 m (elevation 242.9).

#### 5.1.8 Groundwater

Water was observed 75 mm above the culvert invert on June 18, 2013.

Boreholes F-1 and F-2 were dry on completion of drilling. The groundwater level was not established in borehole F-3 because the borehole was charged with water during drilling. The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 5.2 Culvert G (G4) – Station 10+685 EBL

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing G-1. The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on November 22 and 23, December 3 to 5 and December 19, 2012. A total of three boreholes (G-1, G-2 and G-3) were drilled to 0.6 to 10.1 m. Two auger probes were conducted to verify the subsurface conditions. An auger probe was advanced at Station 10+700 of the East Bound Lane Median to verify subsurface conditions. The probe encountered 0.6 m of water over 0.8 m of clayey silt, with no refusal to 1.4 m depth at the east of the culvert. On the west side of the culvert similar conditions were encountered however refusal was encountered below the clayey silt stratum at 1.2 m depth on probable rockfill. It is inferred that refusal was encountered on a boulder. The data was summarized in Record of Boreholes G-1 sheet.

One pavement hole was conducted at the culvert location and the subsurface encountered included 320 mm thick asphaltic concrete over 150 mm thick granular material which in turn was underlain by 240 mm thick sand. The pavement hole met refusal on rockfill at 0.71 m.

The laboratory grain size distribution chart is presented in Figures G-GS-1 and G-GS-2. The laboratory test results are also provided on the Record of Borehole sheets.

The subsurface stratigraphy revealed in the boreholes generally comprised of fill underlain by cohesionless deposits overlying bedrock. Cobbles and boulders were encountered within the fill materials. Probable bedrock was inferred by refusal at 7.3 m in Borehole G-1. Refusal on probable boulders was encountered at 0.6 m in Borehole G-3.

#### 5.2.1 Pavement Structure

A pavement structure was contacted at ground surface in borehole G-2 consisting of 320 mm asphaltic concrete pavement over sand and gravel base course which extended to 0.9 m (elevation 253.7).

#### 5.2.2 Fill

Below the pavement structure in borehole G-2 and from the ground surface in borehole G-1 and G-3, fill was contacted to 0.6 to 7.2 m (elevation 245.5 to 247.9). The fill consisted of sand and gravel with cobbles and boulder in borehole G-1 that extended to 2.8 m (elevation 245.5). Cobbles and boulders were encountered below about 0.9 m (elevation 253.7) in borehole G-2. In borehole G-3 the fill consisted of boulders with sand and gravel to the power auger refusal depth of 0.6 m (elevation 247.9). It is inferred that refusal was encountered on a boulder in borehole G-3.

#### 5.2.3 Organic Silt

Below the fill, a 0.3 m thick layer of organic silt was contacted from 2.8 m (elevation 245.5) to 3.1 m (elevation 245.2) in borehole G-1.

#### 5.2.4 Silt

A silt stratum was contacted below the organic silt at 3.1 m (elevation 245.2) in borehole G-1. The unit was 2.7 m thick extending to sand at 5.8 m (elevation 241.0). N values of 8 and 17 were recorded in the silt indicating loose to compact relative density.

The results of grain size distribution analysis for a representative sample recovered from this stratum are included in Figure G-GS-1.

#### 5.2.5 Sand

A 1.5 m thick sand stratum was encountered at 5.8 m (elevation 242.5) below the silt stratum in borehole G-1. This stratum extended to probable bedrock encountered at 7.3 m (elevation 241.0).

One N value of 23 was encountered within this stratum indicating compact relative density.

#### 5.2.6 Sand and Silt

Below the fill in borehole G-2, a 2.9 m thick sand and silt stratum was contacted to the borehole termination depth of 10.1 m (elevation 244.5). N values of 23 and 12 were recorded in this stratum indicating a compact relative density. Borehole G-2 was terminated at 10.1 m (elevation 244.5) within this stratum.

The results of grain size distribution analysis for a representative sample recovered from this stratum are included in Figure G-GS-2.

#### 5.2.7 Probable Bedrock

Probable bedrock was inferred by refusal at 7.3 m (elevation 241.0) in borehole G-1.

#### 5.2.8 Groundwater

Water was noted at 0.1 m below culvert obvert on June 18, 2013. The boreholes were dry on completion of drilling. The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 5.3 Culvert H (G5) – Station 10+910 EBL

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing H-1. The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on November 26 and December 5 to 7, 2012. A total of three boreholes (H-1, H-2 and H-3) were drilled to 2.4 to 7.0 m. A total of two dynamic cone penetrations (DCPT) were conducted near boreholes H-1 and H-3 to 2.4 m. In addition, three auger probes were advanced with a power auger near boreholes H-1 and H-3. A hand auger probe advanced near Borehole H-1 encountered 0.4 m of water, underlain by clayey silt with organics to 1.1 m. No refusal was encountered in this probe. A hand auger probe advanced at the east side of culvert near Borehole H-3 encountered 0.4 m of water, underlain by clayey silt to 1.1 m, with no refusal. A second auger probe advanced on the west side of the culvert near Borehole H-3 encountered refusal on probable boulders at 0.8 m. The data was summarized in the Record of Borehole H-1 and H-3 sheets.

One pavement hole was conducted at the culvert location and the subsurface encountered included 370 mm thick asphaltic concrete over 280 mm thick granular material. The pavement hole met refusal on boulder at 0.65 m.

The laboratory grain size distribution chart is presented in Figures H-GS-1 to H-GS-3. The plasticity chart is presented on Figure H-PC-1.

The subsurface stratigraphy revealed in the boreholes generally comprised of fill underlain by native silty clay, clayey silt, silty sand and sandy silt overlying probable bedrock. Cobbles were encountered locally within the fill materials. Auger refusal on boulders/probable bedrock as encountered at 2.4 to 7.0 m in all the boreholes.

### 5.3.1 Pavement Structure

A pavement structure was contacted at ground surface in borehole H-2 consisting of 150 mm asphaltic concrete pavement over sand and gravel base course which extended to 0.9 m (elevation 260.1).

### 5.3.2 Topsoil

From the ground surface, 200 mm of topsoil was encountered in borehole H-1 and 100 mm of topsoil was encountered in borehole H-3 to elevation 257.1 and 255.9, respectively.

### 5.3.3 Fill

Below the topsoil in borehole H-1 and H-3 and below the pavement structure in borehole H-2, fill was contacted to 0.7 to 5.8 m (elevation 255.0 to 256.6). The fill consisted of silty sand in borehole H-1, silty clay with cobbles in borehole H-2 and clayey silt in borehole H-3. N values in the fill ranged from 4 to 20. Moisture content determinations ranged from 9 to 34%.

### 5.3.4 Silty Clay

Below the fill, a 1.5 m thick layer of silty clay was contacted from 0.7 m (elevation 256.6) to 2.2 m (elevation 255.1) in borehole H-1. Two N values in the silty clay were 22 and 25 indicating a very stiff consistency. Borehole H-1 encountered refusal on a boulder or probable bedrock at 2.2 m (elevation 255.1) within this stratum. The DCPT advanced near borehole H-1 penetrated through probable silty clay at 2.2 m (elevation 255.1) and extended to 2.4 m (elevation 254.9), where further cone penetration refusal was met on probable boulder.

The result of a grain size distribution analysis for a representative sample recovered from this stratum is presented in Figure H-GS-2. Figure H-PC-2 shows the results of the Atterberg Limits tests conducted on the same sample. The liquid and plastic limits were 35 and 20 with a plasticity index of 15. Two moisture content determinations were 25 and 26%.

#### 5.3.5 Clayey Silt

Below the fill in borehole H-3, a 1.2 m thick clayey silt stratum was encountered from 1.0 m (elevation 255.0) to 2.2 m (elevation 253.8). One N value of 11 was encountered within this stratum indicating a stiff consistency. One moisture content determination was 19%.

#### 5.3.6 Silty Sand

Underlying the fill in borehole H-2, a 1.2 m thick silty sand stratum was encountered at 5.8 m (elevation 255.2). This stratum extended to probable bedrock encountered at 7.0 m (elevation 254.0). One N value of 37 was encountered within this stratum indicating dense relative density.

A grain size analysis was conducted on a representative sample from this stratum. The results of grain size distribution analysis are included in Figure H-GS-3. One moisture content determination was 12%.

#### 5.3.7 Sandy Silt

Below the clayey silt in borehole H-3, a sandy silt stratum was contacted at 2.2 m (elevation 253.8) and met auger refusal on probable bedrock at 2.9 m (elevation 253.1). One N value of 21 was recorded in this stratum indicating a compact relative density.

#### 5.3.8 Probable bedrock

Probable bedrock was encountered at 7.0 m (elevation 254.0) and 2.9 m (elevation 253.1) in boreholes H-2 and H-3, respectively.

#### 5.3.9 Groundwater

Borehole H-1 was dry on completion of drilling. Groundwater was observed at 5.8 m (elevation 255.2) in borehole H-2 on completion of drilling. In borehole H-3, groundwater was established because the borehole was charged with water during drilling. The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 5.4 Culvert I (G7) – Station 12+273 C/L

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing I-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out during the period of November 19 to 21, 2012 and May 3, 6 and 28 and June 5 and 6, 2013. A total of five boreholes (I-1 to I-5) were drilled to 2.1 to 17.2 m. A total of two dynamic cone penetration tests (DCPTs) were conducted at 2.0 m west of borehole I-1 and 2.0 m west of borehole I-5. The DCPTs were advanced to 2.2 and 8.4 m. The data was compiled in Record of Borehole sheets I-1 and I-5. In addition, an auger probe was carried out at the south end of the culvert near borehole I-5 which penetrated through silty clay to 0.7 m where refusal was met on probable rockfill. The data was summarized in Record of Borehole I-5 sheet.

Two pavement holes were conducted, each at EBL and WBL, at the culvert location. The subsurface encountered in the EBL pavement hole included 100 mm thick asphaltic concrete over 500 mm thick granular material. The pavement hole met refusal on rockfill at 0.6 m. The subsurface encountered in the WBL pavement hole included 350 mm thick asphaltic concrete over 200 mm thick granular material which was underlain by 130 mm thick sand. Refusal was met on probable rockfill at 0.68 m.

The laboratory grain size distribution charts are presented in Figures I-GS-1 and I-GS-2 and Atterberg Limits results are presented in Figures I-PC-1 and I-PC-2. All of the test results are summarized on the Record of Borehole sheets.

The subsurface stratigraphy revealed in the boreholes generally comprised a 0.2 and 0.3 m thick topsoil or 1.2 to 10.6 m thick fill underlain by cohesive clay/silty clay soils over cohesionless silty sand/sand. Probable bedrock was inferred by auger or dynamic cone refusal in all boreholes at 2.2 to 17.2 m (elevation 247.6 to 254.6).

### 5.4.1 Topsoil

A 200 and 300 mm thick surficial topsoil layer was encountered surficially in boreholes I-1 and I-5 extending to 0.2 and 0.3 m (elevation 255.8 and 256.5).

### 5.4.2 Pavement Structure

A 100 mm thick asphaltic concrete pavement over sand gravel base course was encountered in boreholes I-2 and I-4 extending to 0.5 m (elevation 264.3 and 263.4).

### 5.4.3 Fill

A 1.2 to 10.1 m thick fill unit was encountered in boreholes I-1 to I-4. A 10.1 and 8.0 m thick rockfill was encountered in boreholes I-2 and I-4 below pavement structure at 0.5 m (elevation 264.3 and 263.4) and extended to 10.6 and 8.5 m (elevation 254.2 and 255.4), respectively. In borehole I-3, about 0.6 m surficial sand and gravel fill was contacted overlying a 3.1 m thick rockfill which in turn was underlain by a 4.8 m thick silty clay fill containing wood chips and organics extending to 8.5 m (elevation 254.9). A 1.2 m thick silty clay was encountered below topsoil in borehole I-1 extending to 1.4 m (elevation 254.6). N values recorded in silty clay fill layer ranged from 4 to 8, indicating firm consistency.

Grain size distribution analysis result of a fill sample is presented in Figure I-GS-1 and the corresponding Atterberg limits are presented in Figure I-PC-1. The liquid limit was 42 and the plastic limit was 22, with plasticity index value of 20. The moisture content determinations ranged from 28 to 32%.

### 5.4.4 Clay

A 2.3 and 3.1 m thick very stiff to stiff clay was contacted below fill in boreholes I-1 and I-3 at 1.4 and 8.5 m (elevation 254.6 and 254.9) and extended to 3.7 and 11.6 m (elevation 252.3 and 251.8), respectively. N values recorded ranged between 4 and 13. Penetrometer test values obtained ranged from 25 to 138 kPa. In-situ vane test conducted within the cohesive deposit obtained a shear strength value of more than 100 kPa.

Grain size distribution analyses results are presented in Figure I-GS-2 and the corresponding Atterberg limits are presented in Figure I-PC-2. The liquid limits were 53 and 59 and the plastic limits were 22 and 24, with plasticity index values of 31 and 35. Moisture content determinations were between 29 and 48%.

### 5.4.5 Silty Clay

Cohesive firm to very stiff silty clay was encountered below clay in boreholes I-1 and I-3 and below fill in boreholes I-2, I-4 and I-5 at 0.3 to 11.6 m (elevation 251.8 and 256.5) and extended to 2.1 to 16.5 m (elevation 248.1 to 254.7). In boreholes I-1, I-4 and I-5, silty clay was mantling probable bedrock at 7.9, 13.7 and 2.1 m (elevation 248.1, 250.2 and 254.7), respectively. N values recorded were between WH (weight of hammer and rods) to 24 with a high N value of 6 blows for 15 cm penetration where refusal was met on probable bedrock. Penetrometer test values obtained 12 to 163 kPa. In-situ vane test conducted within the cohesive deposit obtained shear strength values between 38 and 96 kPa with sensitivity values of 2 to 10. One shear strength value of more than 100 kPa in borehole I-4 was obtained.

Grain size distribution analyses results of selected silty clay samples are presented in Figure I-GS-3 and the corresponding Atterberg limits are presented in Figure I-PC-3. The liquid limits obtained were between 39 and 49 and the plastic limits were between 20 and 23. The plasticity index values were between 17 and 24. The moisture content determinations were from 28 and 48%.

#### 5.4.6 Silty Sand/Sand

A cohesionless silty sand/sand deposit was locally encountered in boreholes I-2 and I-3 at 16.5 and 14.9 m (elevation 248.3 and 248.4). The unit was 0.7 and 0.5 m thick extending to probable bedrock at 17.2 and 15.4 m (elevation 247.6 and 248.0) in borehole I-2 and I-3, respectively.

Two N values recorded were 22 in borehole I-2 and 10 blows for 8 cm penetration in borehole I-3. The stratum was found to have a compact state of compactness. One moisture content determination was about 13%.

#### 5.4.7 Probable Bedrock

The probable bedrock was inferred by auger refusal or dynamic cone in all of the boreholes at 2.2 to 17.2 m (elevation 247.6 to 254.6).

#### 5.4.8 Groundwater

At the time of the investigation, surface water was observed at culvert obvert level at the south end of EBL. During and upon completion of augering, groundwater was observed at 6.1 m (elevation 249.9) in borehole I-1. No water was observed in borehole I-5. The remaining boreholes were charged with water during drilling. The groundwater level is subject to seasonal fluctuations and rainfall patterns.

## 5.5 Culvert J1 (G8) – Station 12+620 WBL

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing J1-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out during the period of November 15 to 20, 2012. A total of three boreholes, J1-1 to J1-3, were drilled to 7.8 to 8.7 m. A total of two dynamic cone penetration tests (DCPT) were conducted at one 2.0 m east of borehole J1-1 and the second one 3.0 m west of borehole J1-3 to 8.2 and 11.3 m, respectively, and the data was compiled in Record of Borehole J1-1 and J1-3 sheets. Borehole J1-1 met auger refusal at 3.4 m and was re-drilled 3.0 m west of the original location. Two auger probes were advanced east and west at the south end of J1 culvert to 1.4 m where refusal was not met. The data was summarized in the Record of Borehole J1-3 sheet.

One pavement hole was conducted at the culvert location and the subsurface encountered included 180 mm thick asphaltic concrete over 110 mm thick granular material. The pavement hole met refusal on rockfill at 0.29 m.

The laboratory grain size distribution charts are presented in Figures J1-GS-1 and J1-GS-2 and Atterberg Limits results are presented in Figures J1-PC-1 and J1-PC-2.

The subsurface stratigraphy revealed in the boreholes J1-1 and J1-3 generally comprised a 3.5 and 4.1 m thick peat overlying 4.3 and 4.2 m thick cohesive clayey stratum. A local 0.4 m thick silt deposit was encountered in between the peat and clay layer in borehole J1-3. The clayey stratum in borehole J1-1 probably extended to 8.2 m where further DCPT advancement refusal was met on probable bedrock at 8.2 m. In borehole J1-3, probable 2.6 m thick sandy silt was underlying the clay layer based on DCPT which mantled the probable bedrock at 11.3 m. In borehole J1-2, a 0.8 m thick pavement structure overlay a 6.5 m thick rockfill unit. The rockfill unit included sand and gravel with cobbles and boulders. Below the rockfill, a cohesive 1.2 m thick cohesive silty clay layer was encountered which mantles the bedrock at 8.5 m.

#### 5.5.1 Topsoil / Peat

Dark brown fine fibrous peat including sand and gravel (in borehole J1-1) and shells (in borehole J1-3) was encountered in boreholes J1-1 and J1-3 at ground surface (elevations 251.3 and 250.7), respectively. The peat layer extended to 3.5 and 4.1 m (elevations 247.8 and 246.6), respectively, in boreholes J1-1 and J1-3. N values recorded were WH (weight of hammer and rods) and 1 with one value of 10 blows for 15 cm penetration was recorded (in borehole J1-1). Moisture content determinations ranged between 146 to 503%.

#### 5.5.2 Pavement Structure / Fill

A pavement structure including a 150 mm thick asphaltic concrete pavement over sand and gravel base course was encountered surficially, elevation 254.8, in borehole J1-2 and extended to 0.8 m (elevation 254.0).

A 6.5 m thick rockfill was encountered below the pavement structure in borehole J1-2 at 0.8 m (elevation 254.0) and extended to 7.3 m (elevation 247.5). This unit included sand and gravel with cobbles and boulders. One N value of 31 was recorded.

#### 5.5.3 Silt

A 0.4 m thick very loose cohesionless silt layer was encountered below the peat layer in borehole J1-3 at 4.1 m (elevation 246.6) which extended to 4.5 m (elevation 246.2). One N value of 3 was recorded.

#### 5.5.4 Silty Clay

A cohesive 4.3 and 1.2 m thick silty clay deposit was encountered below peat in borehole J1-1 at 3.5 m (elevation 247.8) and below the rockfill in borehole J1-2 at 7.3 m (elevation 247.5) and extended to 7.8 m (elevation 243.5) and 8.5 m (elevation 246.3), respectively. Borehole J1-1 was terminated at 7.8 m (elevation 243.5), where auger refusal was met on probable bedrock. The DCPT near borehole J1-1 was extended through probable silty clay to 8.2 m (elevation 243.1) where refusal was met on probable bedrock. N values recorded in borehole J1-1 were WH (weight of hammer and rods) and 5 and in borehole J1-2 one N value of 17 was recorded. In situ vane testing conducted in borehole J1-1 within the cohesive deposit indicated shear strength values of 48 and 44 kPa, with a sensitivity value of 5. Penetrometer test results of 12 to 87 kPa were recorded. The stratum was considered to be firm to very stiff consistency.

A grain size distribution analysis result for a silty clay sample is presented in Figure J1-GS-1 with a corresponding plasticity chart for the sample is presented in Figure J1-PC-1. The liquid and plastic limits of the sample are 39 and 22, with a plasticity index of 17. Three moisture content determinations were 35, 45 and 49%.

#### 5.5.5 Clay

A cohesive 4.2 m thick clay layer was encountered below the silt layer in borehole J1-3 at 4.5 m (elevation 246.2) and extended to 8.7 m (elevation 242.0). N values ranged from WH (weight of hammer and rods) to 4. In suit vane testing conducted within the cohesive deposit indicated shear strength values of 44 and 24 kPa, with sensitivity of 4. The consistency of the clay layer is considered firm to soft.

A grain size distribution analysis result for a clay sample is presented in Figure J1-GS-2 with a corresponding plasticity chart for the sample is presented in Figure J1-PC-2. The liquid and plastic limits of the sample are 52 and 22, with a plasticity index of 30. Two moisture content determinations were 32 and 55%.

#### 5.5.6 Probable Sandy Silt

Based on DCPT, it is inferred that a 2.6 m thick cohesionless compact probable sandy silt deposit was encountered below the clay layer in borehole J1-3 at 8.7 m (elevation 242.0) and extended to 11.3 m (elevation 239.4) where refusal was met on probable bedrock.

#### 5.5.7 Probable Bedrock

The probable bedrock was inferred by DCPT refusal near boreholes J1-1 and J1-3 at 8.2 m (elevation 243.1) and 11.3 m (elevation 239.4), respectively. Borehole J1-2 met refusal at 8.5 m (elevation 246.3) on probable bedrock.

#### 5.5.8 Groundwater

Open water observed at the north end of the culvert was measured at about elevation 250.0.

Groundwater was encountered in boreholes J1-1 and J1-3. During augering, groundwater was observed at 1.5 and 1.2 m (elevations 249.8 and 249.5), respectively, in boreholes J1-1 and J1-3. Upon completion of drilling, groundwater was measured at 0.6 and 1.2 m (elevations 250.7 and 249.5), respectively, in boreholes J1-1 and J1-3. Groundwater level in borehole J1-2 was not determined because the borehole was charged with water during drilling.

The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 5.6 Culvert J2 (G9) – Station 12+630 EBL

Data from borehole J1-3 (common borehole for culverts J1 and J2) was used to assess the subsurface ground condition at the J2 culvert location.

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing J2-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on November 20, 29 and December 5, 2012. A total of three boreholes, J1-3, J2-1 and J2-2, were drilled to 6.7 to 8.7 m. A total two dynamic cone penetration tests (DCPT) were conducted near boreholes J1-3 and J2-2 to 11.3 and 7.8 m, respectively, and the data was compiled in the Record of Borehole sheets. A total of six auger probes were advanced around and at the south end of the culvert to 1.5 m where refusal was not met. In addition, two auger probes were advanced on either side of the north end of J2 culvert to 1.2 and 1.4 m, where refusal was met on probable rockfill. The data was summarized in the Record of Borehole J2-1 and J2-2 sheets.

One pavement hole was conducted at the culvert location and the subsurface encountered included 340 mm thick asphaltic concrete over 350 mm thick granular material which in turn was underlain by 1.1 mm thick sand. The pavement hole met refusal on rockfill at 1.8 m.

The laboratory grain size distribution charts are presented in Figures J2-GS-1 to J2-GS-3 and Atterberg Limits results are presented in Figure J2-PC-1. All of the test results are summarized on the Record of Borehole sheets.

Generally, the subsurface stratigraphy varied between the three boreholes. Surficial peat of 4.1 m and 2.2 m thickness was encountered in boreholes J1-3 and J2-2, respectively, overlying a cohesionless loose silt layer of 0.4 m and 2.1 m thick. In borehole J2-2, cohesionless silty sand (1.7 m thick) and sand (probably 1.8 m thick) was encountered below the silt layer, mantling the probable bedrock at 7.8 m. A 4.2 m thick cohesive soft to firm clay layer was encountered below the silt layer in borehole J1-3, which overlaid probably 2.6 m thick sandy silt (inferred from DCPT) mantling probable bedrock at 11.3 m. Borehole J2-1 comprised 6.0 m thick sand and gravel fill and rockfill from ground surface which overlaid 1.5 m thick compact sand. A 0.4 m thick gravel with rock fragments was encountered below the sand unit covering probable bedrock at 7.9 m.

### 5.6.1 Peat

Dark brown fine fibrous peat including was encountered in boreholes J1-3 and J2-2 at ground surface, elevations 250.7 and 251.6, respectively. The peat layer extended to 4.1 m and 2.2 m (elevations 246.6 and 249.4), respectively, in boreholes J1-3 and J2-2. In borehole J2-2, rockfill mixed with peat was encountered at elevation 250.1. All six auger probes were extended into 1.5 m depth, which revealed peat with boulder inclusions. N values recorded were WH (weight of hammer and rods) and 3. Moisture content determinations ranged between 95 to 503%.

### 5.6.2 Fill

Surficial 700 mm thick sand and gravel fill with was encountered in borehole J2-1 extending to elevation 253.5 overlying a 5.3 m thick rockfill that extended to 6.0 m (elevation 248.2).

### 5.6.3 Silt

A 0.4 m thick very loose cohesionless silt layer was encountered below the peat layer in borehole J1-3 at 4.1 m (elevation 246.6), which extended to 4.5 m, (elevation 246.2). In borehole J2-2, a 2.1 m thick loose silt stratum was encountered at below fill at 2.2 m (elevation 249.4) and extended to 4.3 m (elevation 247.3). N values of 3 to 8 were recorded.

A grain size distribution analysis result for a silt sample is presented in Figure J2-GS-1. Three moisture content determinations were 24, 24 and 26%.

### 5.6.4 Silty Sand

A 1.7 m thick compact silty sand deposit was encountered in borehole J2-2 below silt at 4.3 m (elevation 247.3) and extended to 6.0 m (elevation 245.6). One N value recorded was 13. A moisture content determination was 21%.

#### 5.6.5 Sand

Cohesionless 1.5 and 0.7 m thick compact sand deposits were encountered in boreholes J2-1 and J2-2 at 6.0 m (elevations 248.2 and 245.6) respectively. The stratum extended to 7.5 m (elevation 246.7) and 6.7 m (elevation 244.9) in boreholes J2-1 and J2-2, respectively. Two N values 16 and 31 were recorded.

In borehole J2-2, the borehole was terminated in the sand deposit at 6.7 m (elevation 244.9). From the DCPT advanced near borehole J2-2 the sand layer probably extended to 7.8 m (elevation 243.8), where cone penetration refusal was met on probable bedrock.

Grain size distribution analyses results for sand samples are presented in Figure J2-GS-2. Two moisture content determinations were 6 and 17%.

#### 5.6.6 Clay

A cohesive 4.2 m thick clay layer was encountered below the silt layer in borehole J1-3 at 4.5 m (elevation 246.2) and extended to 8.7 m (elevation 242.0). N values ranged from WH (weight of hammer and rods) to 4. In suit vane testing conducted within the cohesive deposit indicated shear strengths of 44 and 24 kPa, with sensitivity of 4. The consistency of the clay layer is considered firm to soft.

A grain size distribution analysis result for a clay sample is presented in Figure J2-GS-3 with a corresponding plasticity chart for the sample is presented in Figure J2-PC-1. The liquid and plastic limits of the sample were 52 and 22, with a plasticity index of 30. Two moisture content determinations were 32 and 55%.

### 5.6.7 Probable Sandy Silt

Based on DCPT results, it is inferred that a 2.6 m thick cohesionless compact sandy silt deposit was encountered below the clay layer in borehole J1-3 at 8.7 m (elevation 242.0) and extended to 11.3 m (elevation 239.4), where cone penetration refusal was met on probable bedrock .

### 5.6.8 Gravel

A 400 mm thick gravel with rock fragments was encountered below the sand unit in borehole J2-1 at 7.5 m (elevation 246.7) and auger penetration refusal was met on probable bedrock at 7.9 m (elevation 246.3).

### 5.6.9 Probable Bedrock

The probable bedrock was inferred by DCPT refusal in boreholes J1-3 at 11.3 m (elevation 239.4) and in borehole J2-2 at 7.8 m depth (elevation 243.8). Borehole J2-1 met refusal on probable bedrock at 7.9 m (elevation 246.3).

### 5.6.10 Groundwater

Open water around the south end of the culvert was measured at around elevation 249.0.

Groundwater was encountered in borehole J1-3. During augering and upon completion of augering, groundwater was observed at 1.2 m (elevation 249.5) in borehole J1-3. Groundwater levels in boreholes J2-1 and J2-2 were not determined because the boreholes were charged with water during drilling.

The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 5.7 Culvert K (G10)– Station 12+850 WBL

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing K-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on November 14, 16, 22 and 30, 2012. A total of three boreholes, K-1 to K-3, were drilled to 7.2 to 10.5 m. A total of two dynamic cone penetration tests (DCPT) were conducted near boreholes K-1 and K-3 to 9.6 and 8.2 m, respectively, and data was compiled in Record of Borehole sheets. Two auger probes conducted near boreholes K-1 and K-3 met refusal on probable rockfill at 1.1 and 1.4 m, respectively. The data was summarized in the Record of Borehole K-1 and K-3 sheets.

One pavement hole was conducted at the culvert location and the subsurface encountered included 330 mm thick asphaltic concrete over 110 mm thick granular material which in turn was underlain by 210 mm thick sand. The pavement hole met refusal on rockfill at 0.71 m.

The laboratory grain size distribution charts are presented in Figures K-GS-1 to K-GS-3 and Atterberg Limits results are presented in Figures K-PC-1 and K-PC-2. All of the test results are summarized on the Record of Borehole sheets

The subsurface stratigraphy revealed in borehole K-1 included 1.2 m thick peat overlying a 5.1 m thick stiff to firm silty clay. In borehole K-2, a pavement structure overlaid a 2.3 m thick rockfill that in turn overlain a compact 500 mm thick sandy silt layer. Clayey deposits of soft to firm silty clay (1.5 m thick) and clay (3.5 m thick) were encountered below the sandy silt layer. In borehole K-3, surficial 300 mm thick topsoil overlaid a 1.3 m thick loose to compact silt layer, which was underlain by a 3.0 m thick firm to stiff silty clay and a firm 2.7 m thick clay stratum. Below the clayey deposits in all three boreholes, a cohesionless loose to compact sandy silt till deposit was encountered at 6.3 to 7.3 m and extended to 7.2 to 10.5 m where refusal was met on probable bedrock. Based on DCPT results near borehole K-1, probable sandy silt extended to 9.6 m where cone penetration refusal was met on probable bedrock.

#### 5.7.1 Topsoil

A 300 mm thick surficial topsoil was encountered in borehole K-3 extending to elevation 250.4. A moisture content determination was 34%.

#### 5.7.2 Peat

Surficial dark brown fibrous peat was encountered in borehole K-1 which extended to 1.2 m (elevation 250.1). Layers of grey clayey silt were encountered within the peat layer. A moisture content determination was 47%.

#### 5.7.3 Pavement Structure

A pavement structure including a 100 mm thick asphaltic concrete pavement over sand and gravel base course was encountered surficially (elevation 252.9) in borehole K-2 and extended to 1.0 m (elevation 251.9). One N value recorded was 25 blows for 3 cm penetration, indicating presence of rockfill below the sand and gravel unit.

#### 5.7.4 Fill

A 2.3 m thick rockfill was encountered below the pavement structure at 1.0 m (elevation 251.9) in borehole K-2 extending to 3.3 m (elevation 249.6). This unit included rockfill with cobbles and boulders. Two auger probes were extended to 1.1 and 1.4 m below grades where refusal was met on probable rockfill.

#### 5.7.5 Sandy Silt

A 500 mm thick compact localized sandy silt layer was encountered below the rockfill layer in borehole K-2 at 3.3 m (elevation 249.6) which extended to 3.8 m (elevation 249.1). One N value of 11 as recorded. One moisture content determination was 23%.

#### 5.7.6 Silt

A 1.3 m thick loose to compact silt layer was encountered below the topsoil at 0.3 m (elevation 250.4) in borehole K-3 and extended to 1.6 m (elevation 249.1). Inclusions of organics were encountered within the silt layer. A single N value recorded was 13. One moisture content determination was 25%.

#### 5.7.7 Silty Clay

A cohesive continuous 1.5 to 5.1 m thick silty clay layer was encountered below peat at 1.2 m (elevation 250.1) in borehole K-1, below sandy silt in borehole K-2 at 3.8 m (elevation 249.1) and below silt at 1.6 m (elevation 249.1) in borehole K-3. The layer extended to 4.6 to 6.3 m (elevation 245.0 to 247.6). N values recorded ranged from WH (weight of hammer and rods) to 9.

In situ vane testing conducted in boreholes K1 and K3 within the cohesive deposit indicated shear strength values of 40 to 64 kPa, with a sensitivity value of 3. A shear strength value of more than 100 kPa was recorded in borehole K1 at 3.5 m, elevation 247.8. Penetrometer test results of 25 and 100 kPa were recorded. The stratum was considered to be firm to stiff consistency.

Grain size distribution analyses results of selected silty clay samples are presented in Figure K-GS-1 with a corresponding plasticity chart for the sample is presented in Figure K-PC-1. The liquid limits of the samples were 37 and 40 and plastic limits of the samples were between 19 and 21, with plasticity indices between 16 and 21. Moisture content determinations ranged between 21 and 47%.

### 5.7.8 Clay

A cohesive 3.5 and 2.7 m thick clay layer was encountered below the silty clay layer in boreholes K-2 and K-3 at 5.3 and 4.6 m (elevation 247.6 and 246.1) respectively, extending to 8.8 and 7.3 m (elevation 244.1 and 243.4). N values ranged from WH (weight of hammer and rods) to 3. In suit vane testing conducted within the cohesive clay deposit indicated shear strengths of 22 to 48 kPa, with sensitivity values between 2 and 8. The consistency of the clay layer is considered soft to firm.

A grain size distribution analysis result for a clay sample is presented in Figure K-GS-2 with a corresponding plasticity chart for the sample is presented in Figure K-PC-2. The liquid and plastic limits of the sample are 57 and 23, with a plasticity index of 34. Two moisture content determinations were 40 and 64%.

### 5.7.9 Sandy Silt Till

A continuous cohesionless 0.9 to 1.7 m thick loose to compact sandy silt till layer was encountered below silty clay/ clay layers in all three boreholes at 6.3 to 8.8 m (elevation 243.4 to 245.0). The layer extended to borehole termination depths of 7.2 to 10.5 m (elevation 242.0 to 244.1). DCPT in borehole K-1 was extended to 9.6 m (elevation 241.7) in the probable sandy silt till layer and met refusal on probable bedrock. In borehole K-3, DCPT met refusal on probable boulder at 8.1 m (elevation 242.6) within the sandy silt till layer. Three N values recorded were 7, 15 and 23.

A grain size distribution result of a sandy silt till sample is presented in Figure K-GS-3. Three moisture content determinations were 18, 22 and 26%.

### 5.7.10 Probable Bedrock

The probable bedrock was inferred by DCPT refusal in borehole K-1 at 9.6 m (elevation 241.7) and by auger refusal in boreholes K-2 and K-3 at 10.5 and 8.7 m (elevation 242.4 and 242.0) respectively.

#### 5.7.11 Groundwater

Open water around the end of the culvert, north of WBL, was measured at around elevation 250.0.

During and upon completion of augering, groundwater was observed at 7.3 m (elevation 243.4) in borehole K-3. In borehole K-1, groundwater was not encountered during and upon completion of augering. Groundwater level was not established in borehole K-2 because the borehole was charged with water during augering.

The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 5.8 Culvert L (G25) – Station 17+894 C/L

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing L-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on December 13, 2012 and April 29, May 6, 7, 29, 30 and June 5, 2013. A total of six boreholes, L-1 to L-5 and L2-A, were drilled 1.5 to 18.0 m. A total of two dynamic cone penetration tests (DCPT) were conducted adjacent boreholes L-1 and L-5 to 10.0 and 9.1 m, respectively, and data was compiled in Record of Borehole sheets. Borehole L-2A was drilled to 18.0 m because refusal was met within the rockfill in borehole L-2. Two auger advances, 1.0 and 2.0 m west of borehole L-2, met refusal at 4.6 m on probable rockfill.

Two pavement holes were conducted, each at EBL and WBL, at the culvert location. The subsurface encountered in the EBL pavement hole included 320 mm thick asphaltic concrete over 160 mm thick granular material which was underlain by 385 mm thick sand. The pavement hole met refusal on boulder at 0.87 m. The subsurface encountered in the WBL pavement hole included 280 mm thick asphaltic concrete over 200 mm thick granular material which was underlain by 250 mm thick sand. Refusal was met on probable rockfill at 0.73 m.

The laboratory grain size distribution charts are presented in Figures L-GS-1 to L-GS-4 and Atterberg limits results are presented in Figures L-PC-1 and L-PC-2.

The subsurface encountered in the boreholes varied. Surficial 100 and 300 mm thick topsoil was encountered in boreholes L1 and L-5. Surficial pavement structure including 250 to 300 mm thick asphaltic concrete pavement over approximately 200 to 250 mm thick sand and gravel was encountered in boreholes L-2, L-2A and L-4. In borehole L-3, surficial 0.5 m thick sand and gravel fill was penetrated through into rockfill. Boreholes L-2 and L-3 were terminated in the rockfill at 4.9 and 1.5 m, respectively. Below the rockfill in boreholes L-2A and L-4 at 8.5 and 6.4 m cohesive firm to very stiff clayey units of 6.7 and 2.4 m thickness were encountered which overlain a 0.8 and 2.8 m thick very loose to compact silt layer, which mantled probable bedrock at 18.0 and 9.6 m . Underlying the topsoil, a 0.2 m thick loose sand was encountered at 0.3 m in borehole L-1 which overlaid a 6.1 m thick soft to stiff clayey deposit. In borehole L-5, a 6.1 m thick soft to very stiff clayey stratum was encountered below the topsoil layer at 0.1 m. The clayey deposits overlaid a 1.5 and 2.1 m thick compact to very dense silt till. The two DCPTs advanced near boreholes L-1 and L-5 extended through probable silt till layer to 10.0 and 9.1 m, where refusal was met on probable bedrock.

### 5.8.1 Topsoil

Surficial 300 and 100 mm thick topsoil was encountered in borehole L-1 and L-5 and extended to elevation 245.5 and 244.5, respectively. One N value of 5 was recorded in the topsoil unit in borehole L-1.

### 5.8.2 Pavement Structure

A surficial 250 to 300 mm thick asphaltic concrete pavement was encountered in boreholes L-2, L-2A and L-4 which overlain approximately 200 to 250 mm thick sand and gravel base course, extending to elevation 251.3 to 252.7.

### 5.8.3 Fill

Below the pavement structure, a 8.0 and 5.9 m thick rockfill layer was penetrated through in boreholes L2-A and L-4, extending to 8.5 and 6.4 m (elevation 244.7 and 245.4), respectively. In borehole L-2, the rockfill layer was encountered below the pavement structure and extended to 4.9 m (elevation 248.2). Surficial 0.5 m thick sand and gravel fill over 1.0 m thick rockfill was encountered in borehole L-3 and extended to 1.5 m (elevation 249.5). Boreholes L-2 and L-3 were terminated in the rockfill where refusal was met.

### 5.8.4 Silty Clay

A cohesive discontinuous 2.0 to 5.3 m thick soft to very stiff silty clay deposit was encountered below rockfill in borehole L-4 and below clayey silt in boreholes L-1 and L-5 at 0.3 to 6.4 m (elevation 244.3 to 245.4) and extended to 2.3 to 8.8 m (elevation 240.0 to 243.0). N values recorded were between 1 and 16. In situ vane testing conducted in borehole L-1 within the cohesive deposit indicated shear strength values of 20 to 48 kPa, with sensitivity values of 3.

Grain size distribution analyses results of selected silty clay samples are presented in Figure L-GS-1 with a corresponding plasticity chart for the samples is presented in Figure L-PC-1. The liquid limits of the samples were between 38 and 47 and plastic limits of the samples were between 18 and 23, with plasticity indices between 20 and 26. Moisture content determinations ranged between 26 and 66%.

#### 5.8.5 Clayey Silt

A 0.8 m thick soft to firm clayey silt layer was penetrated through in borehole L-1 from 5.8 m (elevation 240.0) to 6.6 m (elevation 239.2) and in borehole L-2A, a 1.9 m thick still clayey silt layer was encountered below the rockfill at 8.5 m (elevation 244.7) which extended to 10.4 m (elevation 242.8). In addition, a layer of 200 mm thick clayey silt, with organic inclusions, was encountered below the topsoil in borehole L-5 at 0.1 m (elevation 244.5) which extended to 0.3 m (elevation 244.3). Three N values recorded were 2, 3 and 6. In situ vane testing conducted within the cohesive deposit in borehole L-2A indicated a shear strength value of more than 100 kPa.

#### 5.8.6 Clay

A cohesive 4.8 and 3.9 m thick clay deposit was encountered below clayey silt in borehole L-2A at 10.4 m (elevation 242.8) and below silty clay in borehole L-5 at 2.3 m (elevation 242.3). The deposits extended to 15.2 and 6.2 m (elevation 238.0 and 238.4), respectively, in boreholes L-2A and L-5. N values recorded were between WH (weight of hammer and rods) to 14.

In situ vane testing conducted in boreholes L-2A and L-5 within the cohesive clay deposit indicated shear strength values of 18 to 68 kPa, with sensitivity values 6 to 10. A shear strength value of more than 100 kPa was recorded in borehole L-2A. The stratum was considered to be soft to stiff consistency.

Grain size distribution analyses results of selected clay samples are presented in Figure L-GS-2 with a corresponding plasticity chart for the samples is presented in Figure L-PC-2. The liquid limits of the samples were 51 and 53 and plastic limits of the samples were 21 and 23, with plasticity indices of 30. Moisture content determinations ranged between 29 and 55%.

#### 5.8.7 Silt

In borehole L-1, a 200 mm thick silt layer with organic inclusions was encountered below topsoil and extended to elevation 245.3. A 2.8 and 0.8 m thick silt deposit was encountered in boreholes L-2A and

L-4 below cohesive units at 15.2 and 8.8 m (elevation 238.0 and 243.0) and extended to termination depths of 18.0 and 9.6 m (elevation 235.2 and 242.2), respectively, where auger refusal was met on probable bedrock (borehole L-2A) / probable boulder (borehole L-4). N values recorded were WH, 12 and 16.

A grain size distribution analysis result for a silt sample is presented in Figure L-GS-3. Two moisture content determinations were 21 and 31%.

#### 5.8.8 Silt Till

Compact to very dense silt till of 1.5 and 2.1 m thickness was encountered below clayey silt and clay in boreholes L-1 and L-5 at 6.6 m (elevation 239.2) and 6.2 m (elevation 238.4), respectively. Boreholes L-1 and L-5 were terminated in the silt till layer at 8.1 m (elevation 237.7) and 8.3 m (elevation 236.3), respectively. Three N values recorded were 10 and 22 in borehole L-5 and 84 blows for 23 cm penetration in borehole L-1 where refusal was met. The DCPTs advanced near boreholes L-1 and L-5 were extended into the probable silt till deposit to 10.0 m (elevation 235.8) and 9.1 m (elevation 235.5), respectively, where refusal was met on probable bedrock.

A grain size distribution result of silt till samples is presented in Figure L-GS-4. Two moisture content determinations were 24 and 31%.

#### 5.8.9 Probable Bedrock

The probable bedrock was inferred by DCPT refusal in boreholes L-1 and L-5 at 10.0 and 9.1 m (elevation 235.8 and 235.5), respectively. In borehole L-2A probable bedrock was inferred at 18.0 m (elevation 235.2), where auger refusal was met.

#### 5.8.10 Groundwater

Groundwater was observed during drilling in boreholes L-1, L-4 and L-5 at 2.6, 8.8 and 6.2 m, (elevation 243.2, 243.0 and 238.4), respectively. In boreholes L-2, L-2A and L-3, boreholes were charged with water during augering.

The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 5.9 Culvert M (G26) – Station 18+882 C/L

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing M-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on November 28 and December 7, 2012 and May 29 and 30, June 4 and 10, 2013. A total of six boreholes, M-1 to M-5 and M-2A, were drilled 2.6 to 18.7 m. One dynamic cone penetration test (DCPT) was conducted 2.0 m south of borehole M-1 to 5.0 m and data was compiled in Record of Borehole No M-1 sheet. Three attempts were made near borehole M-2 location to penetrate through the rockfill unit to the native soils. The first two attempts met refusal on rockfill at 8.0 and 9.4 m on May 30, 2013. Borehole M-2A was the third attempt penetrating the rockfill at 11.6 m.

Two pavement holes were conducted, each at EBL and WBL, at the culvert location. The subsurface encountered in the EBL pavement hole included 760 mm thick granular material. The pavement hole met refusal on boulder at 0.76 m. The subsurface encountered in the WBL pavement hole included 320 mm thick asphaltic concrete over 160 mm thick granular material which was underlain by 385 mm thick sand. Refusal was met on probable rockfill at 0.87 m.

The laboratory grain size distribution charts are presented in Figures M-GS-1 to M-GS-4 and Atterberg Limits results are presented in Figures M-PC-1 and M-PC-2. All of the test results are summarized on the Record of Borehole sheets.

The subsurface encountered in the boreholes varied. Surficial 200 and 300 mm thick topsoil/peat was encountered in boreholes M-1 and M-5. Surficial pavement structure underlain by rockfill was encountered in boreholes M-2, M-2A and M-4. Surficial rockfill with sand and gravel was encountered in borehole M-3. Boreholes M-2 and M-3 were terminated in the rockfill deposit at 9.4 and 2.6 m, respectively. Below the rockfill in boreholes M-2A, M-4 and M-5 and below topsoil in borehole M-1, cohesive very soft to hard clayey units of 4.5 to 6.4 m thickness were encountered. Borehole M-4 was terminated in silty clay layer at 12.8 m where probable bedrock was encountered. In borehole M-1, M-2A and M5, cohesionless 0.6 to 0.9 m thick silt to sand deposits were encountered at 6.0 to 18.0 m below the cohesive soils and extended to 6.6 to 18.7 m where refusal was met on probable bedrock.

### 5.9.1 Topsoil/ Peat

Surficial 300 mm thick topsoil was encountered in borehole M-1 and extended to elevation 247.8. A 200 mm thick surficial peat was encountered in borehole M-5 which extended to elevation 246.7.

### 5.9.2 Pavement Structure

Surficial 200 mm thick asphaltic concrete pavement over approximately 300 mm thick sand and gravel base course layer was encountered in boreholes M-2, M-2A and M-4, which extended to 0.5 m (elevation 256.1 to 256.3).

### 5.9.3 Fill

Below the pavement structure, a 11.1 and 7.3 m thick rockfill layer was penetrated through in boreholes M2-A and M-4, extending to 11.6 m (elevation 245.2) and 7.8 m (elevation 248.8), respectively. In borehole M-2, the rockfill layer was encountered below the pavement structure at 0.5 m (elevation 256.2) and extended to a penetration refusal depth of 9.4 m (elevation 247.3). Surficial rockfill with sand and gravel pockets was encountered in borehole M-3 that extended to the termination depth of 2.6 m (elevation 250.1) on probable rockfill.

#### 5.9.4 Organic Clayey Silt

A 2.0 m thick organic clayey silt was encountered below peat in borehole M-5 at 0.2 m (elevation 246.7), which extended to 2.2 m, elevation 244.7. Three N values recorded were 1, 4 and 8. One moisture content determination was 43%.

#### 5.9.5 Silty Clay

A 2.9 to 6.4 m thick soft to hard silty clay was encountered below topsoil in borehole M-1 at 0.3 m (elevation 247.8), below rockfill in boreholes M-2A and M4 at 11.6 and 7.8 m (elevation 245.2 and 248.8), respectively and below organic clayey silt in borehole M-5 at 2.2 m (elevation 244.7). The silty clay layer extended to 3.2 to 18.0 m depth (elevation 238.8 to 244.9). Borehole M-4 was terminated in the silty clay deposit, where further advancement met refusal on probable boulder. N values recorded in boreholes ranged between WH (weight of hammer and rods) to 24 with a high N value of 50 blows for 10 cm penetration in borehole M4, where cobbles were encountered in the silty clay deposit. In situ vane testing conducted within the cohesive deposit in boreholes M-1, M-2A and M-5 indicated shear strength values between 20 and 96 kPa, with sensitivity values between 1 and 12.

Grain size distribution results of selected silty clay samples are presented in Figure M-GS-1 and their corresponding Atterberg limits are presented in Figure M-PC-1. The liquid limits ranged between 38 and 47 and the plastic limits were between 18 and 21 with plasticity index values ranging between 19 and 27. Moisture content determinations of the recovered samples were between 16 and 60%.

#### 5.9.6 Clayey Silt

Localized deposit of very soft to firm 1.5 m thick cohesive clayey silt was encountered below silty clay at 3.2 m (elevation 244.9) and extended to 4.7 m (elevation 243.4) in borehole M-1. Two N values recorded were WH (weight of hammer and rods). In situ vane testing conducted within the cohesive deposit indicated shear strength values of 18 and 32 kPa with sensitivity values of 6 and 8, respectively.

A grain size distribution result of a clayey silt sample is presented in Figure M-GS-2 and the corresponding Atterberg limits are presented in Figure M-PC-2. The liquid and plastic limits were 31 and 20, respectively with a plasticity index of 11. Two moisture content determinations were 47 and 59%.

#### 5.9.7 Silt

A 0.6 to 0.9 m thick silt was encountered below clayey silt in borehole M-1 at 4.7 m (elevation 243.4), and below silty clay in boreholes M-2A and M-5 at 18.0 and 7.5 m (elevation 238.8 and 239.4) respectively. Three N values recorded were 5, 8 and 22 blows for 23 cm penetration where refusal was met in borehole M-2A on probable bedrock. The silt was generally loose to compact with very dense compactness in borehole M-2A where probable bedrock was underlying the silt layer.

A grain size distribution analysis result for a silt sample is presented in Figure M-GS-3. Two moisture content determinations were 24 and 27%.

#### 5.9.8 Sand and Silt Till

A compact 0.7 m thick sand and silt till was encountered below the silt layer in borehole M-1 at 5.3 m (elevation 242.8) and extended to 6.0 m (elevation 242.1). One N value recorded was 10.

A grain size distribution analysis result for a sand and silt sample is presented in Figure M-GS-4.

#### 5.9.9 Sand

A compact 0.6 m thick sand deposit was encountered below the sand and silt till layer at 6.0 m (elevation 242.1) in borehole M-1 and extended to the termination depth of the borehole at 6.6 m (elevation 241.5), where refusal was met on probable boulder. One N value recorded was 10.

#### 5.9.10 Probable Bedrock

The probable bedrock was inferred by penetration refusal in boreholes M2-A, and M-5 at 18.7 and 8.4 m (elevation 238.1 and 238.5).

#### 5.9.11 Groundwater

Open water around the end of the culvert, north of WBL, was measured at around elevation 247.5.

In boreholes M-1, M-2A, M-4 and M-5, groundwater was not established due boreholes charged with water during augering. Groundwater was not encountered during and upon completion of augering in boreholes M-2 and M-3.

The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 5.10 Culvert N1 (G30)– Station 19+820 WBL

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing N1-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on December 13 and 18, 2012 and on April 29, 2013. A total of three boreholes, N1-1 to N1-3, were drilled to 1.1 to 6.2 m. A single dynamic cone penetration test (DCPT) was conducted 2.0 m northwest of borehole N1-3 to 5.1 m. A total of three auger probes were conducted. One auger probe was advanced near borehole N1-1 to 0.8 m depth where refusal was met on probable bedrock. Two auger probes were advanced near south end of the N1 culvert in the median to 0.5 m where refusal was met on probable boulder and to 1.1 m where refusal was not met. The data was summarized in Record of Borehole N1-3 sheet.

One pavement hole was conducted at the culvert location and the subsurface encountered included 410 mm thick asphaltic concrete over 160 mm thick granular material which in turn was underlain by 250 mm thick sand. The pavement hole met refusal on rockfill at 0.82 m.

The laboratory grain size distribution charts are presented in Figures N1-GS-1 and N1-GS-2 and Atterberg Limits results are presented in Figure N1-PC-1. All of the test results are summarized on the Record of Borehole sheets.

Three boreholes (N1-1 to N1-3) were investigated along the alignment of this culvert to depths ranging from 1.1 to 6.2 m. Generally, in borehole N1-1, surficial sand and gravel fill was encountered to 1.1 m where refusal on probable bedrock was met. A bedrock outcrop was visible 2.0 m north of borehole N1-1 during investigation. An auger probe advanced near borehole N1-1 met refusal on probable bedrock at 0.8 m. In borehole N1-2, surficial 1.1 m thick sand and gravel fill was encountered overlying 1.0 m thick cobbles and boulders under which stiff to very soft 3.4 m thick silty clay and a 0.6 m thick loose to compact silty sand deposit was encountered. Probable bedrock was mantling the silty sand deposit at 6.2 m. Surficial topsoil, mixed with gravel and cobbles, was encountered in borehole N1-3 overlying a soft to firm clayey silt fill that overlain a 0.9 m thick compact silt layer. Refusal was met on probable boulder at 2.1 m in borehole N1-3. The DCPT advanced near borehole N1-3 penetrated through to 5.1 m where refusal was not met. One of the two auger probes conducted near borehole N1-3 met refusal on probable boulder at 0.5 m and the other probe penetrated through to 1.1 m where refusal was not met.

### 5.10.1 Topsoil

Surficial 300 mm thick dark brown topsoil including gravels and cobbles was encountered in borehole N1-3 which extended to elevation 256.9. One N value of 3 was recorded within the topsoil layer. One moisture content determination was 103%.

### 5.10.2 Fill

Surficial fill was encountered in boreholes N1-1 and N1-2 and extended to 1.1 and 2.2 m (elevation 255.6 and 258.7), respectively. In borehole N1-1 the fill included sand and gravel and in borehole N1-2 the fill included sand and gravel overlying cobbles and boulders. A 0.9 m thick firm clayey silt fill layer was encountered below the topsoil layer in borehole N1-3 at 0.3 m (elevation 256.9), which extended to 1.2 m (elevation 256.0). Organics were found mixed with clayey silt deposit. One N value of 5 was recorded.

Borehole N1-1 met refusal on probable bedrock below the fill layer at 1.1 m, elevation 255.6.

### 5.10.3 Silty Clay

A cohesive stiff to very soft silty clay deposit was encountered below the fill at 2.2 m (elevation 258.7) in borehole N1-2 and extended to 5.6 m (elevation 255.3). N values recorded were between 1 and 15 in a decreasing manner with depth. Further, one in-situ vane test conducted in the cohesive layer indicated a shear strength value of 72 kPa, with a sensitivity value of 4.

Grain size distribution analyses results for two silty clay samples are presented in Figure N1-GS-1 with a corresponding plasticity chart for the samples is presented in Figure N1-PC-1. The liquid limits of the samples are 43 and 47 and corresponding plastic limits of 22 and 23, with plasticity indices of 21 and 24. Moisture content determinations were between 22 and 56%.

### 5.10.4 Silty sand

A loose 0.6 m thick silty sand deposit was encountered in borehole N1-2 below the silty clay layer at 5.6 m (elevation 255.3) and extended to 6.2 m (elevation 254.7). N values recorded were 9 and 20 blows for 15 cm penetration where refusal was met on probable bedrock. One moisture content determination recorded was 33%.

### 5.10.5 Silt

A 0.9 m thick compact silt was encountered below the clayey silt layer in borehole N1-3 at 1.2 m (elevation 256.0) and extended to termination depth of borehole 2.1 m (elevation 255.1), where auger refusal was met on probable boulder. One N value recorded was 15. A DCPT was extended into probable sandy silt to 5.1 m (elevation 252.1), where refusal was not met.

A grain size distribution of a sandy silt sample is presented in Figure N1-GS-2. One moisture content determination was 26%.

### 5.10.6 Probable Bedrock

The probable bedrock was inferred by auger refusal in boreholes N1-1 and N1-2 at 1.1 m (elevation 255.6) and 6.2 m (elevation 254.7). An auger probe 2.0 m west of borehole N1-1 met refusal on probable bedrock at 0.8 m depth.

#### 5.10.7 Groundwater

Open water around the end of the culvert, south of WBL, was measured at about elevation 257.2.

In borehole N1-2, groundwater was observed at 5.6 m (elevation 255.3) during augering. In borehole N1-3, groundwater was observed at 1.2 m (elevation 256.0) and upon completion of augering groundwater was observed at ground surface (elevation 257.2). No groundwater was observed in borehole N1-1.

The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 5.11 Culvert N2 (G31) – Station 19+850 EBL

Data from borehole N1-3 (common borehole for culverts N1 and N2) was used to assess the subsurface ground condition at the N2 culvert location.

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing N2-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on December 6 and 18, 2012 and on April 29, 2013. A total of three boreholes, N1-3, N2-1 and N2-2 were drilled 1.4 to 8.7 m. A total of two dynamic cone penetration tests (DCPTs) were Advanced near boreholes N1-3 and N2-2 to 5.1 and 1.1 m, respectively and the data was compiled in Record of Borehole sheets. A total of five auger probes were conducted. Two auger probes were conducted in the median. The auger probe at the south end of N1 culvert extended to 1.4 m where refusal was not met and the auger probe at the north end of N2 culvert extended to 0.5 m where refusal was met on probable boulder. At the south end of culvert N2, three auger probes were advanced to 0.8 to 1.4 m of which one auger probe conducted 3.0 m south of the culvert end met refusal on probable bedrock at 0.8 m. The data was summarized in Record of Borehole N1-3 and N2-2 sheets. The data was summarized in the Record of Borehole N1-3 and N2-2 sheets.

Two pavement holes were conducted at the culvert location and the subsurface encountered included 75 and 130 mm thick asphaltic concrete over 415 and 450 mm thick granular material which in turn was underlain by 310 and 345 mm thick sand. The pavement holes met refusal on rockfill at 0.8 and 0.93 m.

The laboratory grain size distribution chart is presented in Figures N2-GS-1 and N1-GS-2.

The subsurface stratigraphy revealed in borehole N2-1 consisted a 0.8 m thick sand and gravel fill over 3.7 m thick rockfill that overlaid a 1.3 m thick clayey silt. Below the clayey silt, a compact 2.9 m thick sand deposit was encountered and mantled the probable bedrock at 8.7 m. The general stratigraphy in borehole N2-2 revealed a surficial 200 mm thick topsoil overlying a 1.2 m thick loose to compact silt mantling the probable bedrock at 1.4 m. DCPT conducted near borehole N2-2 penetrated through to termination depth of 1.1 m. In borehole N1-3, topsoil of 300 mm thickness overlaid a 0.9 m thick firm clayey silt fill. A compact 0.9 m thick silt layer was encountered below the fill deposit where refusal was met on probable boulder at 2.1 m. DCPT conducted near borehole N1-3 penetrated through probable silt to 5.1 m termination depth.

### 5.11.1 Topsoil

Surficial dark brown silty topsoil of 300 and 200 mm thickness was encountered in boreholes N1-3 and N2-2, extending to elevation 256.9 and 257.1, respectively. Gravel and cobbles were mixed with the topsoil in borehole N1-3. . One N value of 3 was recorded within the topsoil layer in borehole N1-3. The moisture content determination of the topsoil samples were 27 and 103%.

#### 5.11.2 Fill

A 4.5 m thick fill was encountered surficially in borehole N2-1 extending to elevation 256.3. This unit included sand and gravel, extending from the ground surface to 0.8 m (elevation 260.8 to 260.0), overlying rockfill, from 0.8 to 4.5 m (elevation 260.0 to 256.3).

In borehole N1-3, firm clayey silt fill layer of 0.9 m thickness was encountered below the topsoil at 0.3 m (elevation 256.9), which extended to 1.2 m (elevation 256.0). Organics were found mixed with clayey silt deposit. One N value of 5 was recorded.

#### 5.11.3 Clayey silt

A 1.3 m thick firm clayey silt fill layer, mixed with organics, was encountered at 4.5 m (elevation 256.3) and extended to 5.8 m (elevation 255.0). Boulders were encountered within the clayey silt fill layer. Two N values of 5 and 50 blows for 13 cm penetration were recorded. One moisture content determination was 24%.

#### 5.11.4 Silt

A 1.2 and 0.9 m thick compact silt was encountered below the topsoil in borehole N2-2 and below the clayey silt fill layer in borehole N1-3 at 0.2 and 1.2 m (elevation 257.1 and 256.0) and extended to termination depths of 1.4 (elevation 255.9) and 2.1 m (elevation 255.1), respectively. In borehole N2-2, auger refusal was met on probable bedrock underneath the silt layer and in borehole N1-3 refusal was met on probable boulder. Three N values recorded were 6, 15 and 18.

A grain size distribution of a sandy silt sample is presented in Figure N1-GS-2. One moisture content determination was 26%.

#### 5.11.5 Sand

In borehole N2-1, a 2.9 m thick compact sand layer was encountered at 5.8 m (elevation 255.0) that extended to the termination depth of the borehole 8.7 m (elevation 252.1). Two N values of 10 and 23 were recorded.

A grain size distribution analysis result for a sand sample is presented in Figure N2-GS-1. The moisture content determination of the sand sample was 14%.

#### 5.11.6 Probable Bedrock

The probable bedrock was inferred by casing advancement refusal in borehole N2-1 at 8.7 m (elevation 252.1) and in borehole N2-2 at 1.4 m (elevation 255.9).

#### 5.11.7 Groundwater

Open water around the end of the culvert, south of EBL, was measured at about elevation 255.0.

In borehole N1-3, groundwater was observed at 1.2 m (elevation 256.0) and upon completion of augering groundwater was observed at ground surface (elevation 257.2). Groundwater levels in boreholes N2-1 and N2-2 were not determined because the boreholes were charged with water during drilling. The groundwater level is subject to seasonal fluctuations and rainfall patterns.

## 6. SUMMARIZED SUBSURFACE CONDITIONS - GEOGRAPHIC TOWNSHIP OF WATERS

In summarizing the subsurface at each culvert location, reference is made to the appended Record of Borehole Sheets for details of the subsurface conditions including soil classifications, probable bedrock depth, inferred stratigraphy, standard penetration test results, dynamic cone penetration test results, pocket penetrometer test results, in-situ vane test results, and groundwater observations during and upon completion of augering. The results of laboratory Atterberg plasticity limits, grain size distributions and moisture content determinations are also shown on the Record of Borehole Sheets. The laboratory

Atterberg plasticity limits results and grain size distribution results are also presented in plasticity charts and grain size distribution charts.

### 6.1 Culvert O (W4) – Station 11+753 WBL

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing O-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on December 7 and 11, 2012 and on March 5 and October 31, 2013. A total of three boreholes, O-1 to O-3 were drilled 1.5 to 8.5 m. One dynamic cone penetration was conducted adjacent to borehole O-1 to 6.0 m where refusal was met on probable boulder. An auger probe conducted at the north end of the culvert near borehole O-1 was extended to 1.8 m where refusal was met on probable boulder. The data was summarized in Record of Borehole O-1 sheet.

One pavement hole was conducted at the culvert location and the subsurface encountered included 380 mm thick asphaltic concrete over 170 mm thick granular material which in turn was underlain by 400 mm thick sand. The pavement hole met refusal on rockfill at 0.95 m.

The laboratory grain size distribution chart is presented in Figures O-GS-1 and O-GS-2 and the results are summarized on the Record of Borehole sheet.

The subsurface stratigraphy revealed in borehole O-1 consisted peat at 1.1 m below ice and open water. A 1.1 m thick soft to firm silty was encountered below the peat which overlaid 1.5 m thick loose silt. A 3.1 m thick loose to compact sand was underlying the silt and mantled the probable bedrock at 6.9 m. DCPT conducted near borehole O-1 met refusal at 6.0 m on probable boulder. In borehole O-2, a 2.8 m thick fill over 4.5 m thick firm to stiff clayey silt was encountered. A 1.2 m thick loose sand was encountered below the clayey silt, mantling probable bedrock at 8.5 m. A 200 mm thick topsoil over 1.3 m firm clayey silt was encountered in borehole O-3.

### 6.1.1 Peat / Topsoil

In borehole O-1, peat of 100 mm thickness was encountered below ice and open water at 1.1 m (elevation 262.3).

A 200 mm thick topsoil was encountered in borehole O-3 that extended to elevation 264.3.

### 6.1.2 Fill

Surficial 0.8 m thick sand and gravel fill was encountered in borehole O-2 which overlaid a 2.0 m thick rockfill that extended to 2.8 m (elevation 264.3).

### 6.1.3 Silty Clay

Underneath the peat layer, 1.1 m thick very soft to soft silty clay was encountered which extended to 2.3 m (elevation 261.1). N values recorded were 1 and 2. Two moisture content determinations were 35 and 40%.

### 6.1.4 Clayey Silt

A firm to stiff 4.5 and 1.3 m thick clayey silt deposit was encountered below fill in borehole O-2 and below topsoil in borehole O-3 at 2.8 m (elevation 264.3) and 0.2 m (elevation 264.1) and extended to 7.3 m (elevation 259.8) and 1.5 m (elevation 262.8), respectively. N values recorded ranged between 1 and 10. In situ vane testing conducted within the cohesive deposit indicated a shear strength value of 68 kPa with a sensitivity value of 7. Moisture content determinations were 25 and 37%.

### 6.1.5 Silt

Silt was encountered below the silty clay layer in borehole O-1 at 2.3 m (elevation 261.1) and extended to 3.8 m (elevation 259.6). The silt was loose to compact. Two N values recorded were 4 and 8.

A grain size distribution analysis result for a silt sample is presented in Figure O-GS-1. Two moisture content determinations were 28 and 39%.

#### 6.1.6 Sand

Loose to compact sand of 3.1 and 1.2 m thickness was encountered below silt in borehole O-1 at 3.8 m (elevation 259.6) and below clayey silt at 7.3 m (elevation 259.8) in borehole O-2. The sand extended to 6.9 and 8.5 m (elevation 256.5 and 258.6) in boreholes O-1 and O-2, mantling probable bedrock. N values recorded ranged from 5 to 19.

A grain size distribution analysis result for a sand sample is presented in Figure O-GS-2. Two moisture content determinations were 13 and 19%.

#### 6.1.7 Probable Bedrock

The probable bedrock was inferred by casing advancement refusal in borehole O-1 at 6.9 m (elevation 256.5) and in borehole O-2 at 8.5 m (elevation 258.6).

#### 6.1.8 Groundwater

Open water was encountered at elevation 263.4 below ice in borehole O-1 during and upon completion of augering. In borehole O-2, groundwater was observed in borehole O-2 at 7.3 m (elevation 259.8) during augering. The groundwater level is subject to seasonal fluctuations and rainfall patterns.

### 6.2 Culvert P (W14) – Station 13+598 EBL

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing P-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on December 12 and 13, 2012, March 6 and June 12, 2013. A total of three boreholes, P-1 to P-3, were drilled 7.7 to 8.5 m. Two dynamic cone penetration tests (DCPT) were conducted; one DCPT was conducted 1.0 m east of borehole P-1 to 8.5 m and the second one was conducted 2.0 m east of borehole P-3 to 9.1 m. The DCPT data was compiled in Record of Borehole No P-1 and P-3 sheets. A total of four auger probes were conducted. Two auger probes were conducted either side of the north end of the culvert refusing at 0.6 and 0.8 m on rockfill. The other two auger probes were conducted on either side of the south end of the culvert extending to 1.4 and 1.5 m into silty clay. The data was summarized in the Record of Borehole P-1 and P-3 sheets.

A total of four pavement holes were conducted of which two were at the south and north ends of the culvert. The north and south culvert end holes encountered 105 and 200 mm deep water over 185 and 160 mm thick peat. The peat was underlain by 310 mm (north) and 1.1 m (south) thick silty sand layer. A 0.9 m thick silty clay deposit was encountered below the silty sand layer in the north end pavement hole. Refusal was not met at the two culvert end holes. The subsurface of the other two pavement holes included surficial 310 mm thick asphaltic concrete over 140 mm thick granular material underlain by 350 mm thick sand in 13+600 2.1 RT C/L hole and 410 mm thick granular material over 250 mm thick layer sand in 13+600 5.2 RT C/L hole. The two holes met refusal on rock fill at 0.66 and 0.80 m.

The laboratory grain size distribution charts are presented in Figures P-GS-1 to P-GS-3 and Atterberg Limits results are presented in Figures P-PC-1 and P-PC-2. All of the test results are summarized on the Record of Borehole sheets.

The subsurface stratigraphy revealed surficial 3.2 m thick fill, including 0.9 m sand and gravel pavement fill over rockfill, in borehole P-2 overlying a 4.7 m thick very soft to stiff clayey silt. Surficial 300 mm thick peat was encountered in borehole P-1 overlying cohesionless silt and silt till deposits which are underlain by 3.5 m thick firm to stiff silty clay. In borehole P-3, a 200 mm thick topsoil layer was encountered below the surficial ice and snow. Below the topsoil, a 0.9 m thick organic clayey silt was encountered, which in turn overlain 2.9 m thick firm to stiff silty clay and 1.9 m thick clayey silt deposits. Very loose to compact silt was encountered in all three boreholes extending to 7.7 to 8.5 m. Probable bedrock was encountered in borehole P-2 at 8.5 m.

### 6.2.1 Topsoil/Peat

A 300 mm thick peat was encountered in borehole P1 and extended to elevation 245.6 and in borehole P3, a 200 mm thick topsoil was encountered under the surficial ice and snow cover at 0.3 m (elevation 245.5) and extended to 0.5 m (elevation 245.3).

### 6.2.2 Fill

In borehole P-2, a 0.9 m thick pavement fill consisting sand and gravel was encountered surficially, which overlaid rockfill consisting cobbles and boulders to 3.2 m (elevation 245.5).

### 6.2.3 Upper Silt and Silt Till

Below the peat layer in borehole P-1, a 300 mm thick very loose silt deposit was encountered at 0.3 m (elevation 245.6) and extended to 0.6 m (elevation 245.3). One N value of 2 was recorded for the silt deposit with one moisture content determination of 29%. Underlying the silt, a 0.8 m thick compact silt till was encountered extending to 1.4 m (elevation 244.5). One N value of 25 was recorded.

### 6.2.4 Organic Clayey Silt

A 0.9 m thick firm organic clayey silt was encountered in borehole P-3 below topsoil at 0.5 m (elevation 245.3) and extended to 1.4 m (elevation 244.4). Two N values recorded were 5 and 7. Two moisture content determinations were 36 and 39%.

### 6.2.5 Silty Clay

Stiff to firm silty clay of 3.5 and 2.9 m thickness was encountered in borehole P-1 and P-3 at 1.4 m, elevation 244.5 and 244.4, respectively and extended to 4.9 m (elevation 241.0) and 4.3 m (elevation 241.5). N values recorded ranged between 1 and 5. In situ vane testing conducted within the cohesive deposit obtained 36 to 92 kPa with sensitivities ranging between 5 and 34. Penetrometer test results obtained were between 38 and 87 kPa.

Two grain size distribution analyses results of silty clay samples are presented in Figure P-GS-1 and the corresponding Atterberg limits results are presented in Figure P-PC-1. The liquid limits were 45 and plastic limits were 22 and 23 with plasticity indices of 22 and 23. Moisture content determinations obtained are between 28 to 49%.

### 6.2.6 Clayey Silt

Cohesive soft to stiff clayey silt of 4.7 and 1.9 m thickness was encountered in boreholes P-2 and P-3 at 3.2 m (elevation 245.5) and 4.3 m (elevation 241.5), respectively, and extended to 7.9 m (elevation 240.8) and 6.2 m (elevation 239.6) . N values ranged from WH (weight of hammer and rods) to 12. Three In-situ vane test results obtained 28, 40 and more than 100 kPa with sensitivities of 2 and 7. Two penetrometer test results were 25 and 112 kPa.

A grain size distribution analysis chart is presented in Figure P-GS-2 and the corresponding Atterberg limits are presented in Figure P-PC-2. The liquid and plastic limits of the sample were 33 and 23, respectively, with a plasticity index of 10. Natural moisture content determinations ranged between 26 and 45%.

### 6.2.7 Silt

A continuous layer of very loose to compact silt was encountered in all three boreholes at 4.9 to 7.9 m (elevation 239.6 to 241.0) and extended to termination depths of 7.7 to 8.5 m (elevations 237.6 to 240.2). Further advancement with casing was refused in borehole P-1 at

7.7 m (elevation 238.2) on probable boulder and at 8.5 m (elevation 240.2) in borehole P-2 on probable bedrock. N values recorded were from 3 to 6 with refusal N values of 25 blows for 10 cm penetration in borehole P-1 and 20 blows for 15 cm penetration in borehole P-2 on probable bedrock. Two DCPTs carried out near boreholes P-1 (north end of the culvert) and P-3 (south end of the culvert) extending to 8.5 and 9.1 m (elevation 237.4 and 236.7), respectively.

Grain size analyses results of selected samples are presented in Figure P-GS-3. Moisture content determinations ranged between 20 and 38%.

#### 6.2.8 Probable Bedrock

Probable bedrock was encountered in borehole P-2 at 8.5 m (elevation 240.2).

#### 6.2.9 Groundwater

At the time of investigation, the water level at the south end of the culvert was at about elevation 245.0.

Surficial water (elevation 245.8) was encountered in borehole P-3 during and upon completion of augering. In borehole P-1, groundwater was encountered at 0.3 m (elevation 245.6) during augering and at surface (elevation 245.9) upon completion of augering. During augering, groundwater was observed at 7.3 m (elevation 241.4) in borehole P-2; however, groundwater was not observed upon completion of augering. The groundwater level is subjected to seasonal fluctuations and rainfall patterns.

### 6.3 Culvert Q (W17) – Station 14+943 WBL

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing Q-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on April 11, 25 and June 19, 2013. A total of three boreholes, Q-1 to Q-3, were drilled 0.8 to 14.6 m. One dynamic cone penetration test (DCPT) was conducted 2.0 m south of borehole Q-1 to 8.4 m. The data was compiled in Record of Borehole Q-1 sheet. Two auger probes were conducted on either side of the north end of the culvert near borehole Q-1 penetrating through sand to 1.4 m where refusal was not met. The data was summarized in the Record of Borehole Q-1 sheet.

One pavement hole was conducted at the culvert location and the subsurface encountered included 160 mm thick asphaltic concrete over 150 mm thick granular material which in turn was underlain by 270 mm thick sand. The pavement hole met refusal on rockfill at 0.66 m.

The laboratory grain size distribution charts are presented in Figures Q-GS-1 to Q-GS-4 and Atterberg Limits results are presented in Figures Q-PC-1 to Q-PC-3. All of the test results are summarized on the Record of Borehole sheets.

The subsurface stratigraphy revealed surficial 1.0 m thick loose sand in borehole Q-1 overlying a 3.6 m thick firm o stiff clay layer over compact silt to termination depth of 9.0 m. A pavement structure, consisting 150 mm thick asphaltic concrete over sand and gravel base course, was encountered in borehole Q-2 overlying two distinct layers of fill materials to 6.4 m. Underlying the fill, a 2.4 and 3.1 m thick stiff to very stiff clayey silt and firm to stiff clay was encountered, respectively. Compact silt was encountered underlying the clay to 14.6 m. In borehole Q-3, gravel was encountered surficially, which extended to 0.8 m where refusal was met on probable rockfill. Two auger probes were advanced through sand some silt to 1.4 m on either side of the north end of the culvert near borehole Q-1.

### 6.3.1 Pavement Structure

A 150 mm thick asphaltic concrete pavement over sand and gravel base course was encountered in borehole Q-2. The sand and gravel extended to 0.8 m (elevation 268.5).

### 6.3.2 Fill

Underlying the pavement structure in borehole Q-2, two distinct layers of fill materials were encountered. A 4.6 m thick rockfill, consisting cobbles and boulders, overlying a 0.8 m thick silt fill was encountered extending to 6.4 m (elevation 262.9). Two N values of WH (weight of hammer and rods) and 6 were recorded in the silt fill layer. Two moisture content determinations of silt fill were 29 and 30%.

### 6.3.3 Gravel

Surficial gravel was encountered in borehole Q-3 and extended to 0.8 m (elevation 262.3), where auger refusal was met on probable rockfill.

### 6.3.4 Sand

A 1.0 m thick loose sand was encountered surficially in borehole Q-1 and extended to elevation 262.1. One N value recorded was 9.

### 6.3.5 Silty Clay

A firm to stiff 3.6 m thick silty clay layer was encountered below the sand deposit in borehole Q-1 at 1.0 m (elevation 262.1) which extended to 4.6 m (elevation 258.5). N values recorded were between 4 and 7. Penetrometer test values recorded were 25 to 75 kPa. In-site vane tests conducted in the cohesive layer obtained shear strength values of 44 to more than 100 kPa with sensitivity values of 10 and 11.

Grain size distribution analysis of a selected sample in presented in Figure Q-GS-1 and the corresponding Atterberg limits results are presented in Figure Q-PC-1. The liquid and plastic limits were 40 and 18, respectively, with a plasticity index of 22. Moisture content determinations were between 34 and 45%.

### 6.3.6 Clayey Silt

A 2.4 m thick very stiff to stiff clayey silt layer was encountered below fill materials in borehole Q-2 at 6.4 m (elevation 262.9) and extended to 8.8 m (elevation 260.5). Two N values recorded were 9 and 16. In-situ vane test obtained a shear strength value of more than 100 kPa.

A grain size distribution analysis of a selected clayey silt sample is presented in Figure Q-GS-2 and the corresponding Atterberg limits are presented in Figure Q-PC-2. The liquid and plastic limits were 30 and 16, with a plasticity index of 14. Two moisture content determinations were 28 and 33%.

### 6.3.7 Clay

In borehole Q-2, underlying the clayey silt is a 3.1 m thick cohesive stiff to firm clay encountered at 8.8 m (elevation 260.5) and extended to 11.9 m (elevation 257.4). Two N values recorded were 3 and 5. One penetrometer test value obtained was 25 kPa. In-situ vane test obtained a shear value of 80 kPa with a sensitivity value of 8.

A grain size distribution analysis of a selected clay sample is presented in Figure Q-GS-3 and the corresponding Atterberg limits are presented in Figure Q-PC-3. The liquid and plastic limits were 55 and 25, with a plasticity index of 30. One moisture content determination was 45%.

### 6.3.8 Silt

Below the silty clay in borehole Q-1, a 4.4 m thick compact silt was encountered at 4.6 m (elevation 258.5) which extended to 9.0 m (elevation 254.1) where refusal was met on probable bedrock. In borehole Q-2, compact silt was encountered at 11.9 m (elevation 257.4) and extended to the termination depth of 14.6 m (elevation 254.7). N values recorded were 13 to 22 with a high N value of 20 blows for 15 cm penetration in borehole Q-1 where refusal was met on probable bedrock.

A grain size distribution analysis of a selected sample is presented in Figure Q-GS-4. Moisture content determinations were 24 to 27%.

#### 6.3.9 Probable Bedrock

Probable bedrock was encountered in borehole Q-1 at 9.0 m (elevation 254.1) where refusal was met.

#### 6.3.10 Groundwater

Water was encountered on the surface (elevation 265.1) at borehole Q-1 location during and after augering. In borehole Q-2, groundwater was encountered at 13.7 m (elevation 256.1) during augering and at surface (elevation 269.8) after completion of augering, indicating an artesian condition. The groundwater level is subjected to seasonal fluctuations and rainfall patterns.

### 6.4 Culvert R (W25) – Station 15+687 C/L

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing R-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface investigation was carried out on April 10, 11, and 24 and May 8 and 9, 2013. A total of five boreholes, R-1 to R-5, were drilled 1.2 to 15.8 m. A total of two dynamic cone penetration tests (DCPTs) were conducted. One DCPT was conducted 2.0 m north of borehole R-1 to 10.0 m and the second one was conducted 2.0 m west of borehole R-3 to 15.0 m. The data was compiled in Record of Borehole No R-1 and R-3 sheets.

Two pavement holes were conducted, each at EBL and WBL, at the culvert location. The subsurface encountered in the EBL pavement hole included 380 mm thick asphaltic concrete over 230 mm thick granular material which in turn was underlain by 370 mm thick sand. The pavement hole met refusal on rockfill at 0.71 m. The subsurface encountered in the WBL pavement hole included 340mm thick asphaltic concrete over 340 mm thick granular material which in turn was underlain by 300 mm thick silt and 520 mm thick clayey silt. No refusal was met at 1.5 m.

The laboratory grain size distribution charts are presented in Figures R-GS-1 to R-GS-5 and Atterberg Limits results are presented in Figures R-PC-1 and R-PC-2. All of the test results are summarized on the Record of Borehole sheets.

The subsurface stratigraphy revealed surficial 200 and 300 m thick topsoil in boreholes R-1 and R-3. A pavement structure was encountered in borehole R-4 which extended to 1.2 m. Surficial 6.0 m fill was encountered in borehole R-2. Surficial 0.6 m thick clayey silt was overlying a 0.6 m thick silt in borehole R-5. Fill was encountered below the topsoil in borehole R-3 and below the pavement structure in boreholes R-4 extending to 7.0 and 8.8 m, respectively. A 1.0 m thick organic silt was encountered below the topsoil in borehole R-1. A 3.2 m thick stiff to very stiff clayey silt layer overlying a 2.0 m thick compact silt was encountered in borehole R-4 below fill. Very loose to compact cohesionless sandy/silty soils were encountered at 0.6 to 12.0 m and extended 1.2 to 15.8 m.

### 6.4.1 Topsoil

Topsoil of 200 and 300 mm thickness was encountered surficially in boreholes R-1 and R-3 extending to elevation 252.3 and 256.7, respectively.

### 6.4.2 Pavement Structure

A 150 mm thick asphaltic concrete pavement was encountered in borehole R-4 overlying sand and gravel base course fill which extended to 1.2 m (elevation 256.8). One N value recorded was 26. One moisture content determination was 8%.

### 6.4.3 Fill

Fill was encountered surficially in boreholes R-2, below topsoil in borehole R-3 at 0.3 m (elevation 256.7) and below the pavement structure in borehole R-4 at 1.2 m (elevation 256.8) and extended to 6.0 to 8.8 m (elevation 251.8 to 249.2). Fill materials varied between the boreholes and included rockfill (borehole R-4), sand and gravel (borehole R-2), silt and clayey silt. N values recorded were from 4 to 34.

Grain size distribution analyses results of selected samples are presented in Figure R-GS-1. Atterberg limits results are presented in Figure R-PC-1. The liquid limit ranged from 24 to 31 and plastic limit ranged from 18 to 20 with plasticity indices between 6 and 11. Moisture content determinations ranged between 12 to 30%.

#### 6.4.4 Organic Silt

Below the topsoil, a 1.0 m thick organic silt was encountered in borehole R-1 at 0.3 m (elevation 252.3) and extended to 1.2 m (elevation 251.3). Two N values recorded were 3 and 7. Two moisture content determinations were 22 and 25%.

#### 6.4.5 Clayey Silt

Surficial 0.6 m clayey silt was encountered in borehole R-5 extending to elevation 247.8. A 3.2 m thick stiff to very stiff clayey silt layer was encountered below the fill in borehole R-4 at 8.8 m (elevation 249.2) which extended to 12.0 m (elevation 246.0). Two N values recorded were 12 and 17.

A grain size distribution analysis of a selected sample is presented in Figure R-GS-2 and the corresponding Atterberg limits results are presented in Figure R-PC-2. The liquid and plastic limits were 26 and 19 with a plasticity index of 7. Two moisture content determinations were 18 and 24%.

#### 6.4.6 Sand

A 0.8 m thick compact sand layer was encountered below the clayey silt layer in borehole R-4 at 14.0 m (elevation 244.0) and extended 14.8 m (243.2). One N value recorded was 15. One moisture content determination was 17%.

#### 6.4.7 Silt

A 2.0 and 6.5 m thick very loose to compact silt was encountered below organic silt in borehole R-1 at 1.2 m (elevation 251.3) extending to 4.3 m (elevation 248.2), below sandy silt at 8.5 m (elevation 248.5) in borehole R-3 which extended to 15.0 m (elevation 242.0) and below clayey silt in borehole R-4 at 12.0 m (elevation 246.0) extending to 14.0 m (elevation 244.0). Loose to compact silt was encountered below fill layer in borehole R-2 at 6.0 m (elevation 251.8) and extended to the termination depth of 12.8 m (elevation 245.0). Silt was encountered below clayey silt in borehole R-5 at 0.6 m (elevation 247.8) and extended to termination depth of 1.2 m (elevation 247.2). N values recorded were from 1 (possibly due to hydraulic disturbance during sampling in borehole R-3) to 23.

Grain size analyses results of selected samples are presented in Figure R-GS-3. Moisture content determinations were between 18 and 26%.

#### 6.4.8 Sandy Silt

A 3.2 and 1.5 m thick loose to compact sandy silt was encountered in borehole R-1 and R-3 at 4.3 and 7.0 m (elevation 248.2 and 250.0), respectively and extended to 7.5 and 8.5 m (elevation 245.0 and 248.5). A DCPT was advanced, 2.0 m north of borehole R-1, through silt and sand layer into probable sandy silt at 8.2 m (elevation 244.3) and extended to termination depth at 10.0 m (elevation 242.5). N values recorded were 6 to 11. Moisture content determinations were from 22 to 24%.

#### 6.4.9 Silt and Sand

In borehole R-1, loose to compact silt and sand unit was encountered below sandy silt at 7.5 m (elevation 245.0) and extended to termination depth at 8.2 m (elevation 244.3). One N value recorded was 8.

A grain size distribution analysis result is presented in Figure R-GS-4. One moisture content determination recorded was 19%.

#### 6.4.10 Silty Sand

Compact silty sand was encountered below silt in borehole R-4 at 14.8 m (elevation 243.2), and extended to termination depth of 15.8 m (elevation 242.2). One N value recorded was 16. One moisture content determination recorded was 18%.

#### 6.4.11 Sand and Silt

Very loose sand and silt was encountered below silt in borehole R-3 at 15.0 m (elevation 242.0) and extended to termination depth of 15.8 m (elevation 241.2). One N value recorded was 1 in borehole R-3. The low N value resulted possibly due to hydraulic disturbance during sampling.

A grain size distribution analysis result is presented in Figure R-GS-5. One moisture content determination recorded was 23%.

#### 6.4.12 Groundwater

At the time of investigation, no flowing water was observed at the north end of the culvert at WBL.

During augering, groundwater was encountered at 1.2 to 6.9 m (elevation 250.1 to 251.3) in boreholes R1 to R4. Upon completion of augering, groundwater was measured at 6.1 and 6.4 m (elevation 251.7 and 250.6), respectively, in boreholes R-2 and R-3.

The groundwater level is subjected to seasonal fluctuations and rainfall patterns.

## 6.5 Culvert S (W26) – Station 16+125 EBL

The subsurface investigation was carried out on April 23, June 13 and 14, 2013. A total of four boreholes, S-1 to S-4, were drilled 9.7 to 14.3 m. A total of two dynamic cone penetration tests (DCPT) were conducted; one was advanced 3.0 m north of culvert end to 10.7 m and the second one was advanced 2.0 m west of borehole S-4 to 10.4 m. The data was compiled in Record of Borehole No S-1 and S-4 sheets.

The laboratory grain size distribution charts are presented in Figures S-GS-1 to S-GS-6 and Atterberg Limits results are presented in Figures S-PC-1 and S-PC-2. All of the test results are summarized on the Record of Borehole sheets.

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing S-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface stratigraphy revealed surficial 50 mm thick asphaltic concrete pavement in borehole S-2 overlying a 550 mm thick sand and gravel deposit. Surficial fill was encountered in boreholes S-1, S-3 and S-4 which extended to 0.6 to 6.7 m. Underlying the fill, locally, a 200 mm thick topsoil deposit was encountered in borehole S-2, 1.3 m thick deposit of loose to compact silt till was encountered in borehole S-1 and a 1.6 m thick firm clayey silt in borehole S-4. A continuous layer of very loose to compact silt was encountered in all four boreholes at 2.2 to 9.0 m extending to 5.2 to 14.3 m. Boreholes S-2 and S-3 were terminated in the silt stratum. In boreholes S-1 and S-4, compact sandy silt was encountered at 8.2 and 5.2 m below the silt layer and extended to the termination depth for both boreholes to 9.7 m.

### 6.5.1 Pavement Structure

A 50 mm thick surficial asphaltic concrete overlying sand and gravel base course was encountered in borehole S-2 which extended to 0.6 m (elevation 255.7).

### 6.5.2 Fill

Surficial fill was encountered in boreholes S-1, and S-4 which extended to 0.9 m (elevation 249.4) and 0.6 m (elevation 249.2), respectively. A 5.0 m thick variable fill was encountered below the pavement structure in borehole S-2 at 0.6 m (elevation 255.7) and extended to 5.6 m (elevation 250.7). In borehole S-3, a 0.8 m thick sand and gravel base course pavement fill contacted at surface (elevation 256.7) overlaid a 5.9 m thick variable fill was which extended to 6.7 m (elevation 250.0). Fill materials included silt, silty sand and silt and sand with organic inclusions. N values recorded ranged from 1 to 26.

A grain size distribution analyses chart is presented in Figure S-GS-1. Natural moisture content determinations ranged between 4 and 39%.

### 6.5.3 Topsoil

Localized 200 mm thick topsoil deposit was encountered below the fill layer in borehole S-2 at 5.6 m (elevation 250.7) and extended to 5.8 m (elevation 250.5).

### 6.5.4 Silt Till

Underlying the fill layer in borehole S-1 is a local 1.3 m thick loose to compact silt till deposit encountered at 0.9 m (elevation 249.4), extending to 2.2 m (elevation 248.1). Two N values recorded were 8 and 10.

A grain size distribution analysis is presented in Figure S-GS-2. Two moisture content determinations were 14 and 27%.

#### 6.5.5 Silty Clay

In borehole S-3, a local 2.3 m thick firm to stiff silty clay deposit was encountered at 6.7 m (elevation 250.0) and extended to 9.0 m (elevation 247.7). Two N values recorded were WH (penetration due to weight of hammer and rods) and 1. In situ vane testing conducted within the cohesive deposit obtained shear strength values of 60 and 73 kPa with a sensitivity values of 3 and 5.

A grain size distribution analysis result of a silty clay sample was presented in Figure S-GS-3 and the corresponding Atterberg limit results are presented Figure S-PC-1. The liquid and plastic limits obtained 46 and 21 with a plasticity index of 25. Two moisture content determinations were 35 and 42%.

#### 6.5.6 Clayey Silt

A local 1.6 m thick firm clayey silt was underlying the fill in borehole S-4 at 0.6 m (elevation 249.2) and extended to 2.2 m (elevation 247.6). Two N values recorded were 4 and 5.

A grain size distribution analysis result of a clayey silt sample was presented in Figure S-GS-4 and the corresponding Atterberg limit results are presented Figure S-PC-2. The liquid and plastic limits obtained 27 and 15, respectively, with a plasticity index of 12. Two moisture content determinations were 19 and 25%.

#### 6.5.7 Silt

A continuous 3.0 to 8.5 m thick silt layer was encountered in all four boreholes at 2.2 to 9.0 m (elevation 247.7 to 250.5). The layer extended to 5.2 to 14.3 m, (elevation 242.0 to 244.6). The silt layer was penetrated through in boreholes S-1 and S-4. The silt was loose to compact. N values recorded ranged between 2 and 20. Boreholes S-2 and S-3 were terminated in the silt stratum at 14.3 m (elevation 242.0 and 242.4, respectively).

Grain size distribution analyses results of selected silt samples are presented in Figure S-GS-5. Moisture content determination ranged between 17 and 28%.

#### 6.5.8 Sandy Silt

In boreholes S-1 and S-4, compact sandy silt layer was encountered at 8.2 and 5.2 m (elevation 242.1 and 244.6), respectively and extended to the termination depth for both boreholes to 9.7 m (elevation 240.6 and 240.1). N values recorded were between 11 and 19.

Grain size distribution analyses results of selected sandy silt samples are presented in Figure S-GS-6. Moisture content determination ranged between 20 and 27%.

#### 6.5.9 Groundwater

During augering, groundwater was encountered in boreholes S-1, S-2 and S-3 at 2.1 to 7.8 m, elevation 248.2 to 249.6. Surficial water was observed in borehole S-4 during augering at elevation 249.8. Upon completion of augering, groundwater was observed at surface (elevation 250.3) in borehole S-1 and in the remaining three boreholes groundwater was observed at 2.3 to 7.9 m, elevation 247.5 to 248.8.

The groundwater level is subject to seasonal fluctuations and rainfall patterns.

## 7. CLOSURE

Mr. F. Portela and Mr. S. Aziz carried out the field investigations under the supervision of Mr. A. Desira, Project Supervisor and Mr. C. M. P. Nascimento, P. Eng., Project Manager. LandCore Drilling Ltd. and Underground Sonic Drilling Services Inc. supplied the drill equipment for the subsurface exploration. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.

The Foundation Investigation Report (FIR) for Culvert A was prepared by Mr. A. Desira, MEng, P.Eng., Culvert C1 was prepared by Ms. N. Balakumaran, P.Eng., for Culverts F to H were prepared by Mr. H. Gharegrat, MEng, P.Eng and the remaining culvert FIRs were prepared by Mr. N. Rahman, P.Eng, and reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact. Mr. C. M. P. Nascimento, P. Eng., Project Manager conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

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**FOUNDATION DESIGN REPORT**
**for**
**22 CULVERT REPLACEMENTS**
**RESURFACING FROM 0.8 KM EAST OF**
**HIGHWAY 17/ REGIONAL ROAD 55 INTERCHANGE AT**
**SUDBURY WESTERLY 21.8 KM**
**GEOGRAPHIC TOWNSHIPS OF DENISON, GRAHAM AND WATERS**
**GREATER SUDBURY AREA, ONTARIO**
**G.W.P. 5146-09-00**

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April 16, 2014

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Peto MacCallum Ltd.
CONSULTING ENGINEERS

**FOUNDATION DESIGN REPORT**
for
22 Culvert Replacements
Resurfacing from 0.8 km East of
Highway 17/Regional Road 55 Interchange
At Sudbury Westerly 21.8 km
Greater Sudbury Area, Ontario
GWP 5146-09-00

## 1. INTRODUCTION

The project is associated with the planned resurfacing of Highway 17 extending from 0.8 km east of the Highway 17/Regional Road 55 interchange at Sudbury westerly for 21.8 km and involves the replacement of 22 culverts out of the existing 80 culverts within the project limits. The investigation was carried out by Peto MacCallum Ltd. (PML) for AECOM Canada Ltd (AECOM) on behalf of the Ministry of Transportation of Ontario (MTO).

In total, 22 culverts are proposed to be replaced within the project limits and are listed in the following table including the Geographic Township, station, existing culvert type and location of the culvert on Highway 17 with respect to east bound lane (EBL), centreline (C/L) and west bound lane (WBL):

| PML FOUNDATION CULVERT ID | MTO CULVERT ID | GEOGRAPHIC TOWNSHIP | STATION | EXISTING CULVERT TYPE | HIGHWAY 17 SECTION |
|---|---|---|---|---|---|
| A | D4 | Denison | 13+410 | 760x38.6 CSP | EBL |
| B | D7 | | 15+506 | 760x79 CSP | C/L |
| C1 | D9 | | 16+110 | 1070x28.4 CSP | WBL |
| C2 | D9 | | 16+110 | 1070x28.4 CSP | EBL |
| D | D11 | | 16+740 | 910x162.4 CSP | C/L |
| E | D12 | | 16+958 | 760x79.7 CSP | C/L |
| F | G3 | Graham | 10+525 | 1070x21.2 CSP | EBL |
| G | G4 | | 10+685 | 910x33.9 CSP | EBL |
| H | G5 | | 10+910 | 910x36.5 CSP | EBL |
| I | G7 | | 12+273 | 760x83.0 CSP | C/L |
| J1 | G8 | | 12+620 | 1070x33.0 CSP | WBL |
| J2 | G9 | | 12+630 | 1070x30 CPS | EBL |
| K | G10 | | 12+850 | 780x23 CSP | WBL |
| L | G25 | | 17+894 | 1830x95 CSP | C/L |

| PML FOUNDATION CULVERT ID | MTO CULVERT ID | GEOGRAPHIC TOWNSHIP | STATION | EXISTING CULVERT TYPE | HIGHWAY 17 SECTION |
|---|---|---|---|---|---|
| M | G26 | | 18+882 | 1520x81 CSP | C/L |
| N1 | G30 | | 19+820 | 1220x35.6 CSP | WBL |
| N2 | G31 | | 19+850 | 1220x37 CSP | EBL |
| O | W4 | Waters | 11+753 | 910x31.7 CSP | WBL |
| P | W14 | | 13+598 | 910x26.9 CSP | EBL |
| Q | W17 | | 14+943 | 910x36.7 CSP | WBL |
| R | W25 | | 15+687 | 1220x108.2 CSP | C/L |
| S | W26 | | 16+125 | 910x48.2 CSP | EBL |

The detail design for the resurfacing of 21.8 km of the four lane section of Highway 17 west of Sudbury is presented under a separate cover.

It is understood that the culvert replacements will be carried out by closing portions of Highway 17 in one direction and detouring traffic and hence, requirements for shoring to maintain traffic may be minimized.

Based on the foundation investigations at the existing culvert locations, replacement of the culverts by the open excavation method is considered feasible from a geotechnical viewpoint. The geotechnical suitability of installing the culverts using trenchless methods, such as jack and bore, pipe ramming and pipe bursting are discussed as alternative methods.

'Red-flag' and critical issues are noted within the report. Neither MTO nor the consultants assume responsibility or liability for alerting the contractor to all 'red-flag' or critical issues. The requirement to deliver acceptable construction quality remains the responsibility of the contractor.

## 2. DISCUSSION

The project comprises replacement of 22 existing culverts.

This report provides an overview and evaluation of alternatives for rehabilitation or replacement strategies and detail design recommendations for culvert installation by open cut.

The following documents are referenced:

- NA Society for Trenchless Technology No-Dig Show (2010). Culvert Replacement Using Pipe Ramming, Tunneling or Pipe Jacking. Craig Camp, Glenn Boyce and Al Tenbusch.
- ASCE (2009). Osborn, L. (Editor). Trenchless Renewal of Culverts and Storm Sewers. ASCE Manuals and Reports on Engineering Practice No. 120.

Corrugated steel pipes (CSP's) were typically used for drainage through highway embankments including those within the project limits. These culverts had a 50-year life cycle and that life cycle is ending. Many of these culverts have since failed or begun to fail through corrosion, sagging and/or collapse and culvert replacement is now a major issue.

The culverts were constructed using open trench excavation methods while the highway was under construction. Suitable fill was used to bury the culvert as the highway embankment was constructed. The backfill was typically native materials consisting of soil or rock fill.

The critical foundations engineering issue for this project is the existence of mixed fill and rock fill embankments at various culvert locations and the difficult construction challenges that their existence presents. A foundation investigation has been carried out to requirements, but it is not possible to verify the composition of fill embankments along the entire extent of culvert alignments based on a typical pattern of boreholes due to the variable nature of the fill. Hence, there is a high risk of encountering differing subsurface conditions to those discovered at the borehole locations. Due to this uncertainty, it would be prudent to consider options that mitigate the risk of contractor claims, construction delays and resulting increased costs stemming from differing subsurface conditions and the difficulties associated with excavation, shoring and tunnelling in mixed fill and rock fill embankments.

# 3. RECOMMENDATIONS

## 3.1 General Recommendations

While the recommendations presented in this report may be used for detail design purposes in the design-bid-build (DBB) contract environment, they are not intended to be used for detail design or construction purposes in the design-build (DB) contract environment. Where design-build contractor procurement is adopted, the design-build RFP should include requirements for the design-builder to carry out sufficient additional subsurface investigation and foundation design to satisfy themselves as to the sufficiency of the information for their design and construction purposes including their selection of method and equipment.

For either DBB or DB, an NSSP shall be included in the contract documents advising the Contractor that variable mixed fill and rock fill embankments shall be anticipated at all culvert crossings and that the Contractor shall use methods and equipment that are appropriate for the work and capable of dealing with the conditions encountered including, but not limited to various tunnelling techniques and ground improvement techniques such as grouting.

From a foundation engineering perspective, the design and construction of any new or replacement culverts should consider the following; over sizing culverts to permit a future cycle of slip lining before replacement is again necessary, the durability of the culvert material, and the composition of the backfill in order to avoid obstructions to future tunnelling.

Regarding the restoration of the existing culverts, all reasonable options, such as culvert rehabilitation, open cut excavation replacement, various tunnelling techniques and even reducing the number of culvert crossings by combining upstream channels should be evaluated. Depending on the construction staging and traffic interruption constraints, and the hydraulic capacity, size and condition of existing and proposed culverts, the following options for restoring the culverts are recommended for consideration in priority order, generally from lower cost and risk to higher cost and risk:

- Rehabilitation by slip lining
- Replacement by open cut
- Replacement by tunnelling

To clarify, from a foundation engineering perspective, it is recommended

- 1st to rehabilitate by slip lining those culverts that can be rehabilitated
- 2nd to replace by open cut those remaining culverts that are reasonable to replace by open cut
- 3rd to replace by tunnelling any remaining culverts.

The work is required to comply with relevant legislation including the Occupational Health and Safety Act (OHSA) and with the relevant regulations including those for construction projects (O. Reg. 213/19) and confined space (O. Reg. 632/05).

The foundation frost penetration depth at the sites is 2.0 m according to OPSD 3090.101. The granular aggregate materials should conform to OPSS.PROV 1010.

All elevations in the report are expressed in metres. A list of standard specifications and draft non-standard special provisions (NSSP) referenced in this report is compiled in Appendix C.

### 3.2 Excavation

Excavation to the anticipated founding levels of the culverts is expected to extend through the surficial peat, fill and rockfill into/or through the native cohesive and cohesionless deposits. Boulders and cobbles are anticipated within the fill material and native soils. Subject to adequate groundwater control, excavation of the soils should be feasible using conventional equipment. All excavations should be conducted in accordance with OPSS 902.

According to the Occupational Health and Safety Act (Ontario Regulation 213/91) criteria, typically the in-situ firm to stiff cohesive soils and compact to loose non-cohesive soils are classified as Type 3 soils necessitating temporary cut slopes to be inclined no steeper than 1H:1V. The soft to very soft cohesive soils and very loose non-cohesive soils over the project limits are classified as Type 4 soils necessitating temporary cut slopes to be inclined no steeper than 3H:1V. Excavations in rock fill that are to be entered by workers may not be steeper than 1.25H:1V. Below the water table, cut slopes should be shaped no steeper than 3H:1V. Where composite soil types exist, the highest numbered soil type shall govern the excavation and slope geometry.

To maintain base stability at the subgrade level in the existing firm to soft soils, special treatment for the temporary cut slopes of the excavation should be applied during the replacement of the culverts.

The temporary cut slopes should be excavated at 1H:1V and partially excavated (benched) to ensure a maximum height of 1.5 m is maintained for the excavation cut slopes above the subgrade level adjacent to the culvert. The width of each bench should be equal to the total depth of the excavation lift. Heavy construction equipment should not be permitted on the benches adjacent to the culvert excavation.

It may be necessary to implement temporary support systems to provide roadway protection and to permit excavation and backfilling of trenches or excavations for the installation of the culverts and their associated appurtenances. The construction for temporary support system should conform to OPSS 404 and 539. A performance level of 2 for the protection system, according to OPSS 539, should be adopted to prevent excessive lateral and/or vertical movement of the existing embankment during construction. The contractor is responsible for the selection, detailed design and performance of the roadway protection scheme. The contractor should monitor the movement of the roadway protection system.

Where shallow excavation of bedrock may be required, mechanical means such as a large excavator equipped with a tiger-toothed bucket in conjunction with a jack-hammer or hoe ram is the preferred method of excavation to shallow depths in rock at foundation locations. Mass concrete could be employed to level minor variations in the bedrock surface.

The excavation width should be at least 1 m wider than the plan area of a culvert. Near vertical sidewalls may be utilised for excavations in bedrock. Examination of the sidewalls and removal of any loosened rock fragments should be carried out continually for the safety of workmen.

### 3.3 Erosion Control

The protective measures noted in the OPSD 800 series to deal with erosion (inlet/outlet treatment, headwalls, cut-off walls etc.) are considered to be appropriate. The backfill should comprise OPSS Granular A or Granular B Type II. The requirements of CHBDC clauses 1.9.5.6 and 1.9.11.6.5 should be applied where applicable.

Inlet and outlet protection in accordance with OPSS 511, OPSS. PROV 1004 and OPSD 810.010 is recommended to prevent erosion adjacent to the culvert as well as scour.

It is recommended that horizontal inlet cut-offs and rock protection and outlet erosion protection should be used instead of vertical cut-offs and structural head walls. In this case, the following recommendations apply:

- The length and width of horizontal cut-off aprons shall be a minimum of 2.0 m or twice the diameter of the culvert, whichever is less.
- The rock protection shall conform to OPSS 511 with a minimum dimension of 0.3 m and a minimum thickness of 0.5 m and extend to a minimum of 0.3 m above the culvert obvert.
- Clay seals at the inlet shall be in conformance with OPSS 1205 and extend over the area defined under rock protection.
- Drainage and/or filter blankets at the outlet shall extend over the area defined under rock protection and may consist of a natural filter consisting of a minimum thickness of 0.3 m of Granular A or non-woven Class II geotextile with an FOS of 75-150 $\mu$m according to OPSS 1860. The filter shall be placed below the rock protection to minimize the potential for erosion of fine particles from below the treatment.

Where earth slopes are inclined at 2.5H:1V or steeper, the permanent earth slopes should be protected with erosion control blankets. Also, sod (as per OPSS 803) shall be placed where it currently exists with a view to aesthetics. Alternatively rock fill embankments without erosion protection or a minimum 0.5m thickness of rock protection over earth fill may be applied for this purpose.

### 3.4 Groundwater Control

The extent of ground water control will depend on the depth of excavation below the ground water level and the construction technique. Groundwater control refers to both surface water management during and after construction and to dewatering during construction. This aspect is the responsibility of the contractor, but consideration could be given to either constructing in the dry, which would require dewatering or constructing in the wet, which would require construction below the groundwater level.

For construction in the dry, a dewatering system should be designed to maintain and control ground water at least 0.5 m below the excavation base level during construction. The dewatering methods should be the contractor's responsibility. However, the contractor could give consideration to conventional sump pumping techniques, which should be sufficient to allow for drainage of perched and/or surface water although perimeter drainage and/or well points may be required in areas of non-cohesive silts.

Another alternative would be to construct in the wet that involves to excavate and backfill without dewatering. In this case, fill placed below the water level would have to be Granular A material with a low fines content of less than 5% or Granular B Type II material. These materials may be selected depending on local availability and the groundwater level at the time of construction. Where possible the granular material may be placed to a level 0.2 to 0.3 m higher than the water level and compacted from the surface. The compacted material may then be excavated partially to the bedding course level of the culvert. Alternatively, 20 mm clear stone could be used provided that the stone is wrapped in a geotextile for separation from the native soil or rockfill to avoid loss of fines into the surrounding voids. The geotextile should conform to OPSS 1860. In addition, the clear stone should be sealed at the inlet with an impervious material such as Clay Seal per OPSS 1205 to prevent infiltration through the bedding instead of flow through the culvert and to prevent loss of fines in the subgrade material. Where the construction is carried out below the prevailing groundwater level without dewatering, prefabricated concrete box or pipe sections should be used as replacement culverts.

Reference is made to OPSS 517 and 518 which pertain to construction dewatering.

It will be necessary to implement measures to control seasonal ponded or surface water flow at the culvert locations. Conventional procedures such as dam and pump and/or diversion of the any overland flow streams should be sufficient. Groundwater levels are subject to seasonal fluctuations and precipitation patterns. Refer to Appendix C for a draft NSSP to advise the contractor of relevant considerations for surface water control and dewatering and that groundwater control of excavations is the contractor's responsibility.

Where the groundwater table is above the proposed subgrade level, cofferdams may be required for the installation of the culvert. The contractor is responsible for the selection, detailed design and performance of the cofferdams.

It is understood that a permit to take water is required by the Ministry of the Environment for water taking over 50,000 litres per day. Given the relatively low hydraulic conductivity of the cohesive native silty clay and clayey silt soils and that groundwater was typically measured below the anticipated base of the excavation, dewatering during construction is not anticipated to exceed 50,000 litres per day. The expected daily flows at each culvert location should be assessed by the hydraulic engineer.

It is recommended that the work be carried out during the dry months of June to September to minimise the amount of groundwater inflow to be handled and the volume of surface water, if any, to be diverted from the construction area.

All construction work should be carried out in accordance with the Occupational Health and Safety Act and with local/MTO regulations.

## 3.5 Option Specific Recommendations

### 3.5.1 Rehabilitation by Slip Lining

Rehabilitation does not remove the existing culvert material, but instead provides a new inner layer. When considering culvert rehabilitation, the required cross-sectional area needed for the new culvert and in turn the finished diameter of pipe to be installed by the construction method used need to be verified.

Culverts may be rehabilitated by slip lining. However, this method will reduce internal culvert diameter.

The new lining material should be selected based upon safety and life-cycle considerations. Installation of any lining system will decrease the internal diameter, thereby potentially reducing hydraulic capacity. Sometimes, the new smoother finish could maintain or slightly increase the hydraulic capacity. However, rehabilitation does not address or repair eroded bedding or backfill materials. The loss of this material could be a contributing cause of the culvert's failure.

Another problem with the rehabilitation approach is the construction sequence and timing. Upstream damming of streams would be required with by-pass pumping to manage the flow.

In some instances, rehabilitation is not a viable strategy. An existing culvert that is surrounded by unconsolidated or shifting bedding must be replaced. Lining will not mitigate the effects of groundwater eroding the soil along the outside of the host pipe. Many of the problems with a culvert, like squatting or sagging, will not be corrected by inserting a new lining.

If this option is selected, it is recommended that the Design Build process should be adopted to coordinate the design with the contractor's method and equipment. In the Design Build process, the Design Builder should provide recommendations for design and installation of grout to meet performance requirements.

### 3.5.2 Open Cut Excavation Method

Open cut replacement is possible at all culvert locations. However, the extent of excavation and the impact on traffic of an open cut operation would have to be factored into decisions. This method would provide known and controllable costs and time frames for the work, but open trenching of the existing highway may not be an attractive option in all cases due to the increasing amount of excavation required as fill heights increase, possible shoring requirements the related interruption of traffic. If this option is selected, staged construction/shoring and/or detouring would be required. This contract for this method could be DBB or DB as the construction methods would be conventional.

It is recommended that precast concrete box culverts or precast concrete pipe sections will be installed at the existing 22 culvert locations if this replacement method is deployed.

The following table summarizes the approximate invert and obvert levels of the existing culverts, the anticipated excavation depths to replace the culverts and the approximate depth of the soil cover above the culvert. The approximate invert and obvert levels with the anticipated up to excavation and approximate soil cover depths at each culvert were obtained from the profile drawings provided by AECOM via email dated April 1, 2014. The height of the embankment fill and native soil over the existing culverts ranged up to 8.8 m and the anticipated depth of excavation to replace the culverts ranged up to 10.5 m.

| PML FOUNDATION CULVERT ID | APPROXIMATE EXISTING CULVERT INVERT LEVEL | APPROXIMATE EXISTING CULVERT OBVERT LEVEL | ANTICIPATED DEPTH OF EXCAVATION (ELEVATION) | APPROXIMATE DEPTH OF SOIL COVER ABOVE CULVERT |
|---|---|---|---|---|
| **GEOGRAPHIC TOWNSHIP OF DENISON** | | | | |
| A 13+410 EBL | 250.8 to 255.9 | 251.5 to 256.6 | Up to 6.5 m (250.3 to 255.4) | Up to 5.3 |
| B 15+506 C/L | 259.0 to 260.7 | 259.8 to 261.5 | Up to 8.5 m (258.5 to 260.2) | Up to 7.5 |
| C1 16+110 WBL | 271.9 to 272.0 | 273.0 to 273.1 | Up to 5.0 m (271.4 to 271.5) | Up to 3.5 |
| C2 16+110 EBL | 272.2 to 272.4 | 273.3 to 273.5 | Up to 5.0 m (271.7 to 271.9) | Up to 3.1 |
| D 16+740 C/L | 257.4 to 260.6 | 258.4 to 261.5 | Up to 10.0 m (256.9 to 260.1) | Up to 8.5 |
| E 16+958 C/L | 255.7 to 256.9 | 256.6 to 257.8 | Up to 7.5 (255.2 to 256.4) | Up to 6.5 |
| **GEOGRAPHIC TOWNSHIP OF GRAHAM** | | | | |
| F 10+525 EBL | 244.2 to 246.0 | 245.3 to 247.1 | Up to 7.5 m (243.7 to 245.5) | Up to 5.5 m |
| G 10+685 EBL | 248.0 to 248.7 | 248.9 to 249.6 | Up to 7.0 m (247.5 to 248.2) | Up to 5.5 m |
| H 10+910 EBL | 255.8 to 255.9 | 256.7 to 256.8 | Up to 5.5 m (255.3 to 255.4) | Up to 4.3 m |
| I 12+273 C/L | 254.8 to 256.3 | 255.5 to 257.0 | Up to 10.0 m (254.3 to 255.8) | Up to 8.8 m |
| J1 12+620 WBL | 249.9 | 251.0 | Up to 5.5 m (249.4) | Up to 4.2 m |
| J2 12+630 EBL | 249.7 to 249.9 | 250.8 to 251.0 | Up to 5.0 m (249.2 to 249.4) | Up to 3.6 m |
| K 12+850 WBL | 249.9 to 250.2 | 250.6 to 250.9 | Up to 3.5 m (249.4 to 249.7) | Up to 2.5 m |
| L 17+894 C/L | 244.3 to 245.4 | 246.2 to 247.3 | Up to 8.5 m (243.8 to 244.9) | Up to 6.5 m |
| M 18+882 C/L | 246.7 to 247.3 | 248.2 to 248. | Up to 10.5 m (246.2 to 246.8) | Up to 8.4 m |
| N1 19+820 WBL | 256.5 to 256.6 | 257.8 to 257.9 | Up to 5.0 m (256.0 to 256.1) | Up to 3.2 m |
| N2 19+850 EBL | 256.3 | 257.6 | Up to 5.0 m (255.8 to 257.1) | Up to 3.4 m |

| PML FOUNDATION CULVERT ID | APPROXIMATE EXISTING CULVERT INVERT LEVEL | APPROXIMATE EXISTING CULVERT OBVERT LEVEL | ANTICIPATED DEPTH OF EXCAVATION (ELEVATION) | APPROXIMATE DEPTH OF SOIL COVER ABOVE CULVERT |
|---|---|---|---|---|
| **GEOGRAPHIC TOWNSHIP OF WATERS** | | | | |
| O<br>11+753 WBL | 262.4 to 263.9 | 263.3 to 264.8 | Up to 4.5 m<br>(261.9 to 263.4) | Up to 3.5 m |
| P<br>13+598 EBL | 245.5 to 246.2 | 246.4 to 247.1 | Up to 4.5 m<br>(245.0 to 245.7) | Up to 3.5 m |
| Q<br>14+943 WBL | 262.5 to 262.8 | 263.4 to 263.7 | Up to 7.0 m<br>(262.0 to 262.3) | Up to 5.5 m |
| R<br>15+687 C/L | 248.5 to 251.5 | 249.7 to 252.7 | Up to 9.0 m<br>(248.0 to 251.0) | Up to 7.5 m |
| S<br>16+125 EBL | 249.0 to 249.7 | 249.9 to 250.6 | Up to 8.5 m<br>(248.5 to 249.2) | Up to 7.5 m |

Note: * - For the anticipated depth of excavation, the subgrade level of the granular bedding is interpreted to be about 0.5 m below the existing invert levels, allowing for the thickness of the granular bedding and levelling courses.

** - Removal of clayey silt with organic and boulder inclusions is recommended.

### 3.5.3 Foundations

For the anticipated depth of excavation, the subgrade level of the granular bedding is about 0.5 m below the existing invert levels. This zone shall be backfilled with prescribed granular materials to all for the thickness of the granular bedding and levelling courses. Refer to the borehole logs identified for respective crossings for possible subsurface conditions at those culvert locations. It should be noted that the soil conditions may vary between and away from borehole locations.

#### *3.5.3.1 Culverts in Geographic Township of Denison*

Culvert A (D4) Station 13+410 EBL

Under the EBL embankment, loose to compact silt (borehole A-2) was encountered below the anticipated excavation. During augering, groundwater was observed at depths of 3.7 and 7.0 m (elevation 252.8 and 252.0) in boreholes A-1 and A-2, respectively. Upon completion of drilling, groundwater was measured at depths of 6.1 and 13.1 m (elevation 250.4 and 246.9) in boreholes A-1 and A-2 respectively.

The underlying soils below the culvert have been loaded with up to 5.3 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert B (D7) Station 15+506 C/L

The anticipated excavation depths at the culvert location varies up to 8.5 m. Under WBL shoulder embankment (borehole B-2A), compact sandy silt was encountered below the anticipated excavation elevation 259.2. In the median of the highway (borehole B-3), firm to stiff clayey silt fill was encountered. At the EBL shoulder embankment (borehole B-4) very stiff clayey silt was encountered below the anticipated excavation depth of 258.5. Open water was encountered in boreholes B-1 and B-5 during and upon completion of augering at ground surface (elevation 259.4 and 260.2). The remaining boreholes were charged with water during drilling.

The subgrade soils below the culvert have been loaded with up to 7.5 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert C1 (D9) Station 16+110 WBL

Firm to stiff silty clay was encountered (borehole C1-2) under the WBL shoulder below the anticipated excavation depth. During augering, surficial water (elevation 271.9) was observed in borehole C1-1 and at 0.3 m (elevation 272.2) in borehole C1-3. Upon completion of augering, water was observed at ground surface (elevation 272.5) in borehole C1-3.

The underlying soils below the culvert have been loaded with some 3.3 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert C2 (D9) Station 16+110 EBL

Under the EBL shoulder, very stiff to stiff clayey silt (fill) was encountered (borehole C2-1) below the anticipated excavation elevation 271.7. Groundwater was encountered in boreholes C2-2 and C1-3 at 2.5 and 0.3 m (elevation 269.9 and 272.2), respectively. Upon completion of drilling, groundwater was measured at ground surface (elevation 272.5) in borehole C1-3.

The underlying soils below the culvert have been loaded with some 3.2 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying clayey deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert D (D11) Station 16+740 C/L

Under the WBL (boreholes D-2 and D-3), probable bedrock was encountered below the culvert. Under the EBL (borehole D-4) met refusal on probable boulder at elevation 261.8 above the culvert invert. Groundwater was observed at 0.6 m (elevation 261.3) in borehole D-1. Borehole D-2 and D-3 were charged with water during drilling.

It is anticipated that two separate culverts will be placed under WBL and EBL. The options of using a catch basin or two separate culverts could be considered. A catch basin would be utilized to join the two culvert ends in the median of the highway. No settlement is anticipated from the probable bedrock encountered in the borehole locations.

It is considered that cambering will not be required at the culvert location.

Culvert E (D12) Station 16+958 C/L

Under the WBL embankment (borehole E-2), very stiff to firm silty clay was encountered below the anticipated excavation depth. In the highway median (borehole E-3) stiff to firm silty clay was encountered. Under the EBL embankment (borehole E-4) very stiff to firm silty clay is anticipated below the excavation depth. Water was observed below ice and snow cover in borehole E-1 at 0.3 m (elevation 256.9) and in borehole E-5 at 0.6 m (elevation 256.4) during and after completion of augering. Groundwater was observed in boreholes E-2 and E-3 at 8.7 m (elevation 255.1) and 3.7 m (elevation 254.7), respectively, during augering only.

It is anticipated that two separate culverts will be placed under WBL and EBL. The options of using a catch basin or two separate culverts could be considered. A catch basin would be utilized to join the two culvert ends in the median of the highway. The underlying soils below the culvert have been loaded with up to 6.5 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying deposits after the culverts are installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

*3.5.3.2 Culverts in Geographic Township of Graham*

Culvert F (G3) Station 10+525 EBL

Under the EBL embankment (borehole F-2), dense silt was encountered below the anticipated excavation depth. Boreholes F-1 and F-2 were dry on completion of drilling. The groundwater level was not established in borehole F-3 because the borehole was charged with water during drilling.

The underlying soils below the culvert have been loaded with up to 5.5 m thick overlying fill soils for a substantial period of time. Only negligible settlement is anticipated from the underlying clayey deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert G (G4) Station 10+685 EBL

Under the EBL embankment (borehole G-2) and south end of the culvert (borehole G-3), rockfill and boulders were encountered below the anticipated excavation depth. The boreholes were dry on completion of drilling.

The underlying soils below the culvert have been loaded with up to 5.5 m thick overlying soil cover for a substantial period of time. Only negligible settlement is anticipated from the underlying deposits.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert H (G5) Station 10+910 EBL

Under the EBL shoulder embankment (borehole H-2) compact to dense silty sand was encountered below the anticipated excavation depth. Borehole H-1 was dry on completion of drilling. Groundwater was observed at 5.8 m (elevation 255.2) in borehole H-2 on completion of drilling. In borehole H-3, groundwater was not established because the borehole was charged with water during drilling.

The underlying soils below the culvert have been loaded with up to 4.3 m thick overlying soil cover for a substantial period of time. Only negligible settlement is anticipated from the underlying deposits.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert I (G7) Station 12+273 C/L

Under the WBL embankment (borehole I-2), rockfill was encountered below the anticipated excavation depth. In the highway median (borehole I-3), firm to stiff silty clay fill was encountered. Under the EBL embankment (borehole I-4) stiff to firm silty clay was encountered below the anticipated excavation depth. During and upon completion of augering, groundwater was observed at 6.1 m (elevation 249.9) in borehole I-1. No water was observed in borehole I-5. The remaining boreholes were charged with water during drilling.

It is anticipated that two separate culverts will be placed under WBL and EBL. The options of using a catch basin or two separate culverts could be considered. A catch basin would be utilized to join the two culvert ends in the median of the highway. The underlying soils below the culvert have been loaded with up to 8.8 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying clayey deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert J1 (G8) Station 12+620 WBL

In borehole J1-2, rockfill was encountered below the anticipated excavation depth. During augering, groundwater was observed at 1.5 and 1.2 m (elevations 249.8 and 249.5), respectively, in boreholes J1-1 and J1-3. Upon completion of drilling, groundwater was measured at 0.6 and 1.2 m (elevations 250.7 and 249.5), respectively, in boreholes J1-1 and J1-3.

The underlying soils below the culvert have been loaded with up to 4.2 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying clayey deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert J2 (G9) Station 12+630 EBL

Under the EBL embankment (borehole J2-1) rockfill was encountered below the anticipated excavation depth. During augering and upon completion of augering, groundwater was observed at 1.2 m (elevation 249.5) in borehole J1-3. Groundwater levels in boreholes J2-1 and J2-2 were not determined because the boreholes were charged with water during drilling.

The underlying soils below the culvert have been loaded with up to 3.6 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying clayey deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert K (G10) Station 12+850 WBL

Under the WBL embankment (borehole K-2) rockfill was encountered below the anticipated excavation depth. During and upon completion of augering, groundwater was observed at 7.3 m (elevation 243.4) in borehole K-3. In borehole K-1, groundwater was not encountered during and upon completion of augering. Groundwater level was not established in borehole K-2 because the borehole was charged with water during augering.

The underlying soils below the culvert have been loaded with up to 2.5 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying clayey deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert L (G25) Station 17+894 C/L

Under the WBL embankment (borehole L-2A), stiff clayey silt was encountered below the anticipated excavation elevation 244.1. In the median (borehole L-3), subsurface soil below the invert level was not established because refusal on probable rockfill was met at elevation 249.5. Under the EBL (borehole L-4), very stiff silty clay was encountered at the anticipated excavation elevation 243.7. Groundwater was observed during drilling in boreholes L-1, L-4 and L-5 at 2.6, 8.8 and 6.2 m, (elevation 243.2, 243.0 and 238.4), respectively. In boreholes L-2, L-2A and L-3, boreholes were charged with water during augering.

It is anticipated that two separate culverts will be placed under WBL and EBL. The options of using a catch basin or two separate culverts could be considered. A catch basin would be utilized to join the two culvert ends in the median of the highway. The underlying soils below the culvert have been loaded with up to 6.5 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying clayey deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert M (G26) Station 18+882 C/L

Under the WBL embankment (borehole M-2) rockfill was encountered below the anticipated excavation depth. At the median (borehole M-3), existing subgrade was not established below invert level because the borehole met refusal on probable rockfill at elevation 250.1. Under the EBL embankment (borehole M-4) very stiff to stiff silty clay was encountered below the anticipated excavation elevation 246.3. In boreholes M-1, M-2A, M-4 and M-5, groundwater was not established due boreholes charged with water during augering. Groundwater was not encountered during and upon completion of augering in boreholes M-2 and M-3.

It is anticipated that two separate culverts will be placed under WBL and EBL. The options of using a catch basin or two separate culverts could be considered. A catch basin would be utilized to join the two culvert ends in the median of the highway. The underlying soils below the culvert have been loaded with up to 8.4 m thick overlying rockfill for a substantial period of time. Only negligible settlement is anticipated from the underlying soil deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert N1 (G30) Station 19+820 WBL

Under the WBL embankment (borehole N1-2) stiff silty clay was encountered below the anticipated excavation depth. In borehole N1-2, groundwater was observed at 5.6 m (elevation 255.3) during augering. In borehole N1-3, groundwater was observed at 1.2 m (elevation 256.0) and upon completion of augering groundwater was observed at ground surface (elevation 257.2).

The underlying clayey soils below the culvert have been loaded with up to 3.2 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying clayey deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert N2 (G31) Station 19+850 EBL

Under the EBL embankment (borehole N2-1) firm clayey silt was encountered below the anticipated excavation depth. In borehole N1-3, groundwater was observed at 1.2 m (elevation 256.0) and upon completion of augering groundwater was observed at ground surface (elevation 257.2). Groundwater levels in boreholes N2-1 and N2-2 were not determined because the boreholes were charged with water during drilling.

The underlying soils below the culvert have been loaded with up to 3.4 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying clayey deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

*3.5.3.3 Culverts in Geographic Township of Waters*

Culvert O (W4) Station 11+753 WBL

Under the WBL embankment (borehole O-2) firm to stiff clayey silt was encountered below the anticipated excavation depth. Open water was encountered at elevation 263.4 below ice in borehole O-1 during and upon completion of augering. In borehole O-2, groundwater was observed in borehole O-2 at 7.3 m (elevation 259.8) during augering.

The underlying soils below the culvert have been loaded with up to 3.5 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying clayey deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert P (W14) Station 13+598 EBL

Under the EBL embankment (borehole P-2) stiff to very soft clayey silt was encountered below the anticipated excavation depth. Surficial water (elevation 245.8) was encountered in borehole P-3 during and upon completion of augering. In borehole P-1, groundwater was encountered at 0.3 m (elevation 245.6) during augering and at surface (elevation 245.9) upon completion of augering. During augering, groundwater was observed at 7.3 m (elevation 241.4) in borehole P-2; however, groundwater was not observed upon completion of augering.

The underlying soils below the culvert have been loaded with up to 3.5 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert Q (W17) Station 14+943 WBL

[The borehole elevations were approximated from the profile drawing 14+953.474.dwg by exp Geomatics received via email dated February 26, 2014.]

Under the EBL embankment (borehole Q-2) stiff clayey silt was encountered below the anticipated excavation depth. In borehole Q-2, groundwater was encountered at 13.7 m (elevation 256.1) during augering and at surface (elevation 269.8) after completion of augering, indicating an artesian condition.

The underlying soils below the culvert have been loaded with up to 5.5 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert R (W25) Station 15+687 C/L

Under the WBL embankment (borehole R-2) compact silt was encountered below the anticipated excavation depth. In the median of the highway (borehole R-3) the excavation should extend to below the fill to compact sandy silt at 7.0 m (elevation 250.0). Under the EBL embankment (borehole R-4), stiff clayey silt fill was encountered below the anticipated excavation depth. During augering, groundwater was encountered at 1.2 to 6.9 m (elevation 250.1 to 251.3) in boreholes R1 to R4. Upon completion of augering, groundwater was measured at 6.1 and 6.4 m (elevation 251.7 and 250.6), respectively, in boreholes R-2 and R-3.

It is anticipated that two separate culverts will be placed under WBL and EBL. The options of using a catch basin or two separate culverts could be considered. A catch basin would be utilized to join the two culvert ends in the median of the highway. The underlying soils below the culvert have been loaded with up to 7.5 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

Culvert S (W26) Station 16+125 EBL

Under the EBL embankment (boreholes S-2 and S-3) compact to loose silt to firm to stiff silty clay was encountered below the anticipated excavation depth. During augering, groundwater was encountered in boreholes S-1, S-2 and S-3 at 2.1 to 7.8 m, elevation 248.2 to 249.6. Surficial water was observed in borehole S-4 during augering at elevation 249.8. Upon completion of augering, groundwater was observed at surface (elevation 250.3) in borehole S-1 and in the remaining three boreholes groundwater was observed at 2.3 to 7.9 m, elevation 247.5 to 248.8.

The underlying soils below the culvert have been loaded with up to 7.5 m thick overlying soils for a substantial period of time. Only negligible settlement is anticipated from the underlying deposits after the culvert is installed.

Because of the anticipated negligible and uniform settlement, it is considered that cambering will not be required at the culvert location.

### 3.5.4 Subgrade Preparation

Preparation of the subgrade for construction of the culverts should be performed and monitored in accordance with OPSS 902 and SP 902S01.

For culvert replacement, it is recommended to provide a minimum 500 mm of combined granular and levelling course with a minimum 300 mm thick granular bedding below the culvert. The bedding material should be composed of Granular A or Granular B Type II and be compacted in conformance with OPSS 501 (Method A). Alternatively, clear stone can be utilized as discussed in Section 3.4.

Peat and any other deleterious soils revealed during the subgrade preparation beneath the plan limits of the culvert excavation should be excavated to an additional maximum depth of 0.5 m prior to placement of the granular base below the box culvert. The excavation shall be replaced with compacted Granular A or Granular B Type II.

Subgrade preparation, cover, backfill and frost treatment for the proposed culverts should be carried out in accordance with OPSS 422, SP 422S01 and MTOD 803.021. A foundation frost penetration depth in the area is 2.0 m according to OPSD 3090.101.

Rock fill should be placed in accordance with OPSS.PROV 206 and SP 206S03. This is particularly important above the water level within the zone of influence of the culvert, defined by an imaginary line inclined downwards at 2H:1V from a point located at the invert level 1 m beyond the edge of the culvert.

For culverts placed within rock fills, the granular bedding material and rock fill material should be separated by a geosynthetic filter fabric to prevent loss of the granular materials into the voids of the rock fill. The rock fill surface should be chinked in accordance with the requirements of OPSS.PROV 206 and SP 206S03, prior to placing the geotextile. The filter fabric should conform to OPSS 1860 and comprise a Class II non-woven geotextile with a filtration opening size (FOS) of 105 to 210 μm. The filter fabric should be placed beneath the bedding and extend up each side and to the top of the bedding and/or granular cover material.

Where very loose to loose cohesionless soils and soft to firm clayey soils are anticipated at subgrade level and above the prevailing groundwater level, the subgrade should be immediately covered with a layer of biaxial geogrid (25 by 35 mm max. aperture 1.2 to 2.0 kN/m min. peak tensile strength) and backfilled with the select bedding material. It is recommended to provide 300 mm of granular bedding above the geogrid. The bedding material should be composed of 300 mm of Granular A, compacted as detailed above. Below the prevailing ground water level, geogrid is not applicable and granular fill or rock fill may be end dumped directly into the excavation.

If rock fill is placed as a backfill material below the culverts over loose cohesionless soils or firm clayey soils, settlements of the culverts may exceed the 25 mm compression of the founding medium normally allowed for by SLS resistance values due to the rock fill sinking into underlying softer materials and fine soils infiltrating the rock fill voids. A minimum of 300 mm thick granular bedding material shall be placed over the rock fill immediately beneath the culvert. The granular backfill shall be shaped to conform to the shape of the invert of the culvert to reduce the structural distress that may result from the differential settlement as well as to minimise “low areas” in the culvert when settlement is complete.

Where the culverts may be constructed with a bedrock base, mass concrete could be employed to raise the subgrade to the design level of the side walls. Mass concrete could also be placed to provide a level founding surface for the wing wall or head wall footings, if required. Alternatively, the rock surface could be “stepped” to follow variations in the bedrock surface elevation thereby creating a level subgrade by a combination of rock excavation and placement of mass concrete.

The need to expand the plan area at the base of the mass concrete to provide for stress distribution (2V:1H), place reinforcing steel in the mass concrete and/or use high strength concrete to prevent overstressing which will be dictated by the actual thickness of the mass concrete and structural design considerations.

Subject to these comments, the bearing resistance provided for footings bearing on bedrock is considered to be appropriate for mass concrete with an unconfined compressive strength of at least 35 MPa.

### 3.5.5 Culvert Backfill

Backfill adjacent to the box culverts should be placed in accordance with 803.010, OPSS 422 and SP 422S01.

Backfill should be brought up simultaneously on each side of the box culvert. The operation of heavy equipment within 0.5 times the height of the box culvert (each side) should be restricted to minimise the potential for movement and/or damage of the box culvert due to the lateral earth pressure induced by compaction. Refer to SP 105S10 for additional comments.

The box culverts must be designed to resist the unbalanced lateral earth pressure and compaction pressure exerted by the backfill adjacent to the box culvert walls.

The geotechnical bearing resistance will not be an issue since the embankments have experienced the existing embankment load for a substantial period of time and there will be no increase in loading on the subgrade. The following typical bearing resistances (without consideration for the pre-loading effect of the embankments) may be assumed for design purposes.

Factored Geotechnical Bearing Resistance at ULS = 250 kPa

Geotechnical Reaction at SLS = 150 kPa

The Geotechnical Reaction at SLS applies to a tolerable settlement of 25 mm if the foundation subgrade is not disturbed, for example by the removal of the existing culvert. In cases where the ground is disturbed by construction activities thicker layers of bedding material should be placed over the subgrade to obtain uniform founding conditions.

The lateral earth and water pressure, p (kPa), will only be applicable for retaining structures such as head walls and wing walls and should be computed using the equivalent fluid pressures presented in Section 6.9 of the CHBDC or employing the following equation assuming a triangular pressure distribution:

$$P = K(\gamma h_1 + \gamma' h_2 + q) + \gamma_w h_2 + Cp + Cs$$

where $K$ = lateral earth pressure coefficient

$\gamma$ = unit weight of free draining granular material above the design water level ($kN/m^3$)
$\gamma'$ = unit weight of backfill submerged below the design water level ($kN/m^3$)
$h_1$ = depth below final grade (m), above the design water level
$h_2$ = depth below the design water level (m)
q = any surcharge load ($kN/m^2$)
$\gamma_w$ = unit weight of water equal to 9.8 $kN/m^3$
Cp = compaction pressure (refer to clause 6.9.3 of CHBDC)
Cs = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
where Ø = angle of internal friction of retained soil (35º for Granular A)
$\delta$ = angle of friction between soil and wall (23.5º for Granular A)

The following parameters are recommended for design:

| PARAMETER | GRANULAR A, GRANULAR B TYPE II | ROCK FILL |
|---|---|---|
| Angle of Internal Friction, degrees | 35 | 42 |
| Unit Weight, $kN/m^3$ | 22.8 | 18.0 |
| Active Earth Pressure Coefficient ($K_a$) | 0.27 | 0.20 |
| At-Rest Earth Pressure Coefficient ($K_o$) | 0.43 | 0.33 |
| Passive Earth Pressure Coefficient ($K_p$) | 3.69 | 5.04 |

The design should consider both the maximum water level and the stabilised groundwater level condition. The groundwater conditions varied at each culvert location. Seasonal perched water should be anticipated above the silty clay or clayey silt deposits at all of the culvert locations.

The coefficient of earth pressure at rest should be employed to design rigid and unyielding walls and the active earth pressure coefficient for unrestrained structures. Concrete culverts are considered to be constrained.

3.5.6 Seismic Site Coefficient

The seismic site coefficient for the culvert sites is 1.0 –Type I soil profile as per clause 4.4.6 of the CHBDC.

### 3.5.7 Replacement by Tunnelling

Culvert replacement using trenchless methods is another option, but carries risks due to uncertainty with ground conditions and obstructions, especially in mixed fill and rock fill embankments. Refer to the draft NSSP - Obstructions during Tunneling (Addition to OPSS 490) in Appendix C.

Following are issues related to procurement of services in projects involving tunnelling through mixed fill and rock fill embankments. The information is based on the following reference:

- Dots Oyenuga, Ph.D., P.E. July 2004. FHWA Road Tunnel Design Guidelines. Report Number FHWA-IF-05-023

Without equitable risk sharing, potential financial benefits of a competent design process cannot be realized. Hence there is need for additional allowance for increased cost during construction. Reasonable contingencies should be built into the total project cost estimate. It is suggested that the following contingencies be included:

- A construction contingency for cost growth during construction;
- A design contingency based on different levels of design completion;
- An overall Management contingency for third-party and other unanticipated changes;

A Value Analysis should be performed on the project to determine the most economical and advantageous way of packaging the contracts for advertisement.

General descriptions of tunnelling methods are presented in Appendix A.

The following table summarizes the advantages and disadvantages of the tunnelling methods described.

| TUNNELLING METHOD | ADVANTAGES | DISADVANTAGES |
|---|---|---|
| Steerable Jack and Bore | ▪ Contractor availability<br>▪ Grade control is available for alignment corrections<br>▪ Typically least costly method<br>▪ Can accommodate variable soils (sand layers in the onsite till) without major tooling adjustments<br>▪ Small staging areas compared to HDD<br>▪ Minimal tunnel diameter of 610 mm | ▪ Ground water control may be required for the bore and staging pits<br>▪ Elevated potential for ground subsidence if adequate ground water control is not achieved<br>▪ Recommended for drier seasons<br>▪ Minor residual space may be present surrounding liner exterior, which could require grouting<br>▪ Once operation is started it should continue without major stoppage until completion to mitigate potential for sloughing of face and void formation<br>▪ Not suitable for rock fill |
| Pipe Ramming | ▪ Low risk for loss of ground from over augering / collapses at the bore<br>▪ Low sensitivity to ground water seepage compared to jack and bore<br>▪ Can accommodate variable soils (sand layers, cobbles in the onsite till) without major tooling adjustments<br>▪ Small staging areas compared to HDD<br>▪ Has been reported to be effective through rock fill with some reservations | ▪ High ramming resistance required for liner penetration in stiff clayey silt till overburden<br>▪ Thicker steel needed to sustain ramming stresses<br>▪ Poor grade control compared to jack-and-bore, micro-tunneling and HDD<br>▪ Ground water control may be required for staging works<br>▪ More costly than jack and bore<br>▪ May require encroachment into right of way to maintain grade control<br>▪ Grades cannot be corrected once installation has started |

| TUNNELLING METHOD | ADVANTAGES | DISADVANTAGES |
|---|---|---|
| Horizontal Directional Drilling | ▪ Does not require deep staging pits<br>▪ Minimal ground water control required during drilling<br>▪ No wet season restrictions | ▪ Largest tunnel diameter, 900 mm envisaged<br>▪ Requires long slot trench and layout area, likely extending beyond the right of way<br>▪ Potential for inadvertent drilling fluid returns<br>▪ Larger HDD equipment may be required given 900 mm tunnel diameter and the nature of the till deposits<br>▪ Requires drilling fluid to maintain the bore which could allow subsidence<br>▪ May have poor grade control in gravelly soils and very loose or soft material.<br>▪ Potential for oval tunnel cross section<br>▪ Not suitable for rock fill due to high potential to lose drilling fluid |
| Micro-tunnelling | ▪ May not require ground water lowering for the tunnel<br>▪ Machine can be designed to be able to counter- balance earth and water pressures in a controlled manner, thereby reducing the risk of ground losses during tunnelling<br>▪ Good grade control<br>▪ Can be effective in rock fill if pregrouting binds the rock fill | ▪ Limited contractor availability<br>▪ Cost effectiveness depends on availability of existing adequate tunnel boring machines<br>▪ Ground water control is required for staging pits<br>▪ Typically more costly than jack-and-bore, or pipe-ramming |
| Conventional Mining | ▪ Capable of advancing tunnels in through soil embankments, mixed embankments and rock fill embankments | ▪ High cost<br>▪ Larger tunnel diameter required |

The selection of tunneling methods at this site should be the responsibility of the design builder. However, it is anticipated that pipe ramming may be the preferred method of design builders.

#### *3.5.7.1 Replacement Culvert Diameter*

In trenchless techniques through mixed ground and rock fill, the diameter of the new pipe must be large enough to allow personnel entry to physically excavate or remove the existing culvert and any obstructions such as rock fill. Space may also be needed for a rail line to move equipment, supplies, spoils, debris, and personnel. Ventilation lines may be needed to feed fresh air for the workers. Typically, tunnels and jacked pipe need to be 1500+ mm or larger to allow effective personnel entry and working space.

#### *3.5.7.2 Monitoring*

Monitoring is required as a strategy to mitigate the risks and consequences of settlement or other ground movements at the road surface resulting from the tunnelling operation. The design builder for the tunneling work shall comply with the guidelines for design and monitoring specified in Appendix B.

#### *3.5.7.3 Staging Excavations*

It is anticipated that open cut excavations will be used to construct staging areas for tunnelling operations. Reference is made to OPSS 201, 490 and 801 for specifications associated with site preparation.

The replacement culvert would be constructed by either consuming or overpiping the existing pipe with a new culvert or installing a new culvert parallel to the existing culvert and abandoning the existing culvert by plugging the upstream end and filling the opening with cementitious grout. Overpiping would probably reduce the risk somewhat of encountering rock fill and other obstructions during the tunnelling operation. However, it would require damming of the watercourse upstream and piping/pumping to downstream of the culvert. Constructing parallel barrels would present a higher risk of encountering rock fill, but would permit flow through the existing culvert while the new culvert was being installed.

#### *3.5.7.4 Overpiping*

In this approach, a new culvert is installed completely around the old culvert. The advantage is that the culvert's cross sectional area is increased, increasing the culvert's flow capacity. The entire old culvert is removed, and a portion of the bedding materials can be removed, providing a better soil-structure interaction for the new culvert. The size of the installation must consider how the space inside the pipe will be adjusted to match the required grade of the culvert. If the culvert has flowing water, the water can be redirected into a new bypass pipeline and allowed to pass through the existing culvert during construction, if there is room. This method also allows the work to be completed at least partially through presumably engineered backfill that may have a lower risk of containing rock fill or other obstructions.

In the consumption/overpiping approach, if the embankment material is suitable, the work can be completed using pipe ramming, tunneling, or pipe jacking.

#### *3.5.7.5 Parallel Barrel*

The second replacement method is the building of a new barrel parallel to the old culvert. The advantage of this approach is that the culvert's effective cross-sectional area can be increased as needed to meet the flow capacity requirements. The invert of the new barrel can be set to the design profile, so a smaller effective overall diameter is needed with this approach. The old culvert remains in service during construction. Once the second barrel is in place, the old culvert can be replaced by the consumption method, repaired, rehabilitated, or abandoned.

If the stream has flowing water, the parallel approach allows the new barrel to be constructed while the water flows through the original culvert. This is an important benefit during construction. When the new barrel is completed, the water course can be redirected into the new barrel to allow consumption, rehabilitation work or abandonment to occur in the old culvert. In the parallel barrel approach, if the embankment material is suitable and does not contain rock fill or obstructions, the work can be completed using pipe ramming, tunneling, and pipe jacking. The parallel barrel can also be installed with auger boring and the pilot tube method.

##### *3.5.7.6 Pipe Material Selection*

Pipe ramming, tunneling, or pipe jacking permits removal of the existing culvert and allows the substitution of other materials for the replacement culvert. Material selection is important for several reasons, including life cycle and safety. The selected or preferred culvert material must also be compatible with the contractor’s construction method.

#### 3.5.8 Culvert Replacement Design

The design for culvert replacement using pipe ramming, tunneling, or pipe jacking can include size for size replacement if constructed in parallel barrel, but in most cases upsizing is done to increase the cross sectional area for various reasons.

In the overpiping approach, the size will always be larger to envelop the existing culvert. The new pipe diameter must also include upsizing to factor in any squatting that has occurred and any sagging. The final diameter of the new pipe will also consider the position of the new invert relative to the original invert elevation.

Following are additional factors for consideration in evaluating the tunnelling option:

- A Design Build contract is recommended. It is important to have the tunnelling contractor on board at the design stage to coordinate his specialized expertise into the design and into the selection of his methods and equipment.
- Tunnelling contractors should be prequalified. This reduces the risk of construction problems in the tunnelling operation. Based on past MTO experience and experience reported by other jurisdictions, the tunnelling operation is heavily influenced by the contractor’s equipment and expertise level. Tunnelling contractor prequalification has been required by MTO on previous complex tunnelling projects.
- There should be a prebid meeting with mandatory attendance by specialist tunnelling contractors intending to bid on the work. This would reduce the risk of construction problems stemming from misunderstanding of owner expectations and has been required at previous complex tunnelling projects.

- It is recommended that test pits be excavated during the tendering stage so contractors bidding on the project can evaluate the soil and ground water conditions to be encountered and to assess the dewatering requirements.

- Tunnelling may have to be limited to a limited number of specific diameters due to the availability of equipment and the practical constraint that tunnelling may require a minimum tunnel diameter to permit personnel access to remove obstructions that is significantly larger than the actual hydraulic requirement for the culvert diameter. The minimum diameter that permits personnel entry is expected to be in the order of 1.2m to 1.5m.

- There is an advantage to bundling the contracting of multiple culverts in order to realize economies of scale and reduce unit equipment mobilization costs.

- There will be high costs and significant risks of cost overruns and delays for replacement of culverts by tunnelling techniques in mixed material and rock fill embankments. In this case, simply passing along these uncertainties of differing ground conditions along to the construction contractor may not be effective in achieving the best combination of quality, cost and schedule.

- As an illustration of the cost for tunnelling, following is a table summarizing projected minimum costs for replacement of all 22 culverts by pipe ramming.

<table>
<tr><th>DIAMETER (mm)</th><th>SUM OF LENGTHS (m)</th><th>PROJECTED TUNNELLING METHOD</th><th>PROJECTED TUNNELLING DIAMETER (mm)</th><th>COST PER METRE</th><th>TUNNELING COST</th></tr>
<tr><td>760</td><td>280.5</td><td rowspan="4">Pipe Ramming<br>1200 mm diameter Overpiping</td><td rowspan="4">1200</td><td rowspan="4">$5000/m plus contingency</td><td rowspan="4">$4.1M</td></tr>
<tr><td>780</td><td>23.0</td></tr>
<tr><td>910</td><td>376.3</td></tr>
<tr><td>1070</td><td>141.0</td></tr>
<tr><td>1220</td><td>180.8</td><td rowspan="3">Pipe Ramming<br>2100 mm diameter Overpiping</td><td rowspan="3">2100</td><td rowspan="3">$8000/m plus contingency</td><td rowspan="3">$2.9M</td></tr>
<tr><td>1520</td><td>81.0</td></tr>
<tr><td>1830</td><td>95.0</td></tr>
</table>

- The experience of MTO and other jurisdictions is that projected costs for tunnelling operations should carry an additional significant contingency in the order of 30% or more. Hence the total cost for pipe ramming all culverts could be in the order of $9M or more.

- In order to minimize damaging costs including those from delays, consideration should be given to specifying an efficient mechanism for resolution of construction problems and disputes in the contract documents that could include establishing a base line report defining the bidding assumptions and indication of the contractor's unit pricing and also identify an individual or tribunal contract dispute resolution authority.

- The MTO guidelines for tunnelling in Appendix B should be included in the Contract Documents. Included in these guidelines are minimum foundation investigation requirements and monitoring as well as targets for settlement performance of the road surface.

Reference is also given to Ontario Provincial Standard Specifications OPSS 415, Construction Specifications for Tunnelling.

## 4. CLOSURE

This report was prepared by Mr. D. Dundas, P.Eng. and Mr. N. Rahman, P.Eng., and reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact. Mr. C. M. P. Nascimento, P. Eng., Project Manager conducted an independent review of the report.

Yours very truly

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# APPENDIX A

Overview of Tunneling Methods

## <u>Overview of Tunneling Methods</u>

<u>Jack-and-bore</u>

Jack-and-bore involves the simultaneous advancement of a continuous flight auger and steel casing pipe. The auger flight system generally comprises the auger flight and an auger head designed for the ground condition. Jack-and-bore is a common method of trenchless installation in soil conditions due to its relatively low cost.

With a jack-and-bore installation there is a potential for loss of ground from excessive excavation (over augering) which could cause a collapse of the bore or void formation between the tunnel bore and the lining/casing.

It is the responsibility of the contractor to assure that potential loss of ground is minimized and any excessive movements and settlements resulting from the jack-and-bore operations are to be dealt with immediately. A suitable face pressure or soil plug should be maintained to minimize loss of ground during advance. Any over-cut during augering and pipe advance which could potentially create soil disturbance, space or void outside the pipe should be grouted or filled to avoid potential ground movements.

The staging areas for jack-and-bore include a jacking pit to accommodate the boring equipment and assembly of the pipe sections, and a receiving pit for recovery of the boring head. The staging areas for jack-and-bore include a jacking pit to accommodate the boring equipment and assembly of the pipe sections, and a receiving pit for recovery of the boring head. It is understood that the jacking pit would have to be about 16.4 m by 8.2 m in area and the receiving pit would have to be about 4.0 m by 1.5 m in plan area.

Perched ground water could hinder or prevent a jack-and-bore installation. In wet soils there is potential for ground surface subsidence due to running of wet soil into the bore, which could result in voids. Overall, the ground water / perched levels at the site must be below the tunnelling depths and thus may require ground water control measures. Increased surface water flow through the culvert and elevated ground water should be expected during the wetter Spring and Fall seasons. Therefore, it is recommended that a jack-and-bore crossing be under taken during the drier Summer months. However, depending on the contractor's method and experience, Winter excavation and construction may be considered to take advantage of frozen soil conditions. In general, jack-and-bore methods are well suited for installations where more precise grade control of the bore is required. However, the presence of cobbles and / or boulders may increase the risk for alignment deviations to occur. A significant disadvantage of the traditional conventional jack-and-bore method is that the alignment cannot be corrected during pipe advancing. However this is significantly improved with the proposed steerable jack-and-bore system envisaged for this assignment.

Pipe Ramming

Pipe ramming installation is analogous to driving an open ended steel tube pile horizontally. Impact forces from a percussive hammer are used to advance a conduit pipe from an entry pit to a receiving pit. During the advance, most of the soil being penetrated fills the conduit rather than being excavated. The rammed conduit is terminated in a receiving pit at which point the soil contained in the pipe is removed. A steel conduit is used for pipe ramming installation but the wall thickness would have to be more substantial due to the stresses imparted on the pipe during installation, and should conform to the applicable OPSS, CSA and ASTM standards.

Unlike jack-and-bore pipe ramming would be possible through wet soils without dewatering along the tunnel route. Nevertheless, dewatering measures will be required at the entry and receiving pits. Since the tunnelling depth is very close to the ground surface in the ditch areas, the force generated by the hammering operations will increase the risk for soil disturbance. The thin depth of cover in the ditch areas may also result in deflection of the casing during the start of the installation, the pipe may climb upward. Ideally, the pipe ramming sending pit should be located as close to the road embankment as possible to prevent possible grade deviations before the tunnel reaches the highway embankment. Pipe ramming does not allow for alignment corrections during installation.

Pipe ramming can be conducted through soils containing occasional cobbles and boulders, provided these large soil constituents are smaller than the casing. If it becomes essential to clean out the liner prior to completion, a soil plug should be maintained to mitigate against tunnel face failure and void formation. There has been reported success using this method through rock fill embankments. However this method entails risks from obstructions and the tunnel size has to be sufficient to permit personnel entry into the liner if it becomes necessary to remove obstructions.

Horizontal Directional Drilling

HDD involves the boring and enlargement of an uncased near horizontal tunnel which is kept open through use of drilling fluids. Upon completion of boring a conduit pipe is pulled though the bore. The process starts by advancing a relatively small diameter hole along the proposed path. During the pilot bore the cutter head at the lead of the drill string is steered by the drill rig typically, forming a curved boring path. After the pilot hole has been completed the borehole is enlarged using reaming tools until the desired bore diameter is achieved. The conduit is typically pulled through the borehole on the final reaming pass. Water based drilling fluids containing bentonite and/or polymers are used during the pilot bore and reaming processes to convey cuttings out of the bore and to stabilize the bore. The bore is typically oversized to minimize the frictional resistance along the conduit during installation. During the pull through pass the annulus between the pipe and the sounding soil is typically filled by displaced drilling fluids that were not displaced out of the bore.

Multiple reaming passes will be required to achieve the desired HDD tunnel diameter. The potential for alignment deviation and development of an oval shaped tunnel cross section increases with the number of reaming passes.

The actual tunnel diameter would have to be oversized to facilitate installation.

Given the large pipe diameter a very long staging area will be required to layout and align the pipe before it can be pulled into the HDD tunnel. For larger diameter and less flexible pipes the layout area should be in line with the bore and only have gradual directional/grade changes when pulled into the tunnel. This is imperative to reduce the flexural stresses and minimize the pulling forces required. A long gently sloping slot trench will likely have to be excavated to provide grade needed, and a pipe layout and assembly area will also be needed beyond the slot trench. Given the length of culverts, it is envisaged that the layout area may have to extend beyond the highway right of way.

With HDD there is potential for inadvertent drilling fluid returns to the ground surface via hydro fracture of the soil surrounding the bore or if the bore crosses pre-existing fissures / preferential seepage paths, or if the drilling fluid pressure exceeds the soils containing capacity. Inadvertent drilling fluid returns could cause loss of drilling fluid circulation along the bore which may hinder or prevent completion of an HDD installation. It is also understood that inadvertent fluid returns would be a potential environmental concern if drilling fluid migrated to a wetland environment.

If inadvertent drilling fluid returns occur at the slopes and ditch areas, there is potential for localized failure of the slope / highway embankment due to surficial erosion and possible collapse of the borehole. Considering that the bore would be unlined during the HDD process, a significant concern with utilization of HDD is the potential for loss of ground / sinkholes to result on the highway. HDD boring is typically done from the ground surface without the use of deep staging excavations, reducing the extent of ground water control required for staging work. However, those conventional open cut excavations would be required to tie into the other drain sections.

The presence of large soil constituents such as cobbles and / or boulders could also hinder an HDD installation and may necessitate the use of specialized tooling and / or larger HDD drill rigs.

The HDD designer/contractor should submit to the Engineer and project team for review the calculation summary for the installation and the performance of the HDD installed pipe subject to service loads, including ring deflection, and safe stresses during installation, pull back forces, safe tensile stress during pull back, buckling loads, drill fluid types and safe drill fluid pressures to avoid day lighting and potential hydro-fracturing, and the like. The design criteria should commensurate with appropriate OPSS, CSA, and ASTM and agency standards such as and CATT, North American Society for Trenchless Technology (NASTT) documents.

HDD installations should be carried out in accordance with OPSS 450, Construction Specifications for Pipeline and Utility Installation in Soil by Horizontal Directional Drilling.

Micro-Tunnelling

Micro-tunneling involves the advancement of a tunnel boring machine from the jacking pit to the receiving pit. The micro-tunnel boring machine (MTBM) is remotely controlled and offers good grade control. The tunnel segments are pushed from the jacking pit while line and grade are controlled by the tunnel boring machine as it advances. These machines may be designed with, cutter head and hydraulic controlled flood doors, and also utilize pressurized bentonite slurry to counterbalance the earth and water pressures acting at the tunnel face. The excavated soil slurry is withdrawn in a controlled manner to prevent loss of ground during tunnel advance. The slurry is circulated back through the tunnel to transport cuttings to a settling tank. Given the machines' design to control soil and water pressures at the face, dewatering prior to advancing the tunnel would not be necessary with this tunnelling method. However, dewatering of the staging and receiving pits may still be required to provide a dry working platform and reference is given to the Staging Excavation section.

Cognizant of the tunnel size, grade requirements and subsurface conditions, micro-tunnelling may considered for the proposed crossing. However, cost effectiveness will depend on availability of a previously used tunnel boring machine of the required size, setting up a bentonite-slurry delivery system at site, and contractor availability. The substantial cost associated with micro-tunneling may be prohibitive considering the relative costs of other tunneling methods that can be used for the proposed tunnel at this site.

# APPENDIX B

Guidelines For Foundation Engineering – Tunnelling Specialty
For Corridor Encroachment Permit Application

## Guidelines For Foundation Engineering – Tunnelling Specialty
## For Corridor Encroachment Permit Application

These guidelines specify MTO's minimum requirements for the Foundation Engineering – Tunnelling Specialty component of submissions from proponents of development within the Ministry of Transportation's (MTO) corridor permit control area. The Foundation Engineering – Tunnelling Specialty component of submissions is a requirement for the permit application only and do not cover all the design requirements.

The complexity ratings of Foundations Engineering services are defined in Table 1.

**Table 1: Complexity ratings for tunnelling specialty services**

| Highway Classification | Tunnel Excavation Diameter (ϕ) | | | | | |
|---|---|---|---|---|---|---|
| | ≤ 1 m | | >1 m & ≤ 2 m | | >2 m | |
| | Minimum Overburden Cover * (m) | | | | | |
| | ≥ 3 ϕ (or 1.5 m whichever is greater) | < 3 ϕ (or 1.5 m whichever is greater) | ≥ 3 ϕ | < 3 ϕ (or 1.5 m whichever is greater) | ≥ 3 ϕ | < 3 ϕ (or 1.5 m whichever is greater) |
| Kings Highway | **Low** | **Medium** | **Medium** | **High** | **High** | **High** |
| 400 Series Freeway | **Medium** | **High** | **High** | **High** | **High** | **High** |

* Minimum overburden cover is the vertical distance measured from the lowest ground elevation to the crown of the tunnel.

Foundations Engineering consultants that are registered in the MTO consultant acquisition system (RAQS) at complexity ratings identified in Table 1 are eligible to provide Foundations Engineering services for this project. Alternatively, the proponents may propose a Foundations Engineering consultant that is not registered in RAQS, in which case, the proponent must submit sufficient documentation to demonstrate that the consultant's qualifications meet or exceed the RAQS complexity requirements.

For Engineering Materials Testing and Evaluation, the consultant shall be qualified for Soil and Rock testing of complexity level at least equal to that identified for this project.

Consultant services shall be provided in accordance with the most recent editions of the Canadian Highway Bridge Design Code (CHBDC), and the 'Guideline for Professional Engineers Providing Geotechnical Engineering Services' published by the Professional Engineers of Ontario.

The designated principal contact identified for Foundations Engineering services by MTO shall sign, and where required, seal, all submissions and correspondence that are submitted to MTO.

Services include, but are not restricted to, conducting a site investigation that shall be of sufficient scope to verify design assumptions and to provide the contractor with adequate subsurface information for design and construction planning.

Sufficient subsurface (factual) information is required to determine the vertical and horizontal extent of subsurface materials (including both soil and rock) and their pertinent engineering properties and groundwater conditions.

Subsurface information is usually acquired by advancing boreholes, laboratory testing of soil samples and rock core samples, performing in-situ tests such as standard penetration tests, dynamic cone tests, and piezocone tests (CPTU) and test pits.

**Minimum requirements for Subsurface Investigation and Recommendations**

A minimum of one borehole shall be advanced at each end of tunnel crossing. The boreholes shall be located outside but within 2 m of the tunnel's excavated footprint.

Spacing between the boreholes shall not exceed 50 m. In case of larger spacing between the boreholes, additional boreholes shall be advanced except where significant traffic disruptions might occur and where consistent conditions are evident.

Boreholes shall be advanced to 3 tunnel diameters (excavated diameters) below invert. If bedrock is encountered earlier, the borehole shall advance to at least 3 m below the invert of tunnel into the bedrock.

The investigations, if required, shall be supplemented with additional and deeper boreholes to verify consistent conditions and existence of boulders within critical foundation zones.

Sampling and testing, consisting of Standard Penetration Test, thin wall tube sample, rock cores, and MTO Field Vane Test where appropriate, shall be conducted to develop a comprehensive subsurface model. Semi-continuous sampling at 0.75m (2.5ft) intervals is required within overburden; whereas, sampling interval of 1.5m (5.0ft) is required below the tunnel invert.

Where encountered, the bedrock-soil interface shall be determined by geological definition and not the by the material properties.

All aspects of implementation of means of subsurface investigations including, but not limited to, planning, licensing, construction, maintenance, abandonment, and reporting, shall be in accordance with Ministry of the Environment Regulation 903 and its amendments (the water well regulation under the OWRA).

Boreholes and piezometer tubes shall be backfilled with a suitable bentonite/cement mixture. Test pits shall be backfilled with suitable material and either re-vegetated or otherwise protected from erosion. Temporary open holes shall be adequately covered. Holes in roads shall be backfilled as required to prevent future settlement and acceptably patched where pavement surfaces have been damaged. Backfilling requirements shall be described in the Foundation Investigation and Design Report.

Where encountered, artesian groundwater conditions shall be sealed. Details of the artesian condition and the sealing operation shall be included in the Foundation Investigation Report.

Fieldwork shall be carried out in accordance with the Occupational Health and Safety Act.

Traffic protection in accordance with MTO requirements shall be provided during the course of any field investigations. However, where significant traffic disruptions might occur, boreholes may be relocated or numbers reduced with MTO's approval.

The locations and ground surface elevations of all boreholes, test pits and soundings shall be surveyed and referred to fixed reference points and data. Locations are to be identified by co-ordinates (Northing and Easting). The vertical accuracy of survey readings shall be within 0.1m; whereas, horizontal accuracy shall be within 0.5m.

**Minimum Laboratory Testing Requirements:**

Laboratory testing shall consist of routine testing of 25% of samples. One routine lab test is defined as natural water content plus Atterberg Limit plus grain size distribution tests. Complex laboratory testing is defined by all other tests including compressive strength, shear strength, consolidation, permeability and triaxial testing. Laboratory testing requirements shall be supplemented with additional routine and complex tests if required to verify strata boundaries and properties and behaviour of critical subsurface zones.

**Borehole Log Preparation and Foundation Drawing:**

Borehole log sheets, figures and drawings shall be prepared in accordance with MTO standards. The Foundation Drawing shall consist of a plan showing the locations of all borings, test pits and soundings and various stratigraphical longitudinal profiles and stratigraphical cross-sections at each tunnel structure foundation element and groundwater levels.

**Minimum Requirements for the Foundation Investigation and Design Report:**

A Foundation Investigation and Design Report shall consist of the factual subsurface information (including the field and laboratory test information) and the recommendations required for foundation design.

The report shall be signed and sealed by two professional engineers, registered with the Professional Engineers of Ontario, representing the consulting firm; one of them shall be the firm's designated principal contact for MTO's Foundations Engineering projects.

- The Foundation Investigation component of the report shall contain:
- Site Description - including topography, vegetation, drainage, existing land use, and structures.

- Investigation Procedures - including site investigation and lab testing procedures.

- Description of Subsurface Conditions - including soil, boulders, rock and groundwater conditions.

- Miscellaneous Section - that identifies the name of the drilling company, the laboratory where testing was performed, the persons who carried out the field supervision, and those who wrote and reviewed the report.

The Foundation Design component of the report shall present discussion and recommendations for design. The consultant shall analyse field data and test results and make comprehensive and practical recommendations pertaining to temporary, interim and permanent conditions at the Project.

The consultant shall identify and evaluate all reasonable and appropriate alternatives for the proposed tunnel crossing. Alternatives may include, but not limited to, jack & bore, pipe jacking using TBM, pipe ramming, micro-tunnelling (if economically feasible), utility tunnelling using TBM (two pass system), Horizontal Directional Drilling (HDD)and cut and cover methods.

The consultant shall identify and present overview assessments of the advantages, disadvantages, costs and risks/consequences of alternative tunnelling methods in a table. The report should conclude a preferred alternative from foundation engineering and cost effectiveness perspective.

In the development and design of the preferred alternative, the Consultant shall, as applicable, address:

- impacts on the land use and property, traffic and transportation, and environment,

- length and diameter constraints

- control of face stability

- capability of boulder excavation

- evaluation of temporary and permanent support

- alignment control

- estimated settlements and heave and management of these deformations

- special access and egress requirements for TBM's and other similar equipment such as those used for the Jack & Bore method including recommendations for vertical shafts and jacking pits;

- shored and un-shored alternatives for open-cut excavation;

- groundwater control & dewatering;
- the long-term stability of the tunnel;
- relative costs; and
- traffic management and contractor access for each alternative.

If borehole logs available from previous projects are included to meet the requirements of field investigations then the accuracy of subsurface information from these boreholes remains the responsibility of consultant except in situations where MTO specify the use of previous boreholes. Borehole logs from previous studies that are appended to the report shall be reformatted to meet the MTO's requirements.

The final foundation recommendations shall detail the geometric, material and strength properties of the new tunnel crossing plus the liner, bedding and backfill requirements, and slope and embankment restoration requirements. The invert elevation should be assessed in view of the subsurface conditions and the anticipated open face stability control.

The consultant is responsible for developing contract documents sufficient to implement the design. This typically includes:

- Contract specifications for materials and specialized construction activities, and
- Recommendations for methods of overcoming anticipated construction problems, in particular, those relating to dewatering, boulder excavation, alignment control and the stability of excavations and embankments.

The consultant shall develop a detailed instrumentation and monitoring program that meets the requirements of these guidelines. (see Attachment for typical settlement monitoring guidelines).

The consultant is responsible for preparing Traffic Control Plans and to obtain approvals and an Encroachment Permit from the Ministry, which are required for lane closures necessary to install the settlement monitoring points.

The tunnelling consultant shall ensure that the foundations engineering component of the project is adequately reflected in the design drawings, specifications and related contract documents.

Written confirmation is required from the Proponent and the tunnelling consultant that the design package submitted to MTO has been reviewed by the tunnelling consultant and that all recommendations have been satisfactorily incorporated in the contract package.

# 1. SETTLEMENT MONITORING GUIDELINES - TUNNELING

The purpose of settlement monitoring is to prevent damage to existing utilities and highway structures along the tunnel alignment. Ground settlement includes settlement due to lost ground and dewatering/drainage.

## Instrumentation Arrays

All measurement points shall be installed and surveyed before the start of excavation to establish benchmarks/baseline.

## Surface Monitoring Points

Surface monitoring points will be installed to cover the whole length of the tunnel within the right of way under the jurisdiction of MTO (Figure 1).

Surface monitoring points will be located at not greater than 5m intervals along the tunnel alignment. The surface monitoring will be identified using paint marks on the pavement. Surface monitoring points installed on the unpaved right of way shall be founded below frost penetration depths. The interval and/or marking of the points should be changed with MTO’s approval where traffic disruptions might occur.

The final instrumentation plan should be finalised when Contractor’s proposed construction method is available.

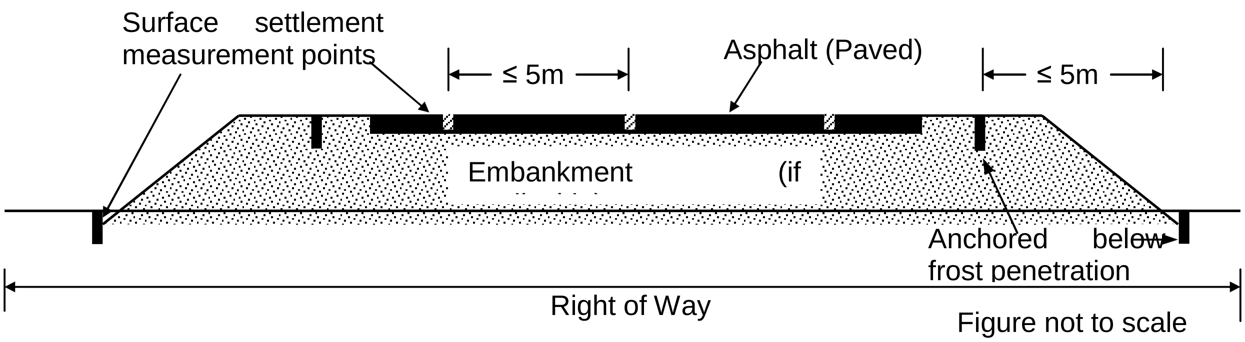


**Figure 1:** Typical configuration of surface settlement monitoring points along the tunnel alignment.

**Condition Survey**

A condition survey for the pavement will be carried out prior to commencement of construction and documented for the purpose of requirement of restoration. The condition survey shall document visible flaws such as cracks, distortions and deviations, heaves, and depressions. This surface survey will be completed during the installation of the monitors and again once the tunnel has been completed.

**Reading Frequency**

An average of at least two readings shall be taken to establish the initial conditions.

The reading and collection of data from the surface monitoring points shall be read and recorded by the Contractor during the construction period and after construction for period of at least 2 weeks provided that further settlement has stopped.

A minimum of three (3) sets of reading be taken daily, provided that movements are within anticipated limits. Otherwise, the frequencies should increase according to a pre-planned interval.

Monitoring of movements is required during work stoppages, such as during non-operation period (off-shifts) or weekends. A minimum of three (3) sets of readings should be taken daily.

Measurements of the monitoring points shall be reported promptly to MTO for review.

**Data Collection and Data Transfer**

A procedure is required to be established in consultation with MTO so that the monitoring data and the interpreted data will reach all parties as soon as necessary. The contract administrator/consultant and the Contractor should interpret monitoring data as needed for the purpose of on-going construction. The Foundation Engineer should be contacted for technical support to the prime Consultant in the interpretation of ground movements and review of the Contractor's response when Review and Alert Levels are reached.

**Criteria for Assessment**

The acceptable surface settlement (or heave) will be according to criteria as specified below.

Baseline Reading – A baseline reading of the instrumentation shall be taken prior to commencement of the work. An average of at least two initial readings shall be recorded as baseline reading.

Review Level – A maximum value of 10 mm relative to the baseline readings is suggested for this project. If this level is reached, the method, rate or sequence of construction, or ground stabilization measures should be reviewed or modified to mitigate further ground displacements.

Alert Level – A maximum value of 15mm relative to the baseline readings is suggested for this project. If this level is reached, the Contractor shall cease construction operations and to execute pre-planned measures to secure the site, to mitigate further movements and to assure safety of public and maintain traffic.

**Review of Contractor's Proposed Method**

MTO, the Proponent's prime consultant and Foundation Engineer should review the Contractor's proposed method of construction. The proposed method should include a description of the potential loss of ground, and calculation of the maximum settlement in relation to the Contractor's procedure and equipment, alternative/remedial measures when review level of measurement is reached; and contingency/remedial measures when alert level of measurement is reached.

**Contractor's Responsibility for Restoration and Warranty Provision**

In addition to the monitoring program to assess the adequacy of the construction method to control potential ground movements and groundwater, the Contractor is responsible for reinstatement (such as surface paving) should movements or other surface distress occur, and provide a reasonable warranty period acceptable to MTO. Remedial measures shall be approved by MTO; however, MTO maintains the right to perform the maintenance at the proponent's expense.

**Construction Monitoring**

The Proponent shall retain a qualified Geotechnical Consultant to supervise the installation of surface settlement points on site and to provide direction, technical input and field inspection on this project.

# APPENDIX C

List of Ontario Provincial Standard Documents Referenced in Report
Draft Non-Standard Specific Provision (NSSP)

## LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

| DOCUMENT | TITLE |
|---|---|
| OPSS 201 | Construction Specification for Clearing, Close Cut Clearing, Grubbing, and Removal of Surface and Piled Boulders |
| OPSS.PROV 206 | Construction Specification for Grading |
| OPSS 404 | Construction Specification for Support Systems |
| OPSS 415 | Construction Specification for Tunneling |
| OPSS 422 | Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut |
| OPSS 490 | Construction Specification for Site Preparation for Pipelines, Utilities, and Associated Structures |
| OPSS 501 | Construction Specification for Compacting |
| OPSS 511 | Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting |
| OPSS 517 | Construction Specification for Dewatering of Pipeline, Utility and Associated Structure Excavation |
| OPSS 518 | Construction Specification for Control of Water from Dewatering Operations |
| OPSS 539 | Construction Specification for Temporary Protection Systems |
| OPSS 801 | Construction Specification for the Protection of Trees |
| OPSS 803 | Construction Specification for Sodding |
| OPSS 902 | Excavation and Backfilling of Structures |
| OPSS.PROV 1004 | Material Specification for Aggregates - Miscellaneous |
| OPSS.PROV 1010 | Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material |
| OPSS 1205 | Material Specification for Clay Seal |
| OPSS 1860 | Material Specification for Geotextiles |
| SP 105S10 | Construction Specification for Compaction |
| SP 206S03 | Construction Specification for Grading |
| SP 422S01 | Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers |
| SP 902S01 | Excavation and Backfilling of Structures |
| OPSD 803.010 | Backfill and Cover for Concrete Culverts with spans less than or equal to 3.0 m |
| OPSD 810.010 | General Rip-Rap Layout Sewer and Culvert Outlets |
| OPSD 3090.101 | Foundation Frost Depth for Southern Ontario |
| MTOD 803.021 | Bedding and Backfill for Precast Concrete Box Culverts |

## DRAFT NON-STANDARD SPECIAL PROVISIONS (NSSP)

NSSP - Variable Mixed Fill and Rock Fill Embankments (Addition to OPSS 902)

The Contractor shall be advised that the embankments contain variable mixed fill and rock fill materials and that the Contractor shall use methods and equipment that are appropriate for the work and capable of dealing with the conditions encountered including, but not limited to various tunnelling techniques and ground improvement techniques such as grouting.

NSSP – Surface Water Control and Dewatering (Addition to OPSS 517 and 518)

The contractor shall be advised that the groundwater is at or near the natural ground surface and that the ground is susceptible to disturbance under conditions of unbalanced hydrostatic head. The contract shall be responsible for designing and implementing measures for surface water control and dewatering. The contractor shall take measures for necessary surface water diversions and drainage and to lower the prevailing groundwater level a minimum of 0.5 m below the base of excavations for work in the dry.

NSSP – Obstructions during Tunneling (Addition to OPSS 490)

The contractor shall be advised that cobbles and boulders are present within the embankment fill and native soils. The contractor shall be responsible for selecting tunnelling methods and equipment that will enable tunnelling operations to advance through the embankment fill and/or native soils including zones where cobbles and boulders are encountered.