



## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by D.M. Wills Associates Ltd. (DMW) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for high fill embankments crossing over swamp areas located within the limits of the new Highway 17 four-laning alignment. The proposed high fill embankments outlined in the project limits are part of the new Highway 17 interchange and extension of the existing four-laning at the West Junction of Sudbury Municipal Road 55, from 20.5 km west of Highway 144, easterly for 6.5 km. The general location of the Highway 17 four-laning extension is shown on the Site Location Plan on Drawing 1.

The Terms of Reference and the Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal, dated March 2011. Golder's proposal for foundation engineering services associated is contained in Section 6.8 of DMW's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated November 11, 2011. The base plan showing the proposed horizontal alignment and a drawing showing the proposed vertical alignment for the Highway 17 four-laning extension was provided to Golder by DMW in January 2012.

This report addresses the investigation carried out for the high fill embankments over swamps only. A detailed list of the locations of the high fill embankments is presented in Table 1. Separate reports detail the results of the foundation investigations for the culverts and bridge structures for this project.

Preliminary subsurface information for this project is available and was supplied by MTO, in the reports and subsequent appendices titled below:

- *Planning, Preliminary Design, and Environmental Assessment Report, Highway 17, Town of Walden, GWP 156-98-00, dated August 2008 by Stantec Consulting Limited*
  - Appendix N: Alternate Route Geotechnical Assessment Report, Highway 17, Town of Walden, GWP 156-98-00, Index No: 080FGR, PML Ref: 05TF059G dated July 29, 2008 by Peto MacCallum Ltd.
  - Appendix O: Alternate Route Foundation Assessment Report, Highway 17, Town of Walden, GWP 156-98-00, Index No: 072FFR, PML Ref: 05TF059F, dated May 20, 2008 by Peto MacCallum Ltd.
- *Planning, Preliminary Design, and Environmental Supplementary Report, Highway 17, Town of Walden, GWP 156-98-00, dated March 2009 by Stantec Consulting Limited*
  - Preliminary Geotechnical Investigation Report, Highway 17, Town of Walden, GWP 156-98-00, Index No: 102FGIR, PML Ref: 05TF059G1 dated March 3, 2009 by Peto MacCallum Ltd.

## 2.0 SITE DESCRIPTION

The overall project consists of the detail design for the four-laning of Highway 17 from the end of the existing four lanes at the west junction of Sudbury Municipal Road 55, from approximately 20.5 km west of Highway 144 easterly for 6.5 km, including a new interchange. The proposed highway alignment approximately follows the existing alignment of Highway 17 generally oriented in an east to west direction and south of the existing highway within the project limits.



In general, the topography of this area consists of rolling terrain, including sparsely populated treed areas and numerous bedrock outcrops separated by low-lying swamps containing areas of standing water and various vegetation types and organic soils. The land use in the general area includes residential and scattered rural farm use. The ground surface within the limits of the study area varies between about Elevation 248 m and Elevation 239 m. A detailed description of each investigated area of high fill embankment over a swamp is presented in Section 4.0.

### 3.0 INVESTIGATION PROCEDURES

The investigation for the Highway 17 high fill embankments crossing over swamp areas was carried out between March 7 and July 12, 2012, between August 27 and October 1, 2012, October 23 and 24, 2012, and January 8 and 9, 2014, during which time a total of one hundred and nineteen (119) boreholes and fifty-three (53) Dynamic Cone Penetration Tests (DCPTs) were advanced. The locations of the boreholes and DCPTs are summarized in Table 1 and are shown on Drawings A1 to C1 in Appendices A to C. In general, boreholes and DCPTs were advanced along the centreline and the toes of the proposed embankment alignment.

The field investigation was carried out using a variety of drilling equipment due to the varying nature of the terrain within the project limits. The details of the drilling equipment and supplier are listed below. Hand excavation methods were used as appropriate depending on the terrain.

| Drilling Equipment   | Supplied and Operated By                      |
|--|---|
| Track Mounted CME-55<br>Track Mounted CME-850<br>Portable Tripod Equipment | Landcore Drilling Inc. of Chelmsford, Ontario |

The boreholes were advanced through the overburden using 108 mm inner diameter hollow-stem augers and/or NW casing with wash boring techniques and NQ size core barrel for bedrock coring, where applicable, to advance through obstructions. In general, soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler (operated by automatic hammers on the drill rigs), in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586, Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils). Boreholes advanced by portable equipment generally employed full weight hammers lifted manually and dropped from the SPT height; however, a limited number of boreholes employed half-weight hammers lifted manually to the SPT height and the 'N'-values were corrected for the lower energy drive. Samples of the cohesive soils were obtained using 76 mm O.D. thin-walled 'Shelby' tubes (ASTM D1587, Standard Practice for Thin-Walled Tube Sampling) for relatively undisturbed samples. Field vane shear tests were conducted in cohesive soils for assessment of undrained shear strengths (ASTM D2573, Standard Test Method for Field Vane Strength Shear Test) using MTO Standard 'N' size vanes. All boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The boreholes were advanced to depths up to about 31 m below existing ground surface, generally penetrating 3 m into competent material, which is defined as material that will provide resistance to settlement or instability of the embankments, or to refusal. Some boreholes and DCPTs were terminated on refusal to further dynamic cone penetration, auger, casing and/or split-spoon advancement. These depths to refusal do not confirm bedrock surface elevations, but may be inferred to indicate the potential proximity to the bedrock surface. At



various locations where refusal was encountered at shallow depth, the bedrock was exposed by hand shovel excavation to confirm the refusal condition.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets provided in Appendices A to C. Groundwater elevations as encountered in the boreholes may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized upon completion of drilling. Furthermore, groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability.

The field work was observed by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury Geotechnical Laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected representative samples. In addition, one-dimensional consolidation (oedometer) tests were carried out on selected samples of the cohesive deposits and the results of the consolidation test results are presented in Table 2. The results of the laboratory testing for each of the high fill embankments are included in the associated appendices.

The proposed centreline of the new Highway 17 alignment was staked in the field by exp. prior to drilling. The as-drilled borehole locations, in stations and offsets, were measured in reference to the centreline alignment and were subsequently converted into coordinates in AutoCAD. Borehole elevations were surveyed by members of our technical staff in reference to the ground surface elevations at temporary benchmarks installed by exp. prior to the commencement of fieldwork. The borehole locations given in the Record of Borehole sheets and shown on the Drawings are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum.

## **4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS**

### **4.1 Regional Geology**

As delineated in the NOEGTS<sup>1</sup> Mapping, the subsoils in this section of Highway 17 are comprised of undulating to rolling glaciolacustrine plain, alluvial plain and organic terrain deposits interspersed with numerous bedrock knobs, outcrops and ridges. In the lower-lying glaciolacustrine plain and alluvial plain areas the primary material consists of wet silts, sands and clays, while the organic terrain deposit the primary material consists of peat. The surface water drainage in the area varies from moderate to poor, corresponding to areas of moderate to low relief.

Based on geological mapping by the Ministry of Natural Resources (Map 2542)<sup>2</sup>, the site is underlain by rocks of the Paleoproterozoic Era belonging to the Huronian Supergroup and Elliot Lake Group consisting of conglomerate, wacke, arkose, quartz arenite, argillite, limestone and dolostone. Areas of mafic and related intrusive rocks comprised of diabase sills, dykes and related granophyre are also present in the vicinity of the

<sup>1</sup>Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Digital Map Reference Number 41ISW.

<sup>2</sup>Ministry of Natural Resources. Bedrock Geology of Ontario – West Central Sheet, Ontario Geological Survey - Map 2542

site. Based on geological mapping by the Ontario Department of Mines (Map 2170)<sup>3</sup> this area is characterized by extensive faults from distinct time periods. The Murray Fault has been identified to run parallel to the approximate proposed alignment of Highway 17.

## **4.2 General Overview of Local Subsurface Conditions**

The detailed subsurface soil and groundwater conditions as encountered in the borings advanced during this investigation together with the results of the laboratory tests carried out on selected soil samples for the respective high fill areas are presented on the Record of Borehole sheets and the laboratory test figures provided in Appendices A to C. The results of the in situ field tests (i.e., SPT 'N'-values and undrained shear strengths from the field vanes) as presented on the Record of Borehole sheets and in Section 4 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of SPTs and in situ testing. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The thickness of the overburden in the investigated areas as inferred from resistance to DCPT results are shown on the Record of Penetration Test sheets in Appendices A to C, as applicable.

The locations of the boreholes and DCPTs advanced in the Highway 17 high fill embankment areas are shown in plan on Drawings A1 to C1, while the inferred soil stratigraphy as encountered shown in profile are shown on Drawings A2 to A4, B2 to B5 and C2 to C4 in Appendices A to C. It should be noted that the orientation (i.e., north, south, east, west) stated in the text of the report is typically referenced to project north and therefore may differ from the Magnetic North shown on the drawings.

In general, the stratigraphy encountered at the various areas investigated is similar. However the overburden (soil materials) thickness is variable, ranging from no cover (bedrock outcrops exposed at the ground surface) to about 31 m. The generalized stratigraphy can be described as follows:

- embankment fill where the existing and proposed alignments overlap;
- surficial layers of topsoil or fibrous peat outside the existing highway;
- cohesionless deposits of sand to sandy silt underlying the organics/fill deposits in some areas;
- cohesive deposits of clayey silt to clay, varved in some areas; and
- cohesionless deposits of silt to sand and gravel below the cohesive deposits to the inferred bedrock surface.

Detailed descriptions of the subsurface conditions encountered at each investigated high fill embankment are provided in the following sections of this report. Where relatively significant thicknesses of overburden were encountered, the various soil types are described in detail for each main deposit or stratum.

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<sup>3</sup> Ontario Department of Mines (1969), Sudbury Mining Area, Sudbury District, Map 2170.



### 4.3 Highway 17 WBL – STA 12+220 to 12+570 (High Fill H1)

The plan and profiles along the centreline and toes of the proposed embankment of the new Highway 17 West Bound Lane (WBL) alignment showing the borehole locations and interpreted stratigraphy between STA 12+220 to STA 12+570 in the Township of Louise are shown in Drawings A1 to A3 in Appendix A. The southerly portion of the alignment extends across a low-lying swampy area with the proposed embankment up to 5.7 m high above the existing ground surface. The northerly portion of the alignment overlaps with the existing Highway 17 embankment. A total of seventeen (17) boreholes (Boreholes H1-1 to H1-9, H1-11 to H1-15, C1-1 to C1-3) and six (6) DCPTs (H1-DC1 to H1-DC6) were completed to investigate the subsurface conditions within this portion of High Fill Area H1.

The subsurface soils along the WBL alignment in High Fill Area H1 consist of embankment fill or a peat/topsoil deposit underlain by a sand and silt deposit and a cohesive deposit of varved, silty clay to clay transitioning into clayey silt, which in turn is underlain by deposits of silt, sand and silt to sand and sand and gravel to gravelly sand.

#### 4.3.1 Asphalt

A 130 mm thick layer of asphalt was encountered at ground surface in Boreholes H1-1 and H1-3.

#### 4.3.2 Fill

Underlying the asphalt in Boreholes H1-1 and H1-3, and from ground surface in Boreholes H1-2, H1-4, H1-5, H1-7, H1-8, H1-9 and H1-12, a stratum of sand to sand and gravel mixed with blast rock fill was encountered. In Borehole H1-8 the sand and gravel transitioned into sandy silt at 2.2 m depth. The surface of the fill stratum was encountered between Elevation 246.5 m and 241.6 m, and the thickness ranges between 0.2 m and 6.1 m. Boreholes H1-1, H1-2 and H1-4 were terminated within this stratum and were located in close proximity to an exposed bedrock cut.

The SPT 'N'-values measured within the sand, sand and gravel and sandy silt fill stratum range from 19 blows to 79 blows per 0.3 m of penetration, indicating a compact to very dense relative density. In Borehole H1-9 a 'N'-value of 113 blows per 0.3 m of penetration was encountered, however it was likely indicative of the frozen nature of the fill at the time of the field investigation. Some instances within the sand and gravel to gravelly sand portions of the fill deposit the split-spoon sampler did not penetrate the full sample depth due to inferred blast rock fill fragments within the fill and rock coring techniques were required to advance the borehole through these zones.

The grain size distributions of two samples of the fill deposit are shown on Figure A1 in Appendix A.

The natural water content measured on a sample of the sand and gravel fill is about 3 per cent. The natural measured water content for a sample of the sandy silt fill is about 13 per cent.

#### 4.3.3 Peat/Topsoil

In Boreholes H1-6, H1-11, H1-13, H1-14, C1-1 and C1-3, a deposit of black fibrous peat was encountered from ground surface. In Boreholes H1-15 and C1-2 a surficial layer of topsoil was encountered at ground surface.

The top of the peat/topsoil layer varies in elevation from 244.6 m to 241.3 m, and the thickness ranges from about 0.1 m to 0.9 m.

The SPT 'N'-values measured within the peat/topsoil deposit range from 0 blows (weight of hammer) to 11 blows per 0.3 m of penetration, indicating a very soft to stiff consistency, however, the higher 'N'-values are attributed to frost.

The natural water content measured on two samples of the peat are about 48 per cent and 157 per cent.

#### **4.3.4 Sand and Silt**

A deposit of sand and silt was encountered underlying the fill in Boreholes H1-3, H1-5 and H1-7 and underlying the peat deposit in Borehole H1-6. The top of the sand and silt deposit was encountered between Elevation 242.1 m and 240.2 m, and the thickness ranges between 1.1 m and 2.3 m. Borehole H1-3 terminated within this deposit.

The SPT 'N'-values measured within the sand and silt layer range from 5 blows to 14 blows per 0.3 m of penetration indicating a loose to compact relative density.

The grain size distributions of two samples of the sand and silt deposit are shown on Figure A2 in Appendix A.

The natural water content measured on two samples of the sand and silt deposit is about 21 per cent and 23 per cent.

#### **4.3.5 Cohesive Deposit**

In Boreholes H1-5 to H1-9, H1-11 to H1-15 and C1-1 to C1-3, a cohesive deposit was encountered beneath the peat/topsoil or fill or sand and silt deposits. In general the cohesive deposit consisted of an upper varved silty clay to clay zone transitioning into a lower clayey silt zone. The top of the cohesive deposit was encountered between Elevation 244.2 m and 238.8 m and the overall deposit ranged between 1.8 m and 7.9 m in thickness.

##### ***Silty Clay to Clay***

Underlying the peat/topsoil in Boreholes H1-11, H1-13, H1-14, H1-15 and C1-1 to C1-3 and underlying the fill and or sand and silt deposits in Boreholes H1-5 to H1-9 and H1-12, a deposit of brown to grey silty clay to clay was encountered. In the majority of the boreholes, the silty clay to clay was observed to be varved, consisting of irregular layers of clayey silt/silty clay and silty clay/clay. The top of the silty clay to clay deposit ranges from Elevation 244.2 m to 238.8 m and the thickness ranges from 1.8 m to 7.9 m.

The SPT 'N'-values measured within this deposit range between 0 blows (weight of hammer) and 17 blows per 0.3 m of penetration, suggesting a very soft to very stiff consistency. In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about 16 kPa to 61 kPa, and the sensitivity is calculated to range from about 2 to 25, typically less than 9. The field vane tests results indicate that the silty clay to clay has a soft to stiff consistency.

The grain size distributions of three samples of the silty clay to clay are shown on Figure A3 in Appendix A.



Atterberg limits tests were carried out on sixteen samples of the silty clay to clay portion of the deposit and indicated liquid limits ranging from about 35 per cent to 61 per cent, plastic limits ranging from about 20 per cent to 26 per cent and plasticity indices ranging from about 13 per cent to 37 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure A4 in Appendix A and indicate that the material is classified as silty clay of intermediate plasticity to clay of high plasticity.

The natural water content measured on twenty-two samples of the silty clay to clay ranges from about 23 per cent to 65 per cent.

### **Clayey Silt**

Underlying the silty clay to clay portion of the deposit in Boreholes H1-6 and H1-15, the silty clay to clay transitioned into grey clayey silt. The top of the clayey silt portion of the deposit ranges from Elevation 241.2 m to 234.1 m and the thickness ranges from 0.9 m to 1.6 m.

The SPT 'N'-values measured within this deposit range between 1 blow and 12 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency. One in situ field vane test carried out within this deposit measured an undrained shear strength of about 44 kPa, and the sensitivity is calculated to be about 9. The field vane test result indicates that the clayey silt has a firm consistency.

An Atterberg limits test was carried out on one sample of the clayey silt portion of the deposit and indicated a liquid limit of about 35 per cent, plastic limit of about 20 per cent and a plasticity index of about 15 per cent. The results of the Atterberg limits test are shown on the plasticity chart on Figure A5 in Appendix A and indicate that the material is classified as clayey silt of low plasticity.

The natural water content measured on one sample of the clayey silt is about 42 per cent.

### **4.3.6 Silt**

Underlying the cohesive deposit in Boreholes H1-5 to H1-9, H1-11, H1-12, H1-14 and C1-1 to C1-3, a deposit of grey silt was encountered. The top of this deposit ranges from about Elevation 241.0 m to 232.4 m, and ranges from 1.3 m to 4.4 m in thickness. A DCPT was advanced below the borehole termination depth in Borehole C1-1.

The SPT 'N'-values measured within the silt to sandy silt portion of the deposit range between 1 blow and 15 blows per 0.3 m of penetration, indicating a very loose to compact relative density. One SPT 'N'-value was measured to be 75 blows per 0.08 m of penetration, likely indicative of the close proximity of the bedrock surface.

The grain size distributions of eight samples of the silt are shown on Figure A6 in Appendix A.

An Atterberg limits test was carried out on one sample of the silt and indicated a liquid limit of about 28 per cent, a plastic limit of about 24 per cent and a plasticity index of just below 4 per cent. The results of the Atterberg limit test are shown on the plasticity chart on Figure A7 in Appendix A and indicate that the material is classified as silt of slight plasticity. The results of four additional Atterberg limits tests indicate that the material is non-plastic.

The natural water content measured on ten samples of the silt deposit ranges between about 23 per cent and 32 per cent.



#### 4.3.7 Sand and Silt to Sand

Underlying the silt in Boreholes H1-4 to H1-9, H1-11 and H1-14 and underlying the cohesive deposit in Boreholes H1-13 and H1-15, a stratum of grey, sand and silt to sand was encountered. The surface of the stratum was encountered between Elevation 240.3 m and 228.5 m. Boreholes H1-5 to H1-9, H1-13 were terminated within this stratum.

The SPT 'N'-values measured within the sand and silt to sand portion of the deposit range between 0 blows (weight of hammer) and 49 blows per 0.3 m of penetration indicating a very loose to dense relative density. One SPT 'N'-value was measured to be 10 blows per 0.1 m of penetration likely indicative of the close proximity of the bedrock surface.

The grain size distributions of six samples of the sand and silt to sand are shown on Figure A8 in Appendix A.

The results of Atterberg limits testing on one sample indicated that the material is classified as non-plastic.

The natural water content measured on six samples of this deposit ranges between about 19 per cent and 24 per cent.

#### 4.3.8 Sand and Gravel

In Boreholes H1-11, H1-13 and H1-15, a deposit of grey sand and gravel was encountered underlying the silty sand deposit. The top of the sand and gravel deposit was encountered between Elevation 239.0 m and 237.6 m and ranged from 1.7 m to 5.4 m in thickness.

The SPT 'N'-values measured within this portion of the deposit range between 3 blows and 24 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The grain size distributions of three samples of the sand and gravel are shown on Figure A9 in Appendix A.

The natural water content measured on three samples of this portion of the deposit ranges between about 8 per cent and 12 per cent.

#### 4.3.9 Refusal

Refusal to split-spoon and dynamic cone penetration, indicating proximity to the inferred bedrock surface was encountered in Boreholes H1-1 to H1-6, H1-11 to H1-15, C1-1 and DCPTs H1-DC1, H1-DC5 and H1-DC6 at depths ranging from 0.2 m and 19.4 m below the ground surface or between Elevation 245.2 m and 223.8 m. In Boreholes C1-2 and DCPTs H1-DC2 to H1-DC4, the DCPT terminated upon recording greater than 100 blows per 0.3 m of penetration, indicative of proximity to the inferred bedrock surface, at depths up to about 17 m below the existing ground surface.

In Borehole H1-12, split-spoon refusal was encountered at 6.6 m depth (Elevation 239.7 m) within the silt stratum. In this instance, refusal is likely indicative of the presence of an obstruction.



#### 4.3.10 Groundwater Conditions

In general, the soil samples taken in the boreholes were moist to wet. Boreholes H1-1 to H1-4 were dry upon completion of drilling. Groundwater levels observed upon completion of drilling in the remaining boreholes range from Elevation 242.7 m to 239.8 m, measured between ground surface and 6.2 m below ground surface. Borehole H1-6 measured a groundwater level of 0.3 m above ground surface (Elevation 241.6 m) upon completion of drilling. It should be noted that the groundwater levels in the area fluctuate seasonally as well as during precipitation events and snowmelt.

#### 4.4 Highway 17 EBL – STA 12+220 to 12+570 (High Fill H1)

The plan and profiles along the centreline and toes of the proposed embankment of the new Highway 17 East Bound Lane (EBL) alignment showing the borehole locations and interpreted stratigraphy between about STA 12+220 to STA 12+570 in the Township of Louise are shown in Drawings A1 to A4 in Appendix A. The EBL extends across a low-lying swampy area with the proposed embankment up to 5.7 m high above the existing ground. A total of seventeen (17) boreholes (Boreholes H1-16 to H1-30, C1-3 and C1-4) and six (6) DCPTs (H1-DC7 to H1-DC12) were completed to investigate the subsurface conditions within this portion of High Fill Area H1.

The subsurface soils along the EBL alignment in High Fill Area H1 consist of a surficial layer of peat/topsoil underlain by a sandy silt deposit and a cohesive deposit of clayey silt to silt transitioning into varved, silty clay to clay transitioning back into clayey silt, which in turn is underlain by deposits of silt to sandy silt, sand and silt to sand and gravelly sand.

##### 4.4.1 Peat/Topsoil

In all boreholes except H1-29 a deposit of black fibrous peat was encountered from ground surface. In Borehole H1-29 a surficial layer of topsoil was encountered at ground surface. The top of the peat/topsoil layer varies in elevation from 241.6 m to 241.0 m and the thickness ranges from 0.2 m to 1.4 m. In Borehole H1-23, a layer of brown to black organic clay was encountered below the peat at 0.3 m depth and was 1.0 m in thickness.

The SPT 'N'-values measured within the peat/topsoil deposit range from 0 blows (weight of hammer) to 7 blows per 0.3 m of penetration, indicating a very soft to firm consistency, however, the higher 'N'-values are typically attributed to frozen ground conditions.

The natural water content measured on seven samples of the peat ranges from about 39 per cent to 420 per cent.

##### 4.4.2 Sandy Silt

A layer of sandy silt was encountered underneath the peat/organics layer in Boreholes H1-16 to H1-19, and H1-22. The top of the deposit was encountered between Elevation 241.0 m and 239.8 m and ranged from 0.2 m to 0.8 m in thickness.

The SPT 'N'-values measured within the sandy silt layer range from 4 blows to 14 blows per 0.3 m of penetration indicating a loose to compact relative density.



The grain size distribution of one sample of the sandy silt is shown on Figure A10 in Appendix A.

The natural water content measured on two samples of the sandy silt deposit are about 23 per cent and 33 per cent.

#### **4.4.3 Cohesive Deposit**

In all boreholes, a cohesive deposit was encountered beneath the peat/organics deposit or beneath the sandy silt deposit. In general the cohesive deposit consisted of an upper clayey silt to silt zone transitioning into varved silty clay to clay, further transitioning back to a lower clayey silt to silt zone. The top of the cohesive deposit was encountered between Elevation 241.2 m and 239.0 m and the overall deposit ranged between 2.0 m and 9.4 m in thickness.

##### ***Clayey Silt to Silt***

Underlying the sandy silt deposit in Borehole H1-17 and underlying the peat/organic clay deposit in Boreholes H1-23 and H1-25, a clayey silt to silt deposit was encountered. The top of the clayey silt to silt portion of the deposit was encountered between Elevation 240.6 m and 240.0 m and ranged from 0.9 m to 2.4 m in thickness.

The SPT 'N'-values measured within this portion of the deposit range between 1 blow and 7 blows per 0.3 m of penetration, indicating a very soft to firm consistency.

The grain size distribution of one sample of the clayey silt to silt portion of the deposit is shown on Figure A11 in Appendix A.

Atterberg limits tests were carried out on three samples of the clayey silt to silt portion of the deposit and test results indicate liquid limits ranging from about 24 per cent to 32 per cent, plastic limits ranging from about 17 per cent to 21 per cent and plasticity indices ranging from about 7 per cent to 12 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure A12 in Appendix A and indicate that the material is classified as clayey silt to silt of low plasticity.

The natural water content measured on three samples of this portion of the deposit ranges between about 26 per cent and 30 per cent.

##### ***Silty Clay to Clay***

In all boreholes a deposit of grey, silty clay to clay was encountered underlying the peat and/or silt to sandy silt and/or the clayey silt deposit. In the majority of the boreholes, the silty clay to clay portion of the deposit was observed to be varved, consisting of irregular layers of clayey silt/silty clay and silty clay/clay. The top of the silty clay to clay portion of the deposit was encountered between Elevation 241.2 m and 238.2 m and ranged from 2.0 m to 8.2 m in thickness.

The SPT 'N'-values measured within this portion of the deposit range between 0 blows (weight of hammer) and 15 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency. In situ field vane tests carried out within this portion of the deposit measured undrained shear strengths ranging from about 13 kPa to 82 kPa and

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the sensitivity is calculated to range from about 2 to 18, typically less than 12. One in situ field vane in Borehole H1-21 was measured to be greater than 100 kPa. The field vane tests results indicate that the silty clay to clay has a soft to stiff consistency.

The grain size distributions of four samples of the silty clay to clay portion of the deposit are shown on Figure A3 in Appendix A.

Atterberg limits tests were carried out on twenty-four samples of the silty clay to clay portion of the deposit. The test results indicate liquid limits ranging from about 36 per cent to 76 per cent, plastic limits ranging from about 18 per cent to 30 per cent and plasticity indices ranging from about 17 per cent to 50 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure A14 in Appendix A and indicate that the material is classified as silty clay of intermediate plasticity to clay of high plasticity.

The natural water content measured on thirty samples of this portion of the deposit ranges between about 21 per cent and 79 per cent.

Laboratory consolidation (oedometer) tests were carried out on two samples of the silty clay to clay, obtained from Shelby tube samples in Boreholes H1-19 and H1-25. The preconsolidation stress was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of 16.3 kN/m<sup>3</sup> and 16.9 kN/m<sup>3</sup> and a specific gravity of 2.77 and 2.78 were measured on the consolidation test samples. The detailed results of the oedometer tests are shown on Figures A15 and A16 in Appendix A, and the test results are summarized below, and in Table 2.

| Borehole/<br>Sample No. | Sample<br>Depth /<br>Elevation | $\sigma_{vo}'$<br>(kPa) | $\sigma_p'$<br>(kPa) | $\sigma_p' - \sigma_{vo}'$<br>(kPa) | OCR | $e_o$ | $C_c$ | $C_r$ | $c_v^*$<br>(cm <sup>2</sup> /s) |
|-------------------------|--------------------------------|-------------------------|----------------------|-------------------------------------|-----|-------|-------|-------|---------------------------------|
| H1-19/Sample 6          | 4.6 m/<br>236.4 m              | 31                      | 256                  | 225                                 | 8.3 | 1.86  | 1.04  | 0.02  | $2.1 \times 10^{-3}$            |
| H1-25/Sample 8          | 6.6 m/<br>234.6 m              | 56                      | 135                  | 79                                  | 2.4 | 1.52  | 0.66  | 0.02  | $3.8 \times 10^{-3}$            |

\*For the normally consolidated stress range

where:  $\sigma_{vo}'$  is the effective overburden stress in kPa  
 $\sigma_p'$  is the preconsolidation stress in kPa  
OCR is the overconsolidation ratio  
 $e_o$  is the initial void ratio  
 $C_c$  is the compression index  
 $C_r$  is the recompression index  
 $c_v$  is the coefficient of consolidation in cm<sup>2</sup>/s

### Clayey Silt

Underlying the silty clay to clay in Boreholes H1-19 to H1-21, H1-23, H1-27 and C1-4, a deposit of grey clayey silt was encountered between Elevation 238.0 m and 231.9 m and ranged from 1.0 m to 2.4 m in thickness.

The SPT 'N'-values measured within the clayey silt deposit range between 0 blows (weight of hammer) and 5 blows per 0.3 m of penetration, suggesting a very soft to firm consistency. In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about 23 kPa to 61 kPa and the sensitivity is calculated to range from about 2 to 11, typically less than 4. The field vane test results indicate that the clayey silt has a soft to stiff consistency.



Atterberg limits tests were carried out on three samples of the clayey silt and indicate liquid limits ranging from about 29 per cent to 33 per cent, plastic limits ranging from about 20 per cent to 21 per cent and plasticity indices ranging from about 9 per cent to 13 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure A17 in Appendix A and indicate that the material is classified as clayey silt of low plasticity.

The natural water content measured on three samples of the deposit ranges between about 33 per cent and 42 per cent.

#### **4.4.4 Silt to Sandy Silt**

Underlying the cohesive deposit is a deposit of grey, wet, silt to sandy silt which was encountered in all boreholes except H1-27. The top of the silt to sandy silt deposit was encountered between Elevation 239.0 m and 230.9 m and ranged from 0.8 m to 7.6 m in thickness, where the deposit was fully penetrated. Borehole H1-24 was terminated within this deposit.

The SPT 'N'-values measured within this portion of the deposit range between 0 blows (weight of hammer) to 26 blows per 0.3 m of penetration, indicating a very loose to compact relative density. Two SPT 'N'-values did not penetrate the full sample depth and were measured to be 22 blows per 0.15 m of penetration and 2 blows per 0.1 m of penetration, likely indicative of the close proximity of the bedrock surface.

The grain size distributions of twelve samples of the silt to sandy silt are shown on Figure A18 in Appendix A.

The results of Atterberg limits testing on five samples indicated that the material is classified as non-plastic.

The natural water content measured on fifteen samples of this deposit ranges between about 22 per cent and 42 per cent.

#### **4.4.5 Sand and Silt to Sand**

Underlying the silt to sandy silt deposit in Boreholes H1-17 to H1-23, H1-25, H1-26, H1-29, H1-30, C1-3 and C1-4, a deposit of grey sand and silt to sand was encountered between Elevation 237.5 m and 227.7 m and ranged from 0.4 m to 8.8 m in thickness where the deposit was fully penetrated. DCPTs were advanced below the borehole termination depth in Boreholes H1-26, H1-30, C1-3 and C1-4. Boreholes H1-17, H1-21 to H1-23, H1-26, H1-29, H1-30, C1-3 and C1-4 were terminated within this deposit.

The SPT 'N'-values measured within this portion of the deposit range between 0 blows (weight of hammer) and 60 blows per 0.3 m of penetration, indicating a very loose to very dense relative density.

The grain size distributions of eight samples of the sand and silt to sand are shown on Figure A19 in Appendix A.

The natural water content measured on nine samples of this portion of the deposit ranges between about 17 per cent and 28 per cent.





#### **4.4.6 Gravelly Sand**

In Boreholes H1-19, H1-25 and H1-27, a deposit of gravelly sand was encountered underlying the sand and silt to sand and/or the cohesive deposit. The top of the gravelly sand deposit was encountered between Elevation 235.6 m and 225.6 m and ranged from 2.8 m to 3.4 m in thickness where the deposit was fully penetrated. A DCPT was advanced below the borehole termination depth in Borehole H1-25. Boreholes H1-19 and H1-25 were terminated within this deposit.

The SPT 'N'-values measured within this portion of the deposit range between 13 blows and 28 blows per 0.3 m of penetration, indicating a compact relative density.

The grain size distributions of two samples of the gravelly sand are shown on Figure A20 in Appendix A.

The natural water content measured on two samples of this portion of the deposit are about 11 per cent and 19 per cent.

#### **4.4.7 Refusal**

Refusal to split-spoon and dynamic cone penetration, indicating proximity to the inferred bedrock surface was encountered in Boreholes H1-16, H1-18, H1-20, H1-25 to H1-30, C1-3 and C1-4 and DCPTs H1-DC7 and H1-DC10 to H1-DC-12 at depths ranging from about 4.0 m and 20.7 m below the ground surface or between Elevation 237.3 m and 220.5 m. In DCPTs H1-DC8 and H1-DC9, the DCPT terminated due to recording greater than 100 blows per 0.3 m of penetration, indicative of proximity to the inferred bedrock surface, at depths up to about 21 m below the existing ground surface.

#### **4.4.8 Groundwater Conditions**

In general, the soil samples taken in the boreholes were moist to wet. Water levels observed upon completion of drilling ranged from Elevation 241.3 m to 238.3 m, corresponding to depths between ground surface and 3.0 m below ground surface. In Boreholes H1-21 and H1-22, 0.2 m of ponded water was encountered at the ground surface. It should be noted that the groundwater levels in the area fluctuate seasonally as well as during precipitation events and snowmelt.

### **4.5 Highway 17 WBL – STA 13+140 to 13+390 (High Fill H2)**

The plan and profiles along the centreline and toes of the proposed embankment of the new Highway 17 WBL alignment showing the borehole locations and interpreted stratigraphy between STA 13+140 and STA 13+390 in the Township of Louise are shown on Drawings B1 to B6 in Appendix B. The alignment extends across a low-lying swampy area with the proposed embankment up to 5.0 m high above the existing ground. A total of twenty-one (21) boreholes (Boreholes H2-1 to H2-21) and ten (10) DCPTs (DCPTs H2-DC1 to H2-DC10) were completed to investigate the subsurface conditions within this portion of High Fill Area H2.

The subsurface soils along the WBL alignment in High Fill Area H2 consist of a surficial layer of peat/organics, underlain by a cohesive deposit of clayey silt transitioning into varved, silty clay to clay transitioning to clayey silt to silt, which in turn is underlain by deposits of silt to sand to gravelly sand.

#### 4.5.1 Peat/Organics

An approximate 0.1 m to 4.0 m thick deposit of black, fibrous to amorphous peat was encountered at the ground surface in Boreholes H2-2 to H2-20. In Borehole H2-1, a layer of organics was encountered at the ground surface. The surface of the peat/organic deposit varies between Elevation 240.7 m and 239.7 m.

The SPT 'N'-values measured within the peat/organic deposit are typically 0 blows (weight of hammer) to 1 blow per 0.3 m of penetration, suggesting a very soft consistency, however 'N'-values up to 31 blows per 0.3 m of penetration were noted through frozen peat/organic materials.

The natural water content measured on seventeen samples of the peat ranges from about 82 per cent to 670 per cent.

#### 4.5.2 Cohesive Deposit

In all boreholes, a cohesive deposit was encountered beneath the peat/organics deposit or from ground surface in Borehole H2-21. In general, the cohesive deposit consisted of an upper clayey silt to silt zone transitioning into varved silty clay to clay, further transitioning to a lower clayey silt to silt zone. The top of the cohesive deposit was encountered between Elevation 240.6 m and 235.8 m and the overall deposit ranged between 1.7 m and 6.9 m in thickness.

##### *Clayey Silt*

In Boreholes H2-1, H2-2 and H2-21 a deposit of grey clayey silt, trace sand, trace organics was encountered underlying the peat/organics deposit. The surface of this portion of the deposit was encountered between Elevation 240.6 m and 238.9 m and ranged from 0.8 m to 4.3 m in thickness.

The SPT 'N'-values measured within this portion of the deposit range between 0 blows (weight of hammer) and 15 blows per 0.3 m of penetration, suggesting a very soft to very stiff consistency. Two in situ field vane tests carried out within this portion of the deposit measured undrained shear strengths of about 43 kPa and 57 kPa and the sensitivity is calculated to be about 5 and 6. The field test results indicate that this portion of the deposit has a firm to stiff consistency.

Atterberg limits tests were carried out on three samples of the clayey silt and indicate liquid limits ranging from about 31 per cent to 34 per cent, plastic limits ranging from about 19 per cent to 22 per cent and plasticity indices ranging from about 9 per cent to 14 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B1 in Appendix B and indicate that the material is classified as clayey silt of low plasticity.

The natural water content measured on five samples of this portion of the deposit ranges between about 20 per cent and 35 per cent.

##### *Silty Clay to Clay*

Underlying the peat/organic deposit in Boreholes H2-3 to H2-20 and underlying the clayey silt layer in H2-21, a deposit of grey to brown, silty clay to clay was encountered. In the majority of the boreholes, the silty clay to clay portion of the deposit was observed to be varved, consisting of irregular layers of clayey silt/silty clay and silty



clay/clay. The surface of this portion of the deposit was encountered between Elevation 240.1 m and 235.8 m and ranged from 1.7 m to 6.8 m in thickness.

The SPT 'N'-values measured within this portion of the deposit range between 0 blows (weight of hammer) and 12 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency. In situ field vane test carried out within this deposit measured undrained shear strengths ranging from about 14 kPa to 79 kPa and the sensitivity is calculated to be between about 2 and 16. The field test results indicate that the deposit has a soft to stiff consistency.

The grain size distributions of four samples of this portion of the deposit are presented on Figure B2 in Appendix B.

Atterberg limits tests were carried out on twenty-two samples of the silty clay to clay and indicate liquid limits ranging from about 38 per cent to 80 per cent, plastic limits ranging from about 21 per cent to 25 per cent and plasticity indices ranging from about 17 per cent to 54 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B3 in Appendix B and indicate that the material is classified as silty clay of intermediate plasticity to clay of high plasticity.

The natural water content measured on thirty samples of this portion of the deposit ranges between about 30 per cent and 73 per cent.

### ***Clayey Silt to Silt***

In Boreholes H2-7, H2-8, H2-11, H2-18, H2-19 and H2-20, the silty clay to clay transitioned into a grey, clayey silt. The surface of this portion of the deposit was encountered between Elevation 237.0 m and 231.0 m and ranged from 1.3 m to 3.8 m in thickness.

The SPT 'N'-values measured within this portion of the deposit range between 0 blows (weight of hammer) and 3 blows per 0.3 m of penetration, suggesting a very soft to soft consistency. In situ field vane test carried out within this portion of the deposit measured undrained shear strengths ranging from about 24 kPa to 53 kPa and the sensitivity is calculated to be between about 3 and 13. The field test results indicate that this portion of the deposit has a soft to stiff consistency.

A grain size distribution for one sample of this portion of the deposit is presented on Figure B4 in Appendix B.

Atterberg limits tests were carried out on four samples of the lower clayey silt to silt. The test results indicate liquid limits ranging from about 27 per cent to 29 per cent, plastic limits ranging from about 20 per cent to 23 per cent and plasticity indices ranging from about 6 per cent to 9 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B5 in Appendix B and indicate that the material is classified as clayey silt of low plasticity to silt of slight plasticity.

The natural water content measured on six samples of this portion of the deposit ranges between about 33 per cent and 44 per cent.

### **4.5.3 Silt to Sandy Silt**

A deposit of grey silt to sandy silt, trace to some clay, trace to some sand was encountered beneath the cohesive deposit in all of the boreholes within this section of the project.



The surface of this deposit ranges from Elevation 238.5 m to 229.6 m and its thickness ranges from 0.5 m to 10.0 m where the deposit was fully penetrated. Boreholes H2-2 to H2-4, H2-9 to H2-11 and H2-13 were terminated within this deposit.

The SPT 'N'-values measured within this deposit range between 0 blows (weight of hammer) and 29 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The grain size distributions of nineteen samples of this deposit are presented on Figure B6 in Appendix B. The results of Atterberg limits testing on seven samples of the silt deposit indicated that the material is classified as non-plastic.

The natural water content measured on twenty-six samples of this deposit ranges between about 18 per cent and 37 per cent.

#### **4.5.4 Sand and Silt to Sand**

A deposit of grey sand and silt to sand, trace clay was encountered underlying the silt deposit in Boreholes H2-5, H2-6, H2-8, H2-14 to H2-18, H2-20 and H2-21. The surface of this deposit ranges from Elevation 233.3 m to 226.6 m. DCPTs were advanced below the borehole termination depth in Boreholes H2-5, to H2-7, H2-14 to H2-18 and H2-21. Borehole H2-8 was terminated within this deposit.

The SPT 'N'-values measured within this deposit range between 0 blows (weight of hammer) and 18 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The grain size distributions of six samples of this deposit are presented on Figure B7 in Appendix B.

The natural water content measured on six samples of this deposit ranges between about 10 per cent and 25 per cent.

#### **4.5.5 Gravelly Sand**

A 0.1 m to 2.5 m thick deposit of gravelly sand was encountered in Boreholes H2-1 and H2-19 underlying the silt to sandy silt deposit at Elevation 238.0 m and 226.9 m, respectively. Borehole H2-19 was terminated in this deposit.

The SPT 'N'-values measured within this portion of the deposit range between 6 blows and 17 blows per 0.3 m of penetration, indicating a loose to compact relative density.

A grain size distribution for one sample of the gravelly sand is presented on Figure B8 in Appendix B.

The natural water content measured on one sample of this portion of the deposit is about 10 per cent.

#### **4.5.6 Refusal**

Refusal to split-spoon, auger or casing advancement or dynamic cone penetration, indicating proximity to the inferred bedrock surface was encountered in Boreholes H2-1 to H2-6, H2-17, H2-18 and H2-20 and in DCPTs H2-DC1 to H2-DC3, H2-DC7 and H2-DC8 at depths ranging from 2.8 m to 20.3 m below the ground surface or between Elevation 237.9 m and 220.2 m. In Boreholes H2-7, H2-12, H2-14 and H2-21 and DCPTs H2-DC4 to





H2-DC6, H2-DC9 and H2-DC10, the DCPTs terminated upon recording greater than 100 blows per 0.3 m of penetration, indicative of proximity to the inferred bedrock surface, at depths up to about 23 m below the existing ground surface.

#### **4.5.7 Groundwater Conditions**

In general, the samples taken in the boreholes were wet. Artesian groundwater levels were measured in Borehole H2-6 and H2-8 with the groundwater level upon completion of drilling measured at 0.5 m and 0.8 m above ground surface, respectively, corresponding to Elevation 240.3 m and 239.9 m. In the remaining boreholes, the groundwater levels observed upon completion of drilling range from about Elevation 235.2 m to 239.9 m, typically measured at the ground surface up to 4.7 m below ground surface, except in Borehole H2-1, which was observed to be dry upon completion of drilling. It should also be noted that the groundwater levels in the area fluctuate seasonally, as well as during precipitation events and snowmelt.

### **4.6 Highway 17 EBL – STA 13+140 to 13+390 (High Fill H2)**

The plan and profiles along the centreline and toes of the proposed embankment of the new Highway 17 EBL alignment showing the borehole locations and interpreted stratigraphy between about STA 13+140 to STA 13+390 in the Township of Louise are shown on Drawings B1 to B6 in Appendix B. The alignment extends across a low-lying swampy area with the proposed embankment up to 5.0 m high above the existing ground. A total of twenty-two (22) boreholes (Boreholes H2-22 to H2-43) and eleven (11) DCPTs (DCPTs H2-DC11 to H2-DC21) were completed to investigate the subsurface conditions within this portion of High Fill Area H2.

The subsurface soils along the EBL alignment in High Fill Area H2 consist of a surficial deposit of peat/organics underlain by a cohesive deposit of clayey silt transitioning into varved, silty clay to clay transitioning to clayey silt, which in turn is underlain by deposits of silt to sand to sand and gravel.

#### **4.6.1 Peat/Organics**

An approximately 0.1 m to 3.7 m thick layer of black, fibrous to amorphous peat was encountered at the ground surface in Boreholes H2-23 to H2-41. In Borehole H2-22, H2-42 and H2-43, a layer of organics was encountered at the ground surface. The top of the peat/organic deposit varies between Elevation 242.1 m and 239.6 m.

The SPT 'N'-values measured within the peat/organic deposit range from 0 blows (weight of hammer) to 2 blows per 0.3 m of penetration, suggesting a very soft consistency, however 'N'-values up to 14 blows per 0.3 m of penetration were noted through frozen peat.

The natural water content measured on eighteen samples of this portion of the deposit ranges from about 297 per cent to 774 per cent.

#### **4.6.2 Cohesive Deposit**

In all boreholes, a cohesive deposit was encountered beneath the peat/organics deposit. In general, the cohesive deposit consisted of an upper clayey silt to silt zone transitioning into varved silty clay to clay, further

transitioning back to a lower clayey silt to silt zone. The top of the cohesive deposit was encountered between Elevation 242.0 m and 236.0 m and overall deposit ranged between 0.5 m and 9.7 m in thickness.

### ***Clayey Silt***

In Boreholes H2-23, H2-30 to H2-35, H2-37, H2-39 and H2-43, a deposit of grey clayey silt trace sand, trace organics was encountered underlying the peat/organics deposit. The surface of this portion of the deposit was encountered between Elevation 242.0 m and 236.0 m and ranged from 0.5 m to 2.1 m in thickness.

The SPT 'N'-values measured within the clayey silt deposit range from 0 blows (weight of hammer) to 14 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency. One in situ field vane test carried out within this portion of the deposit measured an undrained shear strength of about 24 kPa and the sensitivity is calculated to be about 2. The field test results indicate that this portion of the deposit has a soft consistency.

Atterberg limits tests were carried out on four samples of the clayey silt. The test results indicate liquid limits ranging from about 28 per cent to 34 per cent, plastic limits ranging from about 18 per cent to 22 per cent and plasticity indices ranging from about 9 per cent to 13 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B9 in Appendix B and indicate that the material is classified as a clayey silt of low plasticity.

The grain size distribution of one sample of the clayey silt portion of the deposit is shown on Figure B10 in Appendix B.

The natural water content measured on six samples of this portion of the deposit ranges from about 22 per cent to 36 per cent.

### ***Silty Clay to Clay***

A deposit of silty clay to clay was encountered underlying the peat/organic deposit or the clayey silt deposit in Boreholes H2-25 to H2-42. In the majority of the boreholes, the silty clay to clay portion of the deposit was observed to be varved, consisting of irregular layers of clayey silt/silty clay and silty clay/clay. The surface of the silty clay to clay was encountered between Elevation 241.7 m to 234.1 m, and ranged in thickness between 2.1 m and 8.5 m.

The SPT 'N'-values measured within the silty clay to clay deposit range between 0 blows (weight of hammer) and 12 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency. In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about 12 kPa to 91 kPa and the sensitivity is calculated to be between about 2 and 35 (typically less than 16). The field test results indicate that the deposit has a soft to stiff consistency.

The grain size distributions of four samples of the silty clay to clay portion of the deposit are shown on Figure B11 in Appendix B.

Atterberg limits tests were carried out on twenty-four samples of the silty clay to clay. The test results indicate liquid limits ranging from about 36 per cent to 80 per cent, plastic limits ranging from about 19 per cent to 27 per cent and plasticity indices ranging from about 14 per cent to 55 per cent. The results of the Atterberg

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limits tests are shown on the plasticity chart on Figure B12 in Appendix B and indicate that the material is classified as silty clay of intermediate plasticity to clay of high plasticity.

The natural water content measured on thirty samples of this portion of the deposit ranges from about 30 per cent to 81 per cent.

Laboratory consolidation (oedometer) tests were carried out on two samples of the silty clay to clay obtained from Shelby tube samples in Boreholes H2-26 and H2-36. The preconsolidation stress was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of 15.6 kN/m<sup>3</sup> and 16.6 kN/m<sup>3</sup> and a specific gravity of 2.77 and 2.78 were measured on the consolidation test samples. The detailed results of the oedometer tests are shown on Figures B13 and B14 in Appendix B, and the test results are summarized below, and in Table 2.

| Borehole/<br>Sample No. | Sample<br>Depth /<br>Elevation | $\sigma_{vo}'$<br>(kPa) | $\sigma_p'$<br>(kPa) | $\sigma_p' - \sigma_{vo}'$<br>(kPa) | OCR | $e_o$ | $C_c$ | $C_r$ | $c_v^*$<br>(cm <sup>2</sup> /s) |
|-------------------------|--------------------------------|-------------------------|----------------------|-------------------------------------|-----|-------|-------|-------|---------------------------------|
| H2-26/Sa 7              | 6.4 m /<br>233.3 m             | 39                      | 128                  | 89                                  | 3.2 | 1.67  | 0.48  | 0.01  | $2.0 \times 10^{-3}$            |
| H2-36/Sa 8A             | 7.1 m /<br>232.7 m             | 47                      | 132                  | 85                                  | 2.8 | 2.13  | 0.48  | 0.08  | $3.8 \times 10^{-4}$            |

\*For the normally consolidated stress range.

where:  $\sigma_{vo}'$  is the effective overburden stress in kPa  
 $\sigma_p'$  is the preconsolidation stress in kPa  
OCR is the overconsolidation ratio  
 $e_o$  is the initial void ratio  
 $C_c$  is the compression index  
 $C_r$  is the recompression index  
 $c_v$  is the coefficient of consolidation in cm<sup>2</sup>/s

### **Clayey Silt to Silt**

In Boreholes H2-26, H2-32 to H2-34, H2-39 and H2-41, the silty clay to clay transitioned into grey, clayey silt to silt. The surface of this portion of the deposit was encountered between Elevation 237.2 m and 229.5 m and ranged from 1.4 m to 4.9 m in thickness.

The SPT 'N'-values measured within the clayey silt to silt deposit range from 0 blows (weight of hammer) to 6 blows per 0.3 m of penetration, suggesting a very soft to firm consistency. In situ field vane test carried out within this deposit measured undrained shear strengths ranging from about 19 kPa to 91 kPa and the sensitivity is calculated to be between about 2 and 19 (typically less than 7). The field test results indicate that the deposit has a soft to stiff consistency.

Atterberg limits tests were carried out on five samples of the clayey silt to silt. The test results indicate liquid limits ranging from about 26 per cent to 32 per cent, plastic limits ranging from about 20 per cent to 22 per cent and plasticity indices ranging from about 5 per cent to 12 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B15 in Appendix B and indicate that the material is classified as a clayey silt to silt of low plasticity.

The natural water content measured on six samples of this portion of the deposit ranges from about 36 per cent to 41 per cent.

#### **4.6.3 Silt to Sandy Silt**

A deposit of grey silt to sandy silt, trace to some clay, trace to some sand was encountered beneath the organics in Boreholes H2-22 and H2-24 and beneath the cohesive deposit in Boreholes H2-25, H2-28 to H2-38, H2-42 and H2-43. The surface of this deposit ranges from Elevation 240.6 m to 226.5 m and its thickness ranges from about 0.6 m to 10.4 m where the deposit was fully penetrated. Borehole H2-31 was terminated within this deposit.

The SPT 'N'-values measured within the silt to sandy silt deposit range from 0 blows (weight of hammer) to 28 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The grain size distributions of sixteen samples of the silt to sandy silt deposit are shown on Figure B16 in Appendix B. The results of Atterberg limits testing on ten samples of the silt to sandy silt deposit indicated that the material is classified as non-plastic.

The natural water content measured on twenty-two samples of the silt to sandy silt deposit ranges from about 19 per cent to 37 per cent.

#### **4.6.4 Sand and Silt to Sand**

A deposit of grey, sand and silt to sand, trace clay was encountered underlying the silt to sandy silt and/or the cohesive deposit in Boreholes H2-23, H2-28, H2-30, H2-32 to H2-35 and H2-37 to H2-43. The surface of this deposit ranges from Elevation 239.8 m to 217.3 m and ranges from 1.3 m to 4.7 m where fully penetrated. DCPTs were advanced below the borehole termination depth in Boreholes H2-30, H2-42 and H2-43. Boreholes H2-32 to H2-35 were terminated within this deposit.

The SPT 'N'-values measured within the sand and silt to sand deposit range from 0 blows (weight of hammer) to 34 blows per 0.3 m of penetration indicating a very loose to dense relative density.

The grain size distribution of thirteen samples of the sand and silt to sand deposit is shown on Figure B17 in Appendix B.

The natural water content measured on fourteen samples of the sand and silt to sand deposit ranges from about 10 per cent to 27 per cent.

#### **4.6.5 Gravelly Sand to Sand and Gravel**

In Boreholes H2-24, H2-25, H2-27, H2-29, and H2-36 to H2-41, a deposit of gravelly sand to sand and gravel was encountered beneath the silt to sandy silt and/or the sand and silt to sand deposit. The surface of this deposit ranges between Elevation 235.9 m and 222.0 m, with thickness ranging between 0.3 m and 4.2 m where fully penetrated. A DCPT was advanced below the borehole termination depth in Borehole H2-24. Boreholes H2-36, H2-37, H2-40 and H2-41 were terminated within this deposit.

The SPT 'N'-values measured within the gravelly sand to sand and gravel deposit range from 8 blows to 31 blows per 0.3 m of penetration indicating a loose to dense relative density.

The grain size distributions of four samples of the gravelly sand to sand and gravel deposit are shown on Figure B18 in Appendix B.



The natural water content measured on six samples of the gravelly sand to sand and gravel deposit ranges from about 8 per cent to 23 per cent.

#### **4.6.6 Refusal**

Refusal to split-spoon, auger or casing advancement or dynamic cone penetration, indicating proximity to the inferred bedrock surface was encountered in Boreholes H2-22 to H2-29, H2-38 and H2-39 and in DCPTs H2-DC11 to H2-DC13 at depths ranging from 3.2 m to 18.6 m below the ground surface or between Elevation 240.3 m and 221.2 m. In DCPTs H2-DC15 to H2-DC21, the DCPTs terminated upon recording greater than 100 blows per 0.3 m of penetration, indicative of proximity to the inferred bedrock surface, at depths up to about 25 m below the existing ground surface. In DCPT H2-DC14, the rods were observed to be bent at the refusal depth.

#### **4.6.7 Groundwater Conditions**

In general, the samples taken in the boreholes were wet. Artesian groundwater levels were measured in Boreholes H2-27 to H2-31 with the groundwater level upon completion of drilling measured between 0.3 m and 1.5 m above ground surface (Elevation 241.2 m to 240.0 m). In the remaining boreholes, the groundwater levels observed upon completion of drilling range from Elevation 240.2 m to 239.9 m, typically measured at the ground surface to 1.7 m below ground surface. It should also be noted that the groundwater levels in the area fluctuate seasonally, as well as during precipitation events and snowmelt.

### **4.7 St. Pothier Road – STA 9+400 to 9+600 (High Fill H2)**

The plan and profiles along the centreline and toes of the proposed embankment of the new St. Pothier Road alignment showing the borehole locations and interpreted stratigraphy between STA 9+400 to STA 9+600 in the Township of Louise are shown on Drawings B1 to B6 in Appendix B. The alignment extends across a low-lying swampy area with the proposed embankment up to 4.4 m high above the existing ground. A total of seventeen (17) boreholes (Boreholes H2-44 to H2-60) and eight (8) DCPTs (DCPTs H2-DC22 to H2-DC29) were completed to investigate the subsurface conditions within this portion of High Fill Area H2.

The subsurface soils along the St. Pothier alignment in High Fill Area H2 consist of a surficial layer of peat underlain by a cohesive deposit of clayey silt transitioning into silty clay to clay transitioning to clayey silt, which in turn is underlain by deposits of silt to sand to sand and gravel.

#### **4.7.1 Peat**

A 0.5 m to 3.7 m thick deposit of black, fibrous to amorphous peat was encountered at the ground surface in Boreholes H2-44 to H2-60. The surface of the peat deposit ranges from Elevation 239.8 m to 239.5 m.

The SPT 'N'-values measured within the peat deposit typically range from 0 blows (weight of hammer) to 1 blow per 0.3 m of penetration suggesting a very soft consistency, however 'N'-values up to 6 blows per 0.3 m of penetration were noted through frozen peat in Borehole H2-55.



The natural water content measured on eighteen samples of the peat deposit ranges from about 100 per cent to 902 per cent.

#### **4.7.2 Silty Sand**

Beneath the peat deposit in Borehole H2-60, a 0.6 m thick layer of silty sand was encountered at Elevation 239.3 m. One SPT 'N'-value measured of 5 blows per 0.3 m of penetration indicates a loose relative density.

One natural water content measured in the silty sand deposit was about 42 per cent.

#### **4.7.3 Cohesive Deposit**

In all boreholes, a cohesive deposit was encountered beneath the peat or silty sand deposit. In general, the cohesive deposit consisted of an upper clayey silt to silt zone transitioning into a varved silty clay to clay deposit, further transitioning to clayey silt (in two boreholes). The surface of the cohesive deposit was encountered between Elevation 238.7 m and 235.8 m and the overall deposit ranged between 2.5 m and 10.1 m in thickness.

##### ***Clayey Silt***

Underlying the peat in Boreholes H2-44, H2-49 and H2-51 to H2-53, a deposit of grey, clayey silt was encountered. The top of the clayey silt was encountered between Elevation 236.2 m and 235.9 m and the thickness ranges from 0.7 m to 3.8 m.

The SPT 'N'-values measured within the clayey silt portion of the deposit typically range from 0 blows (weight of hammer) to 9 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency. Two in situ field vane test carried out within this deposit measured undrained shear strengths of about 65 kPa and 77 kPa and the sensitivity is calculated to be about 7 and 9. The field test results indicate that the deposit has a stiff consistency.

The grain size distribution of three samples of the clay deposit is shown on Figure B19 in Appendix B.

Atterberg limits tests were carried out on five samples of the clayey silt and the test results indicate liquid limits ranging from about 29 per cent to 34 per cent, plastic limits ranging from about 16 per cent to 25 per cent and plasticity indices ranging from about 4 per cent to 16 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B20 in Appendix B and indicate that the material is classified as a clayey silt of low plasticity.

The natural water content measured on six samples of the clayey silt deposit ranges from about 29 per cent to 36 per cent

##### ***Silty Clay to Clay***

A deposit of silty clay to clay was encountered, underlying the peat or clayey silt in Boreholes H2-45 to H2-60. In the majority of the boreholes, the silty clay to clay portion of the deposit was observed to be varved, consisting



of irregular layers of clayey silt/silty clay and silty clay/clay. The surface of the silty clay to clay was encountered between Elevation 238.7 m to 233.6 m, and ranged in thickness between 2.5 m and 9.8 m.

The SPT 'N'-values measured within the silty clay to clay deposit range from 0 blows (weight of hammer) to 11 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency. In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about 12 kPa to 69 kPa and the sensitivity is calculated to be between about 2 and 54 (typically less than 16). The field test results indicate that the deposit has a soft to stiff consistency.

The grain size distribution of six samples of the silty clay to clay deposit is shown on Figure B21 in Appendix B.

Atterberg limits tests were carried out on twenty-four samples of the silty clay to clay. The test results indicate liquid limits ranging from about 35 per cent to 64 per cent, plastic limits ranging from about 19 per cent to 25 per cent and plasticity indices ranging from about 12 per cent to 40 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figures B22 in Appendix B and indicate that the material is classified as silty clay of intermediate plasticity to clay of high plasticity.

The natural water content measured on twenty-eight samples of the silty clay to clay portion of the deposit ranges from about 30 per cent to 69 per cent.

### **Clayey Silt**

The silty clay to clay transitioned into grey, clayey silt at Elevation 230.8 m and 232.2 m in Boreholes H2-45 and H2-46, respectively. The deposit is between 3.6 m and 3.8 m in thickness.

The SPT 'N'-values measured within the clayey silt deposit are 1 and 5 blows per 0.3 m of penetration, suggesting a very soft to firm consistency. In situ field vane tests carried out within this deposit measured undrained shear strengths of about 21 kPa and 37 kPa and the sensitivity is calculated to be about 4 and 20. The field test results indicate that the deposit has a soft to firm consistency.

A grain size distribution for one sample of the clayey silt is shown on Figure B19 in Appendix B.

Atterberg limits tests were carried out on two samples of the clayey silt. The test results indicate liquid limits of about 29 per cent and 33 per cent, plastic limits of about 22 per cent and 24 per cent and plasticity indices of about 7 per cent and 10 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B20 in Appendix B and indicate that the material is classified as a clayey silt of low plasticity.

The natural water content measured on two samples of this portion of the deposit are about 31 per cent and 45 per cent.

#### **4.7.4 Silt to Sandy Silt**

A deposit of grey silt to sandy silt was encountered underlying the cohesive deposit in Boreholes H2-45 and H2-7 to H2-60. The surface of the silt to sandy silt deposit was encountered between Elevation 236.2 m and 226.0 m, and the thickness ranged from 0.6 m to 11.6 m. Boreholes H2-50 and H2-54 were terminated within this deposit.



The SPT 'N'-values measured within the silt to sandy silt deposit range from 1 blow to 13 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The grain size distributions of twenty samples of the silt to sandy silt deposit are shown on Figure B23 in Appendix B. The results of Atterberg limits testing on ten samples of the silt to sandy silt deposit indicated that the material is classified as non-plastic.

The natural water content measured on twenty-one samples of the silt to sandy silt deposit ranges from about 23 per cent to 41 per cent.

In Borehole H2-50, a 3.1 m thick interlayer of silty sand to sand was encountered within the sandy silt to silt deposit at Elevation 220.2 m. The natural water content measured on one sample is about 27 per cent.

#### **4.7.5 Sand and Silt to Sand**

A deposit of sand and silt to sand was encountered underlying the silt to sandy silt deposit in Boreholes H2-48, H2-49, H2-51 to H2-53 and H2-56 to H2-60 and underlying the cohesive deposit in Borehole H2-46. The surface of the sand and silt to sand deposit was encountered between Elevations 234.2 m and 218.7 m, with the thickness ranging from 0.8 m to 6.1 m, where it was fully penetrated. Boreholes H2-49, H2-51, H2-53, H2-56 and H2-60 were terminated within this deposit.

The SPT 'N'-values measured within the sand and silt to sand deposit range from 1 blow to 70 blows per 0.3 m of penetration, indicating a very loose to very dense relative density. Typically, the values were less than 18 blows indicating that generally, the deposit is very loose to compact. In four samples, the split spoon did not penetrate the full sample depth and these higher values are likely associated with the underlying refusal conditions.

The grain size distributions of eight samples of the sand and silt to sand deposit are shown on Figure B24 in Appendix B.

The natural water content measured on eight samples of the sand and silt to sand deposit ranges from about 15 per cent to 26 per cent.

#### **4.7.6 Sand and Gravel**

In Boreholes H2-44, H2-45, H2-52, H2-55 and H2-58, a deposit of sand and gravel was encountered underlying the silt to sandy silt or the sand and silt to sand deposits. The surface of this deposit was encountered at Elevation 232.4 m and 217.7 m, with thicknesses ranging between 0.1 m and 1.8 m where the deposit was fully penetrated. Boreholes H2-52, H2-55 and H2-58 were terminated within this deposit.

The SPT 'N'-values measured within the gravelly sand to sandy gravel deposit range from 14 blows to 30 blows per 0.3 m of penetration which indicates a compact to dense relative density.

#### **4.7.7 Refusal**

Refusal to split-spoon, auger or casing advancement or dynamic cone penetration, indicating proximity to the inferred bedrock surface was encountered in Boreholes H2-44 to H2-48, H2-57 and H2-59 and in DCPTs

H2-DC22 and H2-DC23 at depths ranging from 8.3 m to 20.2 m below the ground surface or between Elevation 231.3 m and 219.3 m. In DCPTs H2-DC25 and H2-DC27 to H2-DC29, the DCPTs terminated upon recording greater than 100 blows per 0.3 m of penetration, indicative of proximity to the inferred bedrock surface, at depths up to about 24 m below the existing ground surface.

#### **4.7.8 Groundwater Conditions**

In general, the samples taken in the boreholes were wet. The groundwater levels observed upon completion of drilling range from about Elevation 239.8 m to 238.6 m, typically measured at the ground surface up to 0.9 m below ground surface. It should also be noted that the groundwater levels in the area fluctuate seasonally, as well as during precipitation events and snowmelt.

### **4.8 Highway 17 WBL – STA 13+900 to 14+200 (High Fill H3)**

The plan and profiles along the centreline and toes of the proposed embankment of the new Highway 17 WBL alignment showing the borehole locations and interpreted stratigraphy between about STA 13+900 to STA 14+200 in the Township of Denison are shown on Drawings C1 to C4 in Appendix C. The alignment extends across a low-lying swampy area with the proposed embankment up to 3.8 m high above the existing ground. In this area, the existing four-lane highway transitions into a two-lane undivided highway from east to west. A total of thirteen (13) boreholes (Boreholes H3-1 to H3-13) and six (6) DCPTs (DCPTs H3-DC1 to H3-DC6) were completed to investigate the subsurface conditions within this portion of High Fill Area H3.

The subsurface soils along the WBL alignment in High Fill Area H3 consist of surficial layers of peat/topsoil or asphalt and embankment fill, underlain by an upper deposit of sand to sandy silt. These upper deposits are underlain by the main cohesive deposit of clayey silt transitioning into varved silty clay to clay transitioning to clayey silt to silt underlain by a silt deposit, which further is underlain by deposits of sand to sand and silt.

#### **4.8.1 Asphalt**

In Boreholes H3-1, H3-3 to H3-5, H3-8 and H3-13 between 100 mm and 370 mm of asphalt was encountered at ground surface.

#### **4.8.2 Fill**

Embankment fill was encountered underlying the asphalt in Boreholes H3-1, H3-3 to H3-5, H3-8 and H3-13, from ground surface in Boreholes H3-7, H3-9 and H3-11, and beneath the peat/topsoil in Boreholes H3-2, H3-6, H3-10 and H3-12. The embankment fill was comprised of varying layers of sandy gravel to gravelly sand, silty sand to silt, in some areas clayey silt to silty clay, and sandy gravel to sand. The embankment fill was encountered between Elevation 247.8 m and 240.9 m, and the total thickness ranges between 0.9 m and 6.0 m. In Boreholes H3-1, H3-7, H3-9, H3-11 and H3-13, the clayey silt to silty clay fill was between 0.5 m and 1.3 m thick.

The SPT 'N'-values measured within the clayey silt to silty clay portion of the fill range from 8 blows to 29 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency. The SPT 'N'-values measured within the





sand, sandy gravel to gravelly sand and sandy silt to silt fill range from 2 blows to 94 blows per 0.3 m of penetration, indicating a very loose to very dense relative density. Some instances within this portion of the fill, the split-spoon sampler did not penetrate the full sample depth due to inferred blast rock fill fragments within the fill and rock coring techniques were required to advance the borehole through these zones.

The grain size distributions of five samples of the fill deposit are shown on Figure C1 in Appendix C.

The natural water content measured on ten samples of the sand, sandy gravel to gravelly sand and sandy silt to silt fill range from about 2 per cent to 25 per cent.

#### **4.8.3 Peat/Topsoil**

In Boreholes H3-6 and H3-10 a deposit of black fibrous peat was encountered from ground surface. In Boreholes H3-7 and H3-11, the peat was encountered beneath the embankment fill. In Boreholes H3-2 and H3-12 a surficial layer of topsoil was encountered at ground surface. The top of the peat/topsoil layer varies from Elevation 243.5 m to 239.2 m, and the thickness ranges from 0.1 m to 1.9 m.

The natural water content measured on one sample of the peat is about 46 per cent.

#### **4.8.4 Sand to Sandy Silt**

A deposit consisting of sand to sandy silt was encountered underlying the fill deposit in Boreholes H3-1, H3-3, H3-4, H3-7 and H3-10. The surface of the sand to sandy silt deposit was encountered between Elevation 242.6 m and 239.1 m, and the thickness ranges from 0.6 m to 1.6 m.

The SPT 'N'-values measured within the sand to sandy silt deposit range from 1 blow to 19 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

A grain size distribution for one sample of the sand to sandy silt deposit is shown on Figure C2 in Appendix C.

The natural water content measured on two samples of the sand to sandy silt deposit are about 20 per cent.

#### **4.8.5 Cohesive Deposit**

In all boreholes a cohesive deposit was encountered beneath the peat/topsoil, embankment fill or sand to sandy silt deposits. In some cases the cohesive deposit consisted of an upper clayey silt zone transitioning into a varved silty clay to clay zone further transitioning into a lower clayey silt to silt zone in some areas. The surface of the cohesive deposit was encountered between Elevation 241.0 m and 238.5 m and the overall deposit ranged between 4.7 m and 14.1 m in thickness.

#### **Clayey Silt**

In Boreholes H3-2, H3-4, H3-5 and H3-8 to H3-13, the clayey silt portion of the deposit containing trace sand, trace organics was encountered underlying the peat/topsoil, embankment fill or sand to sandy silt deposits. The surface of this portion of the deposit was encountered between Elevation 240.3 m and 238.8 m and ranges from 0.9 m to 3.4 m in thickness.

The SPT 'N'-values measured within the clayey silt deposit range from 0 blows (weight of hammer) to 8 blows per 0.3 m of penetration, suggesting a very soft to firm consistency. In situ field vane tests carried out within this portion of the deposit measured undrained shear strengths ranging from about 25 kPa to 58 kPa (with one field vane noted to be greater than 100 kPa in Borehole H3-10) and the sensitivity is calculated to be between about 3 and 8. The field test results indicate that this portion of the deposit has a firm to stiff consistency.

Atterberg limits tests were carried out on two samples of the clayey silt. The test results indicate liquid limits ranging from about 33 per cent to 34 per cent, plastic limits of about 18 per cent and plasticity indices of about 15 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure C3 in Appendix C and indicate that the material is classified as a clayey silt of low plasticity.

The natural water content measured on four samples of this portion of the deposit ranges from about 35 per cent to 45 per cent.

### ***Silty Clay to Clay***

In all boreholes, a deposit of grey, silty clay to clay was encountered underlying the peat and/or silt to sandy silt deposits or the upper clayey silt portion of the deposit. In the majority of the boreholes, the silty clay to clay portion of the deposit was observed to be varved, consisting of irregular layers of clayey silt/silty clay and silty clay/clay. The top of the silty clay to clay portion of the deposit was encountered between Elevation 241.0 m and 236.6 m and ranges from 4.6 m to 10.9 m in thickness. Borehole H3-9 was terminated within this deposit, likely on an obstruction.

The SPT 'N'-values measured within this portion of the deposit range between 0 blows (weight of hammer) and 11 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency. It was noted that the 11 blows per 0.3 m was likely due to gravel caved in the borehole. In situ field vane tests carried out within this portion of the deposit measured undrained shear strengths ranging from about 18 kPa to 80 kPa and the sensitivity is calculated to range from about 2 to 12. The field vane tests results indicate that the silty clay to clay has a soft to stiff consistency. Typically, the undrained shear strengths measured were less than 50 kPa suggesting the deposit is generally soft to firm.

The grain size distributions of three samples of the silty clay to clay portion of the deposit are shown on Figure C4 in Appendix C.

Atterberg limits tests were carried out on eighteen samples of the silty clay to clay portion of the deposit. The test results indicate liquid limits ranging from about 35 per cent to 71 per cent, plastic limits ranging from about 17 per cent to 24 per cent and plasticity indices ranging from about 15 per cent to 47 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure C5 in Appendix C and indicate that the material is classified as silty clay of intermediate plasticity to clay of high plasticity.

The natural water content measured on twenty-two samples of this portion of the deposit ranges between about 31 per cent and 72 per cent.

A laboratory consolidation (oedometer) test was carried out on one sample of the silty clay to clay portion of the deposit obtained from a Shelby tube sample in Borehole H3-12. The preconsolidation stress was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of 18.1 kN/m<sup>3</sup> and a specific gravity of 2.78 was measured on the consolidation test sample. The

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detailed results of the oedometer test is shown on Figure C6 in Appendix C, and the test result is summarized below, and in Table 2.

| Borehole/<br>Sample No. | Sample<br>Depth /<br>Elevation | $\sigma_{vo}'$<br>(kPa) | $\sigma_p'$<br>(kPa) | $\sigma_p' - \sigma_{vo}'$<br>(kPa) | OCR | $e_o$ | $C_c$ | $C_r$ | $c_v^*$<br>(cm <sup>2</sup> /s) |
|-------------------------|--------------------------------|-------------------------|----------------------|-------------------------------------|-----|-------|-------|-------|---------------------------------|
| H3-12/Sa 7              | 11.0 m /<br>231.4 m            | 97                      | 129                  | 32                                  | 1.3 | 1.13  | 0.41  | 0.02  | $1.3 \times 10^{-3}$            |

\*For the normally consolidated stress range.

where:  $\sigma_{vo}'$  is the effective overburden stress in kPa  
 $\sigma_p'$  is the preconsolidation stress in kPa  
OCR is the overconsolidation ratio  
 $e_o$  is the initial void ratio  
 $C_c$  is the compression index  
 $C_r$  is the recompression index  
 $c_v$  is the coefficient of consolidation in cm<sup>2</sup>/s

### Clayey Silt to Silt

In Boreholes H3-4, H3-6 to H3-8 and H3-13 the cohesive deposit transitions into grey clayey silt to silt, trace sand. The surface of this portion of the deposit was encountered between Elevation 232.7 m and 227.6 m and ranges from 0.9 m to 2.2 m in thickness.

The SPT 'N'-values measured within the clayey silt to silt portion of the deposit range from 0 blows (weight of hammer) to 9 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency. In situ field vane tests carried out within this portion of the deposit measured undrained shear strengths ranging from about 35 kPa to 58 kPa and the sensitivity is calculated to be between about 1 and 5. The field test results indicate that this portion of the deposit has a firm to stiff consistency.

The grain size distribution of one sample of the clayey silt to silt is shown on Figure C7 in Appendix C.

Atterberg limits tests were carried out on a sample of the clayey silt to silt. The test result indicates a liquid limit of about 25 per cent, a plastic limit of about 19 per cent and a plasticity index of about 5 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure C8 in Appendix C and indicate that the material is classified as a clayey silt to silt of slight plasticity.

The natural water content measured on two samples of this portion of the deposit ranges from about 29 per cent to 31 per cent.

### 4.8.6 Silt to Sandy Silt

A deposit of grey, wet, silt to sandy silt was encountered underlying the cohesive deposit in all boreholes except Borehole H3-9. The surface of the silt to sandy silt deposit was encountered between Elevation 236.3 m and 225.9 m and ranges from 1.4 m to 7.0 m in thickness, where the deposit was fully penetrated. Boreholes H3-4 to H3-6 and H3-13 were terminated within this deposit.

The SPT 'N'-values measured within the silt to sandy silt deposit range between 0 blows (weight of hammer) and 29 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The grain size distributions of six samples of the silt to sandy silt are shown on Figure C9 in Appendix C.



The results of Atterberg limits testing on two samples indicated that the material is classified as non-plastic.

The natural water content measured on six samples of this deposit range between about 29 per cent and 32 per cent.

### ***Sand to Sand and Silt (Interlayer)***

In Boreholes H3-3 to H3-6, an interlayer comprised of sand to sand and silt was encountered within the silt to sandy silt deposit. The surface of the sand to sand and silt interlayer was encountered between Elevation 231.4 m and 228.0 m and ranges from 1.5 m to 2.3 m in thickness.

The SPT 'N'-values measured within this portion of the deposit range between 2 and 41 blows per 0.3 m of penetration, indicating a very loose to dense relative density.

The grain size distributions of three samples of the sand to sand and silt interlayer are shown on Figure C10 in Appendix C.

The natural water content measured on three samples of this deposit range between about 24 per cent and 27 per cent.

### **4.8.7 Sand to Sand and Silt**

Underlying the cohesive deposit and/or the silt to sandy silt deposit in Boreholes H3-1 to H3-3 H3-7, H3-8 and H3-10 to H3-12, a deposit of sand to gravelly sand was encountered. The top of the sand to sand and silt deposit was encountered between Elevation 234.9 m and 219.2 m. These boreholes were terminated within this deposit.

The SPT 'N'-values measured in this deposit range between 13 and 33 blows per 0.3 m of penetration, indicating a compact to dense relative density. In one instance the split-spoon did not penetrate the full sample depth indicating proximity to a very dense stratum or inferred bedrock.

The grain size distributions of three samples of the gravelly sand to sand and silt are shown on Figure C11 in Appendix C.

The natural water content measured on four samples of this deposit range between about 11 per cent and 24 per cent.

### **4.8.8 Refusal**

Refusal to split-spoon and dynamic cone penetration, indicating proximity to the inferred bedrock surface was encountered in Borehole H3-1 and in DCPTs H3-DC1 and H3-DC2 at depths ranging from 12.4 m and 20.8 m below the ground surface or between Elevation 232.9 m and 222.1 m. In DCPTs H3-DC3 to H3-DC6, the DCPTs terminated upon recording greater than 100 blows per 0.3 m of penetration, indicative of proximity to the inferred bedrock surface, at depths up to about 28 m below the existing ground surface.

In Borehole H3-9, split-spoon refusal was encountered at 16.0 m depth (Elevation 227.9 m) within the silty clay to clay stratum. In this instance, refusal is likely indicative of the presence of an obstruction.

#### 4.8.9 Groundwater

In general, the samples taken in the boreholes were moist to wet. The groundwater levels observed upon completion of drilling range from Elevation 242.9 m to 240.7 m, typically measured from 0.5 m to 6.0 m below ground surface. It should also be noted that the groundwater levels in the area fluctuate seasonally as well as during precipitation events and snowmelt.

### 4.9 Highway 17 EBL – STA 13+900 to 14+200 (High Fill H3)

The plan and profiles along the centreline and toes of the proposed embankment of the new Highway 17 EBL alignment showing the borehole locations and interpreted stratigraphy between about STA 13+900 to STA 14+200 in the Township of Denison are shown on Drawings C1 to C4 in Appendix C. The alignment extends across a low-lying swampy area with the proposed embankment up to 3.8 m high above the existing ground. In this area, the existing four-lane highway transitions into a two-lane undivided highway from east to west. A total of thirteen (13) boreholes (Boreholes H3-14 to H3-26 inclusive) and six (6) DCPTs (DCPTs H3-DC7 to H2-DC12) were completed to investigate the subsurface conditions within this portion of High Fill Area H3.

The subsurface soils along the EBL alignment in High Fill Area H3 consist of surficial layers of peat/topsoil or asphalt and embankment fill, underlain by an upper deposit of sand to sandy silt. These upper deposits are underlain by the main cohesive deposit of clayey silt transitioning into varved silty clay to clay transitioning to clayey silt to silt underlain by a silt to sandy silt deposit, which further is underlain by deposits of sand to sand and silt.

#### 4.9.1 Asphalt

In Boreholes H3-14, H3-16, H3-18, H3-20, H3-22, H3-24 and H3-26, between 50 mm and 350 mm of asphalt was encountered at ground surface.

#### 4.9.2 Fill

Embankment fill was encountered in all boreholes except Borehole H3-19. The fill was encountered underlying the asphalt in Boreholes H3-14, H3-16, H3-18, H3-20, H3-22, H3-24 and H3-26, from ground surface in Boreholes H3-17 and H3-21, and beneath the peat/topsoil in Boreholes H3-15, H3-23 and H3-25. The embankment fill was comprised of varying layers of silty sand to gravelly sand, silty sand to silt, in some areas clayey silt to silty clay and sand to sand and gravel. The surface of the fill stratum was encountered between Elevation 247.8 m and 241.4 m, and the total thickness ranges between 1.5 m and 7.2 m. In Boreholes H3-16 to H3-18, H3-20, H3-21, H3-24 and H3-26, the clayey silt to silty clay fill was between 0.9 m and 2.9 m thick.

The SPT 'N'-values measured within the clayey silt to silty clay portion of the fill range from 0 blows (weight of hammer) to 21 blows per 0.3 m of penetration, indicating a soft to very stiff consistency. The SPT 'N'-values measured within the silt to silty sand to sand and gravel fill stratum range from 1 blow to 66 blows per 0.3 m of penetration, indicating a very loose to very dense relative density. For one sample interval within the lower sand to sand and gravel portion of the fill stratum, the split-spoon sampler did not penetrate the full sample depth due





to blast rock fill fragments within the fill and rock coring techniques were required to advance the borehole through these zones.

The grain size distributions of eight samples of the fill deposit are shown on Figure C12 in Appendix C.

An Atterberg limits test was carried out on a sample of the silty clay. The test results indicate a liquid limit of about 42 per cent, a plastic limit of about 18 per cent and a plasticity index of about 24 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure C13 in Appendix C and indicate that the fill material is classified as silty clay of intermediate plasticity.

The natural water content measured on eight samples of the silt to silty sand to sand and gravel fill range from about 4 per cent to 22 per cent. The natural water content measured on one sample of the silty clay fill is about 21 per cent.

#### **4.9.3 Peat/Topsoil**

In Boreholes H3-15, H3-19 and H3-23 a deposit of black fibrous peat was encountered from ground surface. In Borehole H3-25 a surficial layer of topsoil was encountered at ground surface. The surface of the peat/topsoil layer varies in Elevation from 243.1 m to 241.8 m, and the thickness ranges from about 0.3 m to 2.0 m.

The SPT 'N'-values measured within the peat deposit range from 0 blows (weight of hammer) to 3 blows per 0.3 m of penetration, indicating a very soft to soft consistency.

The natural water content measured on one sample of the peat is about 390 per cent.

#### **4.9.4 Sand to Sandy Silt**

A deposit of sand to sandy silt was encountered underlying the fill deposit in Boreholes H3-14, H3-16 and H3-18. The surface of the deposit was encountered between Elevation 244.0 m and 239.7 m, and the thickness ranges from 1.7 m to 3.1 m.

The SPT 'N'-values measured within the sand to sandy silt deposit range from 7 blows to 10 blows per 0.3 m of penetration, indicating a loose to compact relative density.

A grain size distribution of one sample of the silty sand deposit is shown on Figure C14 in Appendix C.

The natural water content measured on two samples of the sand to sandy silt deposit are about 18 per cent and 23 per cent.

#### **4.9.5 Cohesive Deposit**

In all boreholes a cohesive deposit was encountered beneath the peat/topsoil, embankment fill or sand to sandy silt deposits. In some cases the cohesive deposit consisted of an upper clayey silt zone transitioning into a varved silty clay to clay zone further transitioning into a lower clayey silt to silt in some areas. The top of the cohesive deposit was encountered between Elevation 240.9 m and 238.0 m and the overall deposit ranged between 4.4 m and 16.8 m in thickness.

### ***Clayey Silt***

In Boreholes H3-20, H3-21 and H3-23 a deposit of grey clayey silt trace sand was encountered underlying the embankment fill. The surface of this portion of the deposit was encountered between Elevation 239.7 m and 238.8 m and ranged from 0.8 m to 1.8 m in thickness.

The SPT 'N'-values measured within the clayey silt deposit range from 4 blows to 7 blows per 0.3 m of penetration, indicating a firm consistency.

### ***Silty Clay to Clay***

In all boreholes a deposit of grey, silty clay to clay was encountered underlying the peat, embankment fill, sand to sandy silt or the upper clayey silt portion of the cohesive deposit. In the majority of the boreholes, the silty clay to clay portion of the deposit was observed to be varved, consisting of irregular layers of clayey silt/silty clay and silty clay/clay. The surface of the silty clay to clay portion of the deposit was encountered between Elevation 240.9 m and 237.1 m and ranged from 3.0 m to 15.2 m in thickness. Borehole H3-23 was terminated within this deposit, likely on an obstruction.

The SPT 'N'-values measured within this portion of the deposit range between 0 blows (weight of hammer) and 8 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency. In situ field vane tests carried out within this portion of the deposit measured undrained shear strengths ranging from about 14 kPa to 84 kPa and the sensitivity is calculated to range from about 1 to 17, typically less than 9. The field vane test results indicate that the silty clay to clay has a soft to stiff consistency. Typically, the undrained shear strengths measured were less than 50 kPa suggesting the deposit is generally soft to firm.

A grain size distribution for one sample of the silty clay to clay portion of the deposit is shown on Figure C15 in Appendix C.

Atterberg limits tests were carried out on eighteen samples of the silty clay to clay portion of the deposit. The test results indicate liquid limits ranging from about 36 per cent to 70 per cent, plastic limits ranging from about 19 per cent to 25 per cent and plasticity indices ranging from about 16 per cent to 45 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure C16 in Appendix C and indicate that the material is classified as silty clay of intermediate plasticity to clay of high plasticity.

The natural water content measured on twenty samples of this portion of the deposit ranges between about 19 per cent and 70 per cent.

Laboratory consolidation (oedometer) tests was carried out on one sample of the silty clay to clay portion, obtained from a Shelby tube sample in Borehole H3-24. The preconsolidation stress was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of about 18.4 kN/m<sup>3</sup> and a specific gravity of about 2.76 were measured on the consolidation test sample. The detailed results of the oedometer test are shown on Figure C17 in Appendix C, and the test result is summarized below, and in Table 2.

**FOUNDATION REPORT – FOUR-LANING HIGHWAY EXTENSION  
HIGH FILL EMBANKMENTS OVER SWAMPS GWP 156-98-00**

| Borehole/<br>Sample No. | Sample<br>Depth /<br>Elevation | $\sigma_{vo}'$<br>(kPa) | $\sigma_p'$<br>(kPa) | $\sigma_p' - \sigma_{vo}'$<br>(kPa) | OCR | $e_o$ | $C_c$ | $C_r$ | $c_v^*$<br>(cm <sup>2</sup> /s) |
|-------------------------|--------------------------------|-------------------------|----------------------|-------------------------------------|-----|-------|-------|-------|---------------------------------|
| H3-24/Sa 9              | 12.5 m /<br>231.3 m            | 149                     | 149                  | 0                                   | 1.0 | 1.14  | 0.36  | 0.05  | $1.2 \times 10^{-3}$            |

\*For the normally consolidated stress range.

where:  $\sigma_{vo}'$  is the effective overburden stress in kPa  
 $\sigma_p'$  is the preconsolidation stress in kPa  
OCR is the overconsolidation ratio  
 $e_o$  is the initial void ratio  
 $C_c$  is the compression index  
 $C_r$  is the recompression index  
 $c_v$  is the coefficient of consolidation in cm<sup>2</sup>/s

### Clayey Silt to Silt

In Boreholes H3-14, H3-19, H3-21, H3-22, H3-25 and H3-26, the cohesive deposit transitions into grey clayey silt to silt, trace sand. The surface of this portion of the deposit was encountered between Elevation 237.9 m and 224.4 m and ranged from 0.9 m to 1.7 m in thickness.

The SPT 'N'-values measured within the clayey silt to silt deposit range from 0 blows (weight of hammer) to 6 blows per 0.3 m of penetration, suggesting a very soft to firm consistency. In situ field vane tests carried out within this portion of the deposit measured undrained shear strengths ranging from about 36 kPa to 65 kPa and the sensitivity is calculated to be between about 2 and 4. The field test results indicate that this portion of the deposit has a firm to stiff consistency.

The grain size distribution of one sample of the clayey silt to silt is shown on Figure C18 in Appendix C.

Atterberg limits tests were carried out on three samples of the clayey silt to silt portion of the deposit. The test results indicate liquid limits ranging from about 28 per cent to 34 per cent, plastic limits ranging from about 19 per cent to 21 per cent and plasticity indices ranging from about 8 per cent to 14 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure C19 in Appendix C and indicate that the material is classified as a clayey silt of low plasticity.

The natural water content measured on four samples of this portion of the deposit ranges from about 30 per cent to 39 per cent.

### 4.9.6 Silt to Sandy Silt

A grey, wet, silt to sandy silt deposit was encountered underlying the cohesive deposit in all Boreholes except H3-14 and H3-23. The surface of the silt to sandy silt deposit was encountered between Elevation 235.9 m and 222.8 m and ranged from 1.6 m to 6.6 m in thickness, where the deposit was fully penetrated. A DCPT was advanced below the borehole termination depth in Borehole H3-13. Boreholes H3-16, H3-18 and H3-25 were terminated within this deposit.

The SPT 'N'-values measured within this portion of the deposit range between 2 blows to 26 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The grain size distributions of ten samples of the silt to sandy silt are shown on Figure C20 in Appendix C.



The results of Atterberg limits testing on one sample indicated that the material is classified as non-plastic.

The natural water content measured on ten samples of this deposit range between about 29 per cent and 33 per cent.

### ***Sand (Interlayer)***

In Borehole H3-18 a 1.5 m thick sand interlayer was encountered within the silt to sandy silt deposit. The top of the sand interlayer was encountered at Elevation 228.9 m.

One SPT 'N'-value measured within the sand interlayer was 9 blows per 0.3 m of penetration, indicating a loose relative density.

### **4.9.7 Sand to Sand and Silt**

Underlying the cohesive deposit or the silt to sandy silt deposit in Boreholes H3-14, H3-15, H3-17, H3-19 to H3-22, H3-24 and H3-26, a deposit of sand to sand and silt was encountered. The surface of the sand to sand and silt deposit was encountered between Elevation 236.3 m and 220.1 m. All boreholes were terminated within this deposit after penetrating a minimum of 1.1 m. The last sample in Borehole H3-15 was observed to comprise of gravelly sand.

The SPT 'N'-values measured within this portion of the deposit range between 7 and 26 blows per 0.3 m of penetration, indicating a loose to compact relative density. In Borehole H3-15, the gravelly sample measured an SPT 'N'-value of 66 blows per 0.3 m of penetration, indicating a very dense relative density.

The grain size distributions of two samples of the sand to sand and silt deposit are shown on Figure C21 in Appendix C.

The natural water content measured on two samples of this deposit are about 17 per cent and 25 per cent.

### **4.9.8 Refusal**

Refusal to split-spoon and dynamic cone penetration, indicating proximity to the inferred bedrock surface was encountered in Boreholes H3-18 and H3-21 and in DCPTs H3-DC7 to H3-DC9 at depths ranging from about 12.3 m and 25.0 m below the ground surface or between Elevation 231.9 m and 219.4 m. In DCPT H3-DC12, the DCPT terminated upon recording greater than 100 blows per 0.3 m of penetration, indicative of proximity to the inferred bedrock surface, at depths up to about 26 m below the existing ground surface.

In Borehole H3-23, split-spoon refusal was encountered at 15.6 m depth (Elevation 226.3 m) within the silty clay to clay stratum. In this instance, refusal is likely indicative of the presence of an obstruction.

### **4.9.9 Groundwater**

In general, the samples taken in the boreholes were moist to wet. The groundwater levels observed upon completion of drilling range from Elevation 243.8 m to 238.7 m, typically measured from 1.6 m to 6.0 m below ground surface. In Boreholes H3-15 and H3-19, 0.2 m of ponded water was encountered at the ground surface.



It should also be noted that the groundwater levels in the area fluctuate seasonally as well as during precipitation events and snowmelt.

## **5.0 CLOSURE**

The field personnel supervising the drilling program were Messrs. Shane Albert, Gabriel Mathieu, Ed Savard, Indulis Dumpis, Trevor Moxam, Lubo Kosci and Adam Core, under the direction of Mr. Evan Childerhose, P.Eng. This report was prepared by Mr. Adam Core, E.I.T. and the technical aspects were reviewed by Ms. Sarah Poot, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M.A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal of Golder, conducted an independent quality control review of the report.





## Report Signature Page

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# **PART B**

## **FOUNDATION DESIGN REPORT**

### **HIGH FILL EMBANKMENTS OVER SWAMPS**

#### **HIGHWAY 17 FOUR-LANING EXTENSION FROM 20.5 KM**

#### **WEST OF HIGHWAY 144, EASTERLY 6.5 KM**

#### **MINISTRY OF TRANSPORTATION, ONTARIO**

#### **GWP 156-98-00**



## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides an interpretation of the geotechnical data obtained during the subsurface investigation and recommendations on the foundation aspects of the design of the proposed works. The recommendations provided are intended for the guidance of the design engineer. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

### 6.1 General

Golder Associates Ltd. (Golder) has been retained by D. M. Wills Ltd. (DMW) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for high fill embankments crossing over swamp areas located within the limits of the new Highway 17 alignment. The proposed high fill embankments outlined in the project limits are part of the Highway 17 new interchange and extension of the existing four-laning at the West Junction of Sudbury Municipal Road 55, from 20.5 km west of Highway 144, easterly for 6.5 km. As part of this work, foundation recommendations are required for the high fill embankments over swamp crossings (about 1.1 km in total length). Table 1 summarizes the locations of the areas investigated within the project limits that require foundation design for the new Highway 17 alignment and maximum proposed embankment height for that section of swamp crossing.

This report presents the results of embankment stability and settlement analyses and provides recommendations for stable embankment geometry and embankment fill materials, and implementation of mitigation alternatives that may be required as a means to improve stability (if necessary) and reduce post-construction settlements. The report also addresses potential construction concerns and geotechnical problems associated with embankment construction, sub-excavation of soft/organic materials and placement of new fill materials.

### 6.2 High Fill Embankments and Embankments Over Swamps

Based on the vertical profiles of the proposed Highway 17 and St. Pothier road alignment provided to Golder by DMW, the new highway/road crossings over high fill/swamp areas will require fill embankments ranging in height from about 2 m to about 5.5 m.

Sections 6.2.2 and 6.2.3 of this report summarize the methods used to analyze the stability and settlement for critical sections of high fill embankment/swamp crossing construction for the new four-lane extension of Highway 17. Section 6.4 provides discussions of potential alternatives for mitigating embankment stability and settlement and related design and construction issues. The embankment height and location of the critical embankment stability sections and recommendations for mitigating stability and time-dependent settlements for each individual high fill/swamp crossing area, where applicable, are presented in Section 6.5. General aspects of subgrade preparation and embankment construction are presented in Section 6.6.

At all high fill/swamp crossing areas, the analyses assume that organic materials (i.e., peat, topsoil and soils with organics) will be removed prior to construction of the new and widened embankments (as discussed in Section 6.6.1). The thickness of organic deposits at each high fill/swamp crossing area and the soil parameters



employed in the stability and settlement analyses are presented in Table 3. The piezometric conditions required in the analyses are based on the groundwater levels noted during drilling, which were generally measured at or near the level of the natural ground surface at most borehole locations.

The analyses also assume the profile grades provided by DMW. Should the final grade be changed after issuing this report, Golder should be contacted to review the recommendations contained herein. We further understand that the median of the twinned embankments is likely to be filled with rock fill to some level and this has also been assumed in our analyses.

### **6.2.1 Embankment Fill Types and Berm Requirements**

Different embankment fill alternatives (i.e., rock fill and granular fill) provide relative advantages and disadvantages in terms of availability, weight (i.e., driving force and applied load to the founding deposit), construction cost and time, ease of construction and post-construction performance.

We understand that rock fill is the preferred embankment fill material for this project due to its availability from rock blasting for road cuts required elsewhere on the project. In this regard, the stability and settlement analyses discussed in Section 6.5 have been carried out assuming the new highway embankments will generally be constructed of rock fill. Where the existing embankments are to be widened, granular fill has also been considered in the analyses.

#### ***Rock Fill***

The advantages of constructing new embankments using rock fill include the ability to achieve steeper side slopes (1.25H:1V), which is required in areas with limited right-of-way, as well as reducing the overall quantity of fill material required for the project, and the material can be placed in sub-excavated areas under water. Rock fill will also be available locally, either from excavations in deep cuts through bedrock outcrops within this and other phases of the project alignment or from rock borrow areas close to the project limits. The disadvantage of using rock fill for the construction of embankments is that some post-construction settlement of the embankment fill itself will occur. Settlement of the rock fill is discussed further in Section 6.2.3.3.

In accordance with MTO Northern Region Pavement Practices and Guidelines (1997) as amended by MTO Memorandum "Use of Mid-Slope Berms for Rock Fill Embankments" dated February 8, 2005, 2 m wide berms should be incorporated into the rock fill embankment side slope profile for uninterrupted slopes greater than 10 m high. Given that none of the proposed new embankments are greater than 5.7 m high, the 2 m wide mid-slope berms will not be required within these project limits.

#### ***Granular Fill***

The main advantages of using granular fill for embankment construction are the ease of construction and negligible post-construction settlement within the embankment fill itself. However, this option will require a larger volume of fill and potentially wider right-of-way because the side slopes of granular fill embankments (2H:1V) are flatter than those of rock fill embankments. For this project, acceptable granular fill is considered to be well graded, locally available and/or imported granular material. The use of granular fill may be preferred in areas of embankment widening where rock fill may be difficult to place.





Granular fill would also need to be used as backfill below the ground surface, after removal of the organic deposits and near surface cohesive soils, to allow for installation of wick drains should they be considered the preferred settlement mitigation alternative.

### 6.2.2 Stability

The following report sections outline the methodology used to evaluate embankment stability at the various high fill/swamp crossing areas and also present the parameters used in the analyses for each of the critical section(s). The results of the stability analyses for each high fill/swamp crossing area, the results of the settlement analyses, and recommendations regarding possible design and construction alternatives to mitigate stability issues and/or post-construction settlement are presented in Section 6.5.

#### 6.2.2.1 Methodology

Stability analyses were carried out for the critical sections of the proposed fill embankments in each high fill/swamp crossing area. Critical sections correspond to the greatest new embankment height and/or the maximum thickness of soft, compressible cohesive soils and/or the largest amount of peat/topsoil/organic sub-excavation. Generally, one critical section was identified for each high fill/swamp crossing area. In all areas where cohesive deposits were encountered, the stability of the proposed new embankment section(s) was analyzed using limit equilibrium methods. The stability analysis assumes that the organic deposits and near surface cohesive soils have been removed and replaced in accordance with OPSD 203.010 (Embankments Over Swamps) prior to construction of the new embankment or OPSD 203.020 (Embankments over Swamp, Existing Slope Excavated to 1H:1V) for embankment widening. The thicknesses of the organic deposits are presented in Table 3.

All limit equilibrium slope stability analyses were carried out using the commercially available program GeoStudio 2007 (Version 7.19), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static total stress (undrained) conditions at the end of construction and for long-term effective stress (drained) conditions for MTO embankments. This FoS is considered adequate for the embankments at these sites considering the design requirements and the field data available and is based on deep-seated, global failure surfaces that would affect the operation of the highway. In order to obtain the critical failure surface, circular or "rotational" and/or block or "wedge" failure surfaces were modeled in the analysis. The stability analyses were carried out to assess the minimum FoS for the various embankment heights and geometries, with the results of the stability analyses compared to the target minimum FoS for each critical section.

#### 6.2.2.2 Parameter Selection

The simplified stratigraphy together with the associated strength(s) and unit weight(s) employed for the different native soil types at the critical sections in each high fill/swamp crossing area are summarized in Table 3. Additional details of foundation engineering parameters employed for the cohesive deposits (i.e., clayey silt / silty

clay / clay) encountered are provided on Figures A21, B25 and C22 in Appendices A, B and C, respectively. The rock fill modelled in the analyses (below and above the water table) is assumed to have a unit weight of  $19 \text{ kN/m}^3$  and an effective friction angle of  $40^\circ$ . The granular fill (assume Granular B Type II) modelled in the analyses is assumed to have a unit weight of  $20 \text{ kN/m}^3$  (uncompacted below the water table) and  $21 \text{ kN/m}^3$  (above the water table) and an effective friction angle of  $35^\circ$ . The stability of the Highway 17 and St. Pothier Road embankments were analyzed for a side slope geometry of 1.25H:1V assuming rock fill construction and backfill. For embankments constructed of granular fill (typically for widened embankments), side slopes of 2H:1V were analyzed. Granular fill is also required to backfill sub-excavated areas to allow for wick drain installation if this is the preferred mitigation option.

The overburden encountered in the various areas is generally composed of organic deposits and/or embankment fill underlain by interlayered deposits of either granular soils (sand, silty sand, sandy silt, silt and sand, silt) or a combination of cohesive deposits (clayey silt, silty clay and/or clay). For granular soils, effective stress parameters were employed in the analyses assuming drained conditions. The effective stress parameters (effective friction angle and effective cohesion) for the organics and granular soils were estimated from empirical correlations using the results of in situ SPTs, in conjunction with engineering judgement based on experience in similar soil conditions.

For cohesive deposits, total stress parameters were employed in the analyses assuming undrained conditions. The total stress parameters (i.e., average mobilized undrained shear strength –  $s_u$ ) for the cohesive soils were assessed based on the results of in situ field vane shear tests, inferred from the laboratory consolidation tests results, and estimated from correlations with the SPT results and other laboratory test data (i.e., natural water content), where appropriate. For the consolidation tests, the following correlation proposed by Mesri (1975) was employed to estimate the undrained shear strength:

$$s_u = 0.22\sigma'_p$$

where:

|             |   |  |
|-------------|---|--|
| $s_u$       | = | average mobilized undrained shear strength (kPa) |
| $\sigma'_p$ | = | preconsolidation pressure (kPa)                  |

Where appropriate, Bjerrum's correction factor for plasticity was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests as follows:

$$s_{u(mob)} = \mu s_{u(FV)} \quad (\text{after Bjerrum, 1973})$$

where:

|              |   |   |
|--------------|---|---|
| $s_{u(mob)}$ | = | average mobilized undrained shear strength (kPa)      |
| $s_{u(FV)}$  | = | undrained shear strength from field vane test (kPa)   |
| $\mu$        | = | Bjerrum's correction factor based on Plasticity Index |

Where varved clay was encountered, an additional reduction factor of 25 per cent was employed to account for the angle of minimum shearing resistance (Milligan and Lo, 1967).

### 6.2.3 Settlement

The following sections outline the methods used to carry out the settlement analyses at the various high fill/swamp crossing areas and also present the parameters used in the analyses for each of the embankment



critical section(s). The results of the analyses are presented in Section 6.5 for each high fill/swamp crossing area where they are discussed together with the results of the stability analyses and recommendations regarding possible design and construction alternatives to mitigate stability issues and/or post-construction settlement.

#### **6.2.3.1 Methodology**

To estimate the magnitude of the expected settlements, analyses were carried out at the critical sections of the proposed fill embankments using the commercially available program Settle<sup>3D</sup> (Version 2.0) produced by Rocscience Inc. and/or hand/spreadsheet calculations. Critical sections correspond to the greatest new embankment height and/or the maximum thickness of soft, compressible cohesive soils and/or the maximum thickness of peat removal. The settlement analysis assumes that the organic deposits and near surface cohesive soils have been removed and replaced in accordance with OPSD 203.010 (Embankments Over Swamps) prior to construction of the new embankment or OPSD 203.020 (Embankments over Swamp, Existing Slope Excavated to 1H:1V) for widened embankments.

The sources of settlement are considered to include:

- immediate settlement of the native granular soils;
- primary time-dependent consolidation of the cohesive deposits (using Terzaghi's one-dimensional consolidation theory);
- secondary time-dependent (creep) consolidation of the cohesive deposits (long-term); and
- self-weight compression of the embankment fill materials (short-term and long-term).

The thickness of the compressible foundation soils and the height of the embankments vary along the proposed highway alignment within each high fill/swamp crossing area, and as such the settlement along the length of a given alignment will similarly vary. Given that the analyses were carried out at the critical sections of each high fill/swamp crossing area, the settlement estimated will generally represent the maximum value along a given section of the alignment.

#### **6.2.3.2 Parameter Selection**

The simplified stratigraphy together with the associated deformation and time-rate consolidation parameters employed for the different native soil types for the critical sections in each high fill/swamp crossing area are given in Table 3. Additional details of foundation engineering parameters employed for the cohesive deposits (i.e., clayey silt/silty clay/clay) encountered in areas H1, H2 and H3 are provided on Figures A21, B25 and C22 in Appendices A, B and C, respectively.

The immediate compression of the cohesionless deposits were modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in Canadian Highway Bridge Design Code and its commentary (CHBDC 2006) and adjusted, if appropriate.

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation tests and, where appropriate, in situ field vane tests to estimate the deformation parameters for the cohesive deposits. In addition, the results of the laboratory index testing were also employed to further assess deformation parameters (i.e., recompression and compression indices) using empirical correlations proposed in literature by Koppula (1986), Terzaghi and Peck (1967), Kulhawy and Mayne (1990) and Azzouz et al. (1976). The correlation by Terzaghi and Peck (1967) and Koppula (1986) relating the natural water content and liquid limit to the compression index was found to be the most consistent with the results of laboratory consolidation tests for the clayey soils at this site.

The following correlation relating in situ undrained shear strength to preconsolidation pressure (Mesri, 1975) was employed:

$$\sigma'_p = \frac{s_{u(mob)}}{0.22}$$

where:

$$\begin{aligned} s_{u(mob)} &= \mu s_{u(FV)} \\ \sigma'_p &= \text{preconsolidation pressure (kPa)} \\ s_{u(mob)} &= \text{average mobilized undrained shear strength (kPa)} \\ s_{u(FV)} &= \text{undrained shear strength from field vane test (kPa)} \\ \mu &= \text{Bjerrum's correction factor based on Plasticity Index} \end{aligned}$$

The coefficient of consolidation,  $c_v$  ( $\text{cm}^2/\text{s}$ ), required in the time-rate settlement analysis, was established for the high fill/swamp crossing areas (H1, H2 and H3) using the combined results of the laboratory consolidation tests and the estimated  $c_v$  values based on the Unified Facilities Criteria (U.S. Navy, NAVFAC 1986) correlation with liquid limit assuming normally consolidated soils.

In addition to primary consolidation within the cohesive deposits (i.e., clayey silt to clay), secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after full dissipation of excess pore pressure under a constant stress. The following relationship has been employed for estimating the magnitude of creep settlement over the life of the embankment following the completion of primary settlement at each location:

$$S_c = HC_{\alpha\epsilon} \log\left(\frac{t}{t_{EOP}}\right)$$

where:

$$\begin{aligned} S_c &= \text{secondary consolidation (creep) settlement (mm)} \\ C_{\alpha\epsilon} &= \text{modified secondary compression index as estimated from laboratory consolidation tests} \\ H &= \text{initial thickness of compressible clay deposit (mm)} \\ t &= \text{post-construction period of interest (20 years)} \\ t_{EOP} &= \text{time to reach end of primary consolidation (years)} \end{aligned}$$

In addition to estimating the modified secondary compression index from consolidation tests, the following empirical correlation by Mesri (1973) was also utilized to estimate  $C_{\alpha\epsilon}$  from water content:

$$C_{\alpha\epsilon} = \frac{w_n}{10,000}$$

where:

$$w_n = \text{natural water content (\%)}$$



### 6.2.3.3 Settlement of Embankment Fill

Where rock fill is used for the construction of the proposed embankments, there will be settlement due to compression of the rock fill itself under self-weight, in addition to the settlement of the underlying foundation soils as described above. The magnitude of settlement of the rock fill depends on the following factors:

- type of rock/strength of particles;
- size and shape of rock particles;
- gradation of rock fill;
- total height/thickness of rock fill (stress level); and
- method of construction and sequence of placement (including lift thickness, compactive effort and state of packing).

The settlement of rock fill occurs as a result of re-arrangement of rock particles under load and wetting and as a result of localized crushing of rock particles at point contacts. The magnitude of both the short-term and long-term post-construction settlement of the rock fill is a function of the height of fill as well as the method of fill placement (i.e., compacted versus dumped rock fill) as outlined in MTO Foundations' Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates (MTO, 2010).

Rock fill should be placed, whenever possible, in a controlled manner (i.e., not end-dumped) in accordance with OPSS.PROV 206 (Grading). Blading, dozing and 'chinking' the rock fill to form a dense, compact mass is required to minimize voids and bridging and to reduce settlements and should be used to construct rock fill embankments above the existing groundwater table. Where rock fill cannot be placed in a controlled manner (i.e., below the groundwater table), the post-construction settlement of the rock fill is expected to be greater.

### Short-Term Rock Fill Settlement

The magnitude of short-term post-construction settlement associated with compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (MTO, 2010), as follows:

| Height of Rock Fill, H | Short-Term Rock Fill Settlement |                  |
|------------------------|---------------------------------|------------------|
|                        | Compacted Rock Fill             | Dumped Rock Fill |
| Up to 5 m              | 0.5% H                          | 1.0% H           |
| >5 m to 10 m           | 0.75% H                         | 1.5% H           |
| >10 m to 15 m          | 1.0% H                          | 2.0% H           |

Approximately 90 per cent of the short-term settlement may be expected to occur within the first six (6) months following construction of the embankment to full height. The short-term settlement is expected to be fully completed within one (1) year following the completion of embankment construction to full height.



### Long-Term Rock Fill Settlement

The magnitude of long-term post-construction settlement for compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (MTO, 2010), as follows:

| Total Height of Rock Fill, H | Long-Term Rock Fill Settlement |                  |
|------------------------------|--------------------------------|------------------|
|                              | Compacted Rock Fill            | Dumped Rock Fill |
| Up to 15 m                   | 0.1% H                         | 0.2% H           |

The long-term rock fill settlement is expected to occur from one (1) year following the completion of construction over the life of the embankment.

### 6.2.4 Wick Drain Parameters

A detailed wick drain analysis has not been completed as part of this report. Where wick drains have been recommended to mitigate post construction settlement, total stress analysis has been used to analyze the stability of the embankments. Detailed wick drain design, which should be carried out if this mitigation option is the preferred option, typically utilizes effective stress analysis.

For purposes of the preliminary wick drain settlement analysis in this report, we have assumed that the smear ratio,  $(k_h/k_s)$ , (i.e. the ratio of horizontal permeability of the undisturbed soil ( $k_h$ ) to the permeability of the soil in the smear zone ( $k_s$ )), is 5. We have also assumed the ratio of the coefficient of consolidation in the horizontal direction to the coefficient of consolidation in the vertical direction,  $(c_h/c_v)$ , is 1.2.

## 6.3 Settlement Performance Requirements

The settlement performance criteria for the design of high fill embankments and embankments over swamp crossings are in accordance with MTO Foundations' Guideline for Embankment Settlement Criteria for Design (MTO, 2010). In general, new embankments not approaching a structural element are to be designed as follows:

| Type                                  | Maximum Limits During Pavement Design Life |                              |
|---------------------------------------|--|------------------------------|
|                                       | Total Settlement                           | Differential Settlement Rate |
| Freeways (i.e., Highway 17)           | 100 mm                                     | 200:1                        |
| Non-Freeways (i.e., St. Pothier Road) | 200 mm                                     | 100:1                        |

Widened embankments are to be designed as follows, such that drainage is not impeded:

| Type                                  | Maximum Limits During Pavement Design Life |                              |
|---------------------------------------|--|------------------------------|
|                                       | Total Settlement                           | Differential Settlement Rate |
| Freeways (i.e., Highway 17)           | 50 mm                                      | 200:1                        |
| Non-Freeways (i.e., St. Pothier Road) | 75 mm                                      | 100:1                        |



These total and differential settlement rates are considered to be applicable over a 20-year period following completion of construction (i.e., final paving). These performance criteria form part of the overall design performance for each high fill/swamp crossing area.

Where new embankments approach structural elements, more stringent settlement criteria will apply, in accordance with the MTO Foundations' Guideline (MTO, 2010).

## **6.4 Stability and Settlement Mitigation Options**

At each high fill embankment and embankment over swamp crossing location, stability and settlement have been assessed based on existing subsurface conditions and proposed embankment fill heights. The presence of weak/soft, compressible soils underlying a proposed embankment can lead to the potential for instability or unacceptably large settlements with the placement of fills. There are a number of options for mitigating the potential for instability and/or settlements. A brief discussion on these alternatives is given below.

Details of the mitigation options for the high fill/swamp crossing areas requiring measures to mitigate stability/settlement issues of the foundation soils are provided in Section 6.5. These measures include: preloading of the subsoil with or without surcharging the embankments; installation of wick drains into the cohesive stratum; incorporation of lightweight fill (expanded polystyrene (EPS) or cellular concrete) into the embankment; full sub-excavation of unsuitable (organic/soft/compressible) soils; and potential combinations of these measures. Other ground improvement measures such as rammed aggregate piers and deep soil mixing are also considered as discussed in Section 6.4.6. Associated monitoring programs are discussed in Section 6.4.7. The advantages, disadvantages, relative costs and risks/consequences of mitigation alternatives for the high fill/swamp crossing areas, where required, are summarized in the Evaluation of Stability/Settlement Mitigation Options Tables A1, A2, B1, B2 and C1 provided in the respective appendices. In addition, a comparison of the estimated post-construction settlement over a 20-year period between the base case (i.e., no settlement mitigation carried out) and the various mitigation alternatives considered was carried out for each of the high fill/swamp crossing areas. The results of the settlement analyses are summarized in Table 4.

Depending on the area, one alternative or a combination of alternatives to mitigate stability and/or settlement issues may be more advantageous than others. A summary of the preferred foundation mitigation option for each high fill/swamp crossing area, including the recommended embankment fill type and embankment side slope, maximum depth of organics encountered, stability/settlement mitigation, estimated settlement (during construction and post-construction), recommended width of platform widening and recommended excavation guideline is provided in Table 5.

In areas where the embankments are being widened and the overall grade raise is limited (i.e. less than about 300 mm), it is anticipated that there will not be any significant risk of instability of the embankments. In these areas there is typically no need to implement any special construction procedures or schedule to maintain stability or to mitigate differential settlement of the foundation soils.

### **6.4.1 Preloading (with Stability Berms and/or Staged Construction)**

Preloading of the foundation soils may be considered for improving the stability and reducing post-construction settlements of the proposed embankments. Preloading refers to the placement of fill either up to the proposed profile grade of the highway/roadway or a portion thereof (i.e., partial preload), in one or more stages, in

advance of embankment completion and final pavement construction, in order to preconsolidate the underlying compressible soils. Preloading reduces the magnitude of long-term, post-construction settlements by promoting such settlements to occur under fill loads in advance of final grading of the embankment. It also may increase the strength of cohesive soils underlying the embankment footprint, thereby improving stability.

In general, preloading requires placement of embankment fill (either in whole or part), and in some cases monitoring of settlement, and possibly pore pressures, for a period of time corresponding to approximately the 'End of Primary' (EoP) consolidation of cohesive soils. EoP consolidation times will vary depending on the properties and thicknesses of the cohesive deposits, and the height of the fill. Where secondary consolidation (creep) settlements are expected to be small over the design life of the embankment, final grading for construction can proceed once the estimated EoP consolidation has occurred. Where creep settlements are considered to be large enough to affect the long-term performance of the highway, these settlements can be reduced by constructing a portion of the final embankment with lightweight fill upon the completion of the design preload period or by surcharging.

In areas where cohesive deposits are thick and/or weak/soft, and where such conditions coincide with proposed high embankment fills, it may be necessary to construct stability berms along the embankment toes and/or place the embankment fill in stages in layers of limited thickness to ensure that the stability of the embankment is maintained. Stability (toe) berms consist of rock fill buttresses placed against the toe of the proposed embankment fill, producing a stepped embankment cross-section geometry. This stepped configuration produces a similar effect (i.e., increased stability) as using flatter embankment slopes, but often requires less fill material. Depending on the subsurface conditions and the proposed embankment height, toe berms will typically be on the order of about one third to one half of the height of the final embankment. The lateral extent (width) of toe berms will vary depending on the results of the stability analyses, but could range from half to one times the highway embankment height, or greater. Where staged construction is required, the individual layers of fill would have limited thickness and each construction stage would be separated by a suitable time interval to allow pore pressures to dissipate and strength gain to occur in the underlying cohesive soils while limiting the potential for instability of the embankment.

It should also be noted that with preloading, it is still required that all existing organic deposits and near surface cohesive soils be sub-excavated prior to placement of any fill, because these soils are highly compressible and experience significant secondary consolidation (creep) settlement. The organic deposits should not be used as toe berm material.

This option is most suited for areas where removal of cohesive soils and their replacement with rock fill is not considered practical (i.e., the depth to the bottom of cohesive deposits is greater than 12 m) and where a delay in the construction schedule is acceptable or can be accommodated.

The advantages of this option are:

- reduced generation of excess excavation spoil compared with full sub-excavation;
- will not require a larger right-of-way corridor, unless large toe berms are required; and
- the quantity of rock fill is limited to that required for sub-excavation and replacement of the organic deposits and near surface cohesive soils (if toe berms are not required), and to compensate for consolidation and settlement of the foundation soil.

The disadvantages of this option are:

- construction is delayed to allow for all or a portion of primary consolidation to be completed, and possibly for staged construction (if required);
- increased quantity of rock fill if toe berms are required for stability;
- may require lightweight fill for a portion of the construction of the final embankment to reduce long-term post-construction settlements if creep settlements are expected to be large;
- requires an instrumentation and monitoring program to assess when the settlement performance criterion has been achieved; and
- requires re-grading to account for settlement prior to construction of the final pavement structure.

An operational constraint governing the embankment construction will be required for preloading. In addition, a monitoring program will be required (see Section 6.4.7).

#### **6.4.2 Surcharging (and Stability Berms and/or Staged Construction)**

Similar to preloading, surcharging refers to the placement of embankment fill in advance of final pavement construction to reduce long-term, post-construction settlements (including creep settlement). The difference between preloading and surcharging is the amount of fill placed and the time required for consolidation to be achieved. With surcharging, the fill is placed to the full embankment height, followed by an additional lift of fill (the surcharge) above that required to construct the final embankment geometry. This additional lift of fill applies greater stress to the underlying cohesive deposits and reduces the time to achieve primary consolidation over that achieved by preloading only, resulting in over consolidation of the underlying compressible foundations soils. At the EoP consolidation, the portion of the surcharge fill remaining above the required embankment height (sub-base level) is removed. The surcharge fill can also be left in place for a longer duration to reduce the magnitude of long-term, creep settlement.

As with preloading, it may be necessary to construct toe berms or stage the placement of embankment fill and surcharge to limit the potential for instability.

Surcharging is most suited to those areas considered appropriate for preloading, where the stability of the higher surcharged embankment can be practically maintained by reasonably sized excavations, toe berms or staged construction, but where sufficient time for primary consolidation settlements to occur under preload fill loads alone is not available. Surcharging can also be considered for areas where large creep settlement is expected.

The advantages of this option are:

- reduced generation of excess excavation spoil over full sub-excavation;
- reduced secondary consolidation (creep) settlement;
- will not require a larger right-of-way corridor, unless large toe berms are required;
- the quantity of rock fill is limited to that required for sub-excavation and replacement of organic deposits and near surface cohesive soils, and to compensate for consolidation and foundation soil settlement (if toe berms are not required); and



- decreased delay time for construction over preloading alone.

The disadvantages of this option are:

- construction is delayed, albeit less than for preloading, to allow for primary consolidation to occur;
- longer construction time if staged construction is required or to reduce secondary consolidation (creep) settlement;
- larger quantity of rock fill if toe berms are required for stability as compared to preloading alone;
- requires an instrumentation and monitoring program to assess when EoP consolidation is reached; and
- increased handling of the surcharge fill.

An operational constraint governing the embankment construction will be required for surcharging. In addition, a monitoring program will be required (see Section 6.4.7).

### **6.4.3 Wick Drains**

Where the time required to reach the settlement performance criterion is considered too long (i.e., unacceptable to meet a specific construction schedule), even with surcharging, consideration may be given to installing wick drains in conjunction with surcharging to accelerate the rate of primary consolidation. Wick drains are prefabricated geotextile drains installed vertically from ground surface into or through the soft, compressible foundation soils in order to increase the rate of excess pore pressure dissipation. Typically, wick drains are installed on a 1 m to 3 m triangular grid spacing over the embankment footprint.

The use of wick drains is most suited to areas with thick (i.e., greater than about 5 m) deposits of soft, compressible foundation soils and proposed high embankment fills where primary consolidation times are large even under surcharge conditions.

It would still be necessary to sub-excavate and remove organic deposits and near surface cohesive soils and place a granular drainage blanket at ground surface prior to the installation of the wick drains.

If the thickness of sub-excavation of the organic deposits and near surface cohesive soils is greater than about 4 m, then pre-drilling to install wick drains may be required to advance the drains through the backfill material. In any case, the backfill material must consist of granular material rather than rock fill to facilitate installation of the drains with or without pre-drilling.

The advantages of this option are:

- decreased consolidation time under surcharging; and
- decreased rate of staged construction, if required to maintain stability during construction.

The disadvantages of this option are:

- additional time and expense to install wick drains prior to embankment construction;
- may require pre-drilling at wick drain locations if a substantial thickness of backfill is present above the clay, incurring additional time and expense;

- additional long-term settlements due to creep settlement of the cohesive deposit (if not compensated for by surcharging or lightweight fill);
- requires an instrumentation and monitoring program to assess when the settlement performance criterion has been achieved; and
- requires re-grading to account for settlement prior to construction of the final pavement structure.

If wick drains are utilized, an NSSP for the supply and installation of the wick drains and the drainage blanket will be required. An operational constraint governing the embankment construction will also be required. In addition, a monitoring program will be required (see Section 6.4.7).

#### **6.4.4 Lightweight Fill**

An alternative for reducing the magnitude of long-term settlement and improving stability in areas of weak/soft, compressible foundation soils is to use lightweight fill, such as EPS or cellular concrete, for embankment construction. The use of lightweight fill reduces the load applied to the foundation soils due to the low density of the fill materials. This in turn reduces the magnitude of post-construction settlement and reduces the potential for instability.

Lightweight fill is not considered a practical option for general use due to the expense and/or shipping costs for the supply of these types of fills. Rather, lightweight fill is most suited for areas underlain by deep compressible subsurface deposits, where sub-excavation is not practical or feasible, where long-term post-construction creep settlements affect the performance of the highway and where there is no available time in the construction schedule for a sufficient preload or surcharge period. In addition, lightweight fill can be used in conjunction with preloading, surcharging and wick drain designs in order to optimize the design.

Where a stability and/or settlement mitigation option requires the use of lightweight fill as part of the construction of the embankments, rock fill cannot be used as the levelling pad or protective cover for the lightweight fill due to the size of rock fill particles. As such, for these embankments, granular fill is to be used for levelling pad construction and for the side slope protective cover.

The advantages of this option are:

- improved stability;
- reduced long-term post-construction settlements; and
- shortened construction schedule.

The disadvantages of this option are:

- requires embankments to be constructed with 2H:1V side slopes given the need for granular fill for levelling pad and conventional soil cover on side slopes (i.e., cannot use rock fill);
- significant additional expense of lightweight fill (depending on the volume required); and
- not feasible to install below the groundwater table (due to buoyancy forces) and in low height embankments (due to minimum conventional soil cover requirements on top of the EPS).



It should be noted that slag fill may also be considered for use as lightweight fill for embankment construction. However, the specific environmental restrictions for this type of material should be considered by the designer prior to recommending its use.

#### 6.4.5 Full Sub-Excavation

Sub-excavation of the weak/soft and compressible (i.e., clayey) deposits underlying the footprint of a proposed embankment in advance of the placement of rock fill is a viable option for improving the stability and controlling long-term settlement of the proposed embankments in some areas of this site. The removal of the soft, compressible cohesive soils would result in improved stability and significantly reduce settlement within the areas underlain by relatively thinner cohesive deposits and/or where high embankment fills are proposed. It should be noted that despite the reduction in settlement, the post-construction settlement of rock fill may still exceed the settlement performance criterion. As such, the embankments may need to be preloaded for a period of time to be able to attain adequate construction settlement and subsequently meet the post-construction settlement criterion associated with long-term performance of the roadway. The additional below grade rock fill embankment should be constructed with the same side slope profile as that of the above grade embankment (i.e., 1.25H:1V for rock fill) since the natural slope of the rock fill should not be affected by placement under water. This option has the advantage that construction of the above grade embankment could proceed upon completion of sub-excavation and replacement without concerns of instability. However, full sub-excavation may produce a large volume of spoil material for disposal and may require a large volume of rock fill replacement. The necessity to develop stable side slopes or back slopes within the excavation may result in cut slope geometries ranging from 1H:1V to as flat as 3H:1V. Flatter slopes would increase the lateral extent of the excavation and may require a wider right-of-way.

Based on the results of the subsurface investigation in the high fill/swamp crossing areas, there is typically a near-surface compressible organic deposit, which must be removed, and a lower compressible cohesive deposit of which the depth to the bottom of the deposit within the high fill/swamp crossing area varies across the project site. The depth to the bottom of the near-surface compressible organic deposit is up to about 4 m below existing ground surface and the depth to the bottom of the lower compressible deposit is up to about 20 m below existing ground surface. In general, groundwater was encountered at/near the existing ground surface. We understand that based on MTO field experience on similar embankment construction projects, the practical maximum depths that can be reached with conventional and long stick excavator equipment is about 6 m and 12 m, respectively. Below a depth of 12 m, specialized drag-line equipment would be required. As such, in the absence of unforeseen conditions which would prohibit its application, sub-excavation of organic and soft compressible soils up to a practical depth of about 12 m and replacement with rock fill is considered a generally feasible option for construction of the highway embankments and would result in enhanced stability and reduced settlement of the embankments.

This option is most suited for areas where there is a limited thickness of organic deposits and near surface weak/soft compressible soils underlying the proposed embankment (i.e., less than 12 m), making their removal practical where there are no requirements for setbacks, where adequate right-of-way is available, and where there are no conflicts with encroachment on existing adjacent features.

The advantages of this option are:

- improved stability;



- reduced post-construction settlements of the foundation;
- potential reduced delay in construction; and
- no requirement for stabilizing toe berms.

The disadvantages of this option are:

- generation of large volume of excavation spoil requiring disposal/management;
- increased post-construction settlement of rock fill, typically requiring a preload period, in addition to sub-excavation, to satisfy the settlement performance criterion;
- potential for requiring a larger right-of-way corridor due to backslope requirements;
- potential need for temporary excavation support where an embankment widening is constructed adjacent to existing highway embankments, and
- requires greater quantities of rock fill.

#### **6.4.6 Ground Improvement**

Ground improvement techniques (such as rammed aggregate piers or deep soil mixing) could also be considered to improve the performance of the subsoils. Rammed aggregate piers are not considered practical at this site due to the relatively great depth required to extend the columns to an appropriate bearing layer and due to the lack of lateral confinement that can be provided by the soft consistency of the cohesive deposits. Deep soil mixing, however, may be technically feasible but would require an in situ trial section to be constructed followed by completion of the soil mix design for the entire embankment.

Deep soil mixing involves the in-situ mechanical blending of soil columns with a binder material in a grid pattern under the full width and length of the affected embankment to improve the engineering properties (compressive strength, shear strength and/or permeability) of the soil layer(s). The binder material typically consists of a cement-based grout slurry (wet soil mixing) or a cement-slag powder (dry soil mixing). For wet soils (i.e., moisture contents greater than about 60 per cent), dry soil mixing is generally more cost effective than wet soil mixing due to the costs associated with supplying of grout and mixing it adequately into the soil.

The strength of the improved soil ("soilcrete") develops differently with time depending on the type of soil and the amount and type of binder material, but generally increases with increasing binder dosage. The strength improvements are generally highest for inorganic soils with moderate water contents. Organic soils and peat soil mixtures, as encountered in the near surface cohesive deposits of the high fill/swamp areas, can be stabilized with a binder material but information on the adequacy or practicality of increasing the strength and stiffness of the soils is not readily available.

The "soilcrete" column type, size and spacing would have to be designed by a specialist ground improvement contractor. The amount of binder material required in each column to provide adequate increase in the stiffness of the soil mass is not possible to estimate without bench scale testing and may possibly require a field test program. However, in general, the higher the water content and lower the strength of the existing soils, the larger the amount of binder material that is required and the closer the required spacing of columns will result in a sufficient level of improvement.



The high cost of mobilization of equipment and materials, the use of specialized equipment and potential large volume of mixing additive required for the extent of the soil improvement area may offset the cost savings associated with reduced sub-excavation and soil disposal.

The advantages of the deep soil mixing option are:

- improved stability;
- reduced post-construction settlements of the foundation; and
- reduced sub-excavation and soil disposal quantity.

The disadvantages of the deep soil mixing option are:

- specialized testing and design required;
- specialized construction equipment and supplies required;
- additional time for soil mixing required prior to embankment construction; and
- risk that strength gains may not be cost effective and/or more conventional mitigation measures may still be required.

#### 6.4.7 Instrumentation and Monitoring

For some areas where the preloading and surcharging options are adopted and in all areas where staged construction and/or wick drains foundation options are adopted, the magnitude and time-rate of settlement as well as dissipation of pore pressures during and after construction of the embankments should be assessed with monitoring instrumentation. Such monitoring would consist of installing settlement pins/stakes (Ss), settlement plates (SPs) and vibrating wire piezometers (VWPs) below the embankments and taking regular measurements/readings at given intervals of time during and after construction of the embankments for the duration of the preloading/surcharging period. In addition, standpipe piezometers (SPPs) may be required and are usually installed to provide background pore pressure readings for the vibrating wire piezometers. This monitoring instrumentation is particularly important where it is considered necessary to carefully monitor the stability of the foundation soils during staged placement of embankment fill.

The extent of instrumentation and the frequency of monitoring required will depend on the foundation treatment alternative chosen for a given site and the height of the proposed embankment fill. Specifications for the type, number and layout of the instrumentation, together with the supply, installation, protection and monitoring frequency and action levels should be included as a NSSP in the Contract Documents.

### 6.5 Results of Analysis

The results of the stability and settlement analyses for each high fill/swamp crossing area are provided in the following sections. In addition, the options and recommendations for achieving the target FoS for the required embankment geometry and for minimizing the time-dependent, post-construction settlements are also discussed. For high fill/swamp crossing areas H1, H2 and H3 that require stability and/or settlement mitigation, the advantages, disadvantages, relative costs, and risks/consequences of various alternatives for these areas

are summarized and ranked in the Evaluation of Stability/Settlement Mitigation Options Tables A1, B1, B2 and C1 in the respective Appendices.

In areas where the embankments are being widened and the overall grade raise is limited, it is anticipated that there will not be any significant risk of instability of the embankments. In these areas there is typically no need to implement any special construction procedures or schedule to maintain stability or to mitigate differential settlement of the foundation soils.

In areas where the foundation soils are comprised of cohesive deposits and the presence of weak/soft cohesive deposits constitutes zones of potential instability for the proposed embankments, time-dependent settlements of the new embankments are expected. In these areas, consideration must be given to an enhanced design and/or to follow a construction sequence that will achieve the minimum target FoS of 1.3 for the proposed new embankment height and geometry and limit the post-construction settlements and subsequent maintenance of the new highway pavement structure.

For new embankments constructed with rock fill or where sub-excavation and backfilling with rock fill is carried out, settlement of the embankment rock fill is also expected due to compression of the rock fill itself (see Section 6.2.3.3). In these areas, the post-construction settlement of rock fill may exceed the settlement performance criterion. As such, the embankment would need to be preloaded to obtain acceptable post-construction settlements associated with long-term performance.

#### **6.5.1 Highway 17 – STA 12+220 to 12+570 High Fill H1**

Along this section of the alignment, the proposed WBL will be a widening of the existing Highway 17 embankment while the proposed EBL will be constructed over the native low-lying swampy ground to the south of the existing highway and proposed widening. Therefore, the EBL and the WBL embankments in this high fill/swamp area will be considered separately in Sections 6.5.1.1 and 6.5.1.2, respectively.

##### **6.5.1.1 Highway 17 WBL – STA 12+220 to 12+570 High Fill H1**

Generally, the proposed Highway 17 WBL embankment will be constructed essentially at the same location as the existing Highway 17 embankment between STA 12+220 and 12+570. The proposed WBL centreline will be slightly shifted to the south from the existing Highway 17 centreline resulting in embankment widening up to about 9 m and a grade raise up to 0.8 m at the south edge of pavement, particularly towards the eastern portion of this section. The existing embankment is up to about 5 m high above the surrounding swamp area although greater than 5 m of fill was encountered in some boreholes, suggesting that some peat was removed and/or settlement has occurred over time since embankment construction.

The subsurface soils along the proposed WBL alignment in High Fill Area H1 consists of embankment fill or a peat/topsoil deposit underlain by a sand and silt deposit and a cohesive deposit of varved, silty clay to clay transitioning into clayey silt, which in turn is underlain by deposits of silt, sand and silt to sand and sand and gravel to gravelly sand. The cohesive deposit is up to about 8 m thick in some locations and extends to depths up to 13.4 m below ground surface. Resistance to dynamic cone penetration and borehole advancement was



encountered at depths of up to 19.4 m below ground surface. Details of the subsurface conditions for this swamp crossing area are presented in Section 4.3 and shown on Drawings A1 to A3 in Appendix A.

The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 3. Additional details of foundation engineering parameters employed for the cohesive deposits (i.e., clayey silt/silty clay/clay) encountered in H1 are provided on Figure A21 in Appendix A. The piezometric condition used in the analyses assumes the water table is at the interface between the native and fill materials (at about Elevation 241.2 m).

The critical section for this area is located at about STA 12+400 along the proposed Hwy 17 WBL alignment and corresponds to the thickest cohesive deposit up to 7.9 m. At the critical section, the embankment widening is approximately 8 m and the grade raise at the south edge of pavement is approximately 0.5 m. The widening at the critical section creates a loading equivalent to about a 4.6 m above the existing ground at the toe of slope. Sub-excavation of peat at the toe of slope of up to 1.1 m is required, resulting in a total fill thickness of 5.7 m in the widened area.

#### **6.5.1.1.1 Stability**

The stability analyses carried out at the critical section indicates that the south side of the existing embankment has a Factor of Safety (FoS) less than 1.3 for a deep-seated, global failure surface that would impact the operation of the highway. The stability analyses carried out at the critical section after completion of construction (which includes the removal and replacement of the organic deposit and the construction of the WBL widening and Hwy 17 EBL embankment), indicates that the embankment will have a Factor of Safety (FoS) greater than 1.3 for a deep-seated, global failure surface that would impact the operation of the highway. Therefore, stability mitigation is not required for the WBL embankment in the High Fill Area H1, as shown on Figure A22.

#### **6.5.1.1.2 Settlement**

To estimate the magnitude of the expected settlements due to new construction, analysis was carried out at the critical section representative of the subsurface conditions within the high fill area, at approximately STA 12+400. Based on the results of the settlement analysis for the embankment widening, the short-term settlement of the foundation soils is estimated to be about 80 mm at the south edge of pavement. This estimated settlement in the Highway 17 WBL is comprised of about 10 mm of immediate settlement due to compression of the cohesionless deposits and about 70 mm of primary consolidation of the 7.9 m thick cohesive deposit.

Based on an average coefficient of consolidation ( $c_v$ ) of about  $2.0 \times 10^{-3} \text{ cm}^2/\text{s}$  estimated for the cohesive deposit in the normally consolidated range, the imposed loading conditions for the approximately 5.7 m of rock fill for the widening at the toe of slope and assuming two-way drainage of the 7.9 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 3 years.

The magnitude of secondary consolidation (creep) settlement for the normally consolidated portion of the cohesive deposit is expected to be about 55 mm per log-cycle of time for this area corresponding to about 50 mm over a 20-year period following completion of construction.

Although it has been assumed that rock fill would be utilized for the embankment widening, the raise across the existing embankment would be up to about 0.5 m and would be constructed of granular fill, which properly placed and compacted would have nominal settlement.

Since the total post-construction settlement, estimated to be about 120 mm at the south edge of pavement (comprised of 70 mm primary consolidation and 50 mm of creep) exceeds the settlement criterion of 50 mm (for an embankment a widening), settlement mitigation measures are required for the Highway 17 WBL embankment in the High Fill Area H1. Further, differential settlement of 60 mm between the new centreline and new south edge of pavement (assumed to be the edge of the travelled lane for this analysis) also exceeds the criteria of 200:1. It should be noted that higher magnitudes of total and differential settlement will occur beyond the south edge of pavement along the median slope.

#### **6.5.1.1.3 Mitigation of Time Dependent Settlements**

The presence of the up to 7.9 m thick silty clay to clay deposit influences both the stability and the settlement of the widened embankment for the proposed WBL. In order to minimize post-construction settlements, the alternatives presented below can be considered. The alternatives described have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table A1 in Appendix A. A summary of the settlement results for each alternative is provided in Table 4.

Given the presence of the 7.9 m thick cohesive deposit and the associated magnitude of primary and secondary consolidation settlement (about 120 mm) of the foundation soils under the south edge of pavement (i.e., approximately 0.5 m additional fill above the existing ground surface) and 60 mm of differential settlement between the new centreline and new south edge of pavement, the most practical method of construction is to preload the embankment allowing settlement to occur while the traffic is using the highway, followed by maintenance of the roadway in the future. This method does not meet the MTO settlement criteria for post-construction settlement but is more practical from a cost and a construction standpoint compared to other technically feasible options.

##### **6.5.1.1.3.1 Consolidation and Maintenance**

We recommend a construction approach that involves constructing the embankment to its final height, utilizing the embankment as a travelled highway and then conducting ongoing maintenance, as may be required, as approximately 90 per cent of the primary consolidation settlement is expected to be complete in about 3 years. This construction option does not meet the MTO settlement criteria; however, this alternative is the most practical option given the magnitude of total settlement at the new outside edge of the WBL north travelled lane which is about 120 mm (70 mm primary and 50 mm creep). Provided the peat is sub-excavated under the widened portion of the embankment in accordance with OPSD 203.020 (Embankments over Swamp, Existing Slope Excavated to 1H:1V) and the EBL embankment is constructed at the same time, the widened WBL embankment will remain stable while in use, however, the expected post constructed settlements may require maintenance. This alternative relies on the fact that while the expected settlements exceed the MTO settlement criteria for a widened embankment (Figure 3, MTO 2010), the embankment can still function as a travelled lane while consolidation takes place.

This construction approach may require the installation of instrumentation to monitor for settlement, however it is not required to monitor pore pressures for embankment stability. The embankment monitoring can consist of a



series of settlement plates installed in the embankment at the crest of the widened portion of the embankment provided guide rail installation is not impacted and the monitoring points remain accessible. Settlement plates in the shoulder or on the slope (or nail pins in pavement) will be required to be installed along the full length of the high fill section on the south side of the widened embankment at offsets to be determined once the final cross sections are known. For a 350 m long section, settlement plates at approximately 50 m spacing would be appropriate. At two stations, an array of settlement plates should be installed to monitor the cross-sectional settlement across the embankment widening. Monthly readings of settlement plates are recommended for the first year of monitoring and bi-monthly readings afterwards until the decrease in the rate of settlement indicates that the remaining settlement is within the MTO criteria, likely up to 3 years.

The total post-construction settlement (i.e., after construction of the widened embankment is complete and traffic is using the lanes) for the Highway 17 WBL embankment expected during the consolidation period is estimated to be 120 mm which exceeds the settlement criteria. The differential settlement of 60 mm between the new centreline and new south edge of pavement also exceeds the settlement criterion. During an approximately two year consolidation period, approximately 65 mm of settlement will have occurred at the new south edge of pavement and approximately 10 mm at the north edge of pavement. The estimated total and differential settlement after approximately 3 years of consolidation is about 55 mm and 50 mm at the new south and north edge of pavement, respectively, and a differential settlement of 5 mm, which meet the MTO criteria. The results of this analysis are shown on Figure A23.

The main advantages of this option are that there is neither a delay in the construction schedule nor any need to divert traffic for multiple years during staged construction. The disadvantages of this option are that it does not meet the MTO settlement criteria and future highway maintenance is likely to be required.

#### **6.5.1.1.3.2      Lightweight Fill**

In order to meet the settlement criteria without preloading, lightweight (EPS) fill could be incorporated into the widened embankment mass. The Highway 17 WBL embankment can be widened to the south using a 1 m thick layer of EPS under the pavement structure of the new embankment (full width, not to interfere with guide rail installation) and an additional 1.5 m of EPS (for a total of 2.5 m) would be required within the remainder of the south half of the new embankment. This will required sub-excavation of the existing fill to accommodate the EPS. The EPS should be stepped in 0.3 m to 0.5 m increments across the embankment and in the taper zones longitudinally along the highway. Essentially, with this quantity of EPS, the induced settlement will be minimal and will not result in the induction of creep settlement. Total and differential post-construction settlement is estimated to be less than 10 mm.

The main advantages of this option are that it meets the post-construction settlement criteria as well there is a no delay in the construction schedule. The disadvantage of this option is the substantial cost of EPS fill, which is typically an order of magnitude higher than other fill materials as well as having to excavate a portion of the existing embankments in order to install the EPS.

#### **6.5.1.1.3.3      Full Sub-Excavation**

The option of full sub-excavation of the cohesive deposit up to 8 m thick and up to 13.4 m below the road surface or up to 10.4 m below the ground surface at the toe of slope is not considered practical. To fully remove this deposit below the widened portion of the embankment, extensive shoring would be required along the full



350 m length of this high fill section. Further, preloading of the widened embankment would still be required due to the compression of rock fill, otherwise maintenance would need to be carried out at a later date based on the results of a monitoring program as described in Section 6.5.1.1.3.1.

#### **6.5.1.2 Highway 17 EBL – STA 12+220 to 12+570 High Fill H1**

Between STA 12+220 and 12+570, the Highway 17 EBL embankment will require a fill embankment up to 5.7 m high above the existing ground surface to achieve the vertical highway profile. The alignment extends across a low-lying swampy area. The subsurface soils along the EBL alignment consist of a surficial layer of peat/topsoil underlain by a sandy silt deposit and a cohesive deposit of clayey silt to silt transitioning into varved, silty clay to clay transitioning back into clayey silt, which in turn is underlain by deposits of silt to sandy silt, sand and silt to sand and gravelly sand. The cohesive deposit is up to 9.4 m thick in some locations and extends to depths up to 10.4 m below ground surface. Resistance to dynamic cone penetration and borehole advancement was encountered at depths of up to about 20.7 m below ground surface. Details of the subsurface conditions at this embankment are presented in Section 4.4, and are shown on Drawings A1 to A4 in Appendix A.

The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 3. Additional details of foundation engineering parameters employed for the cohesive deposits (i.e., clayey silt/silty clay/clay) encountered in H1 are provided on Figure A21 in Appendix A. The piezometric condition used in the analyses is the water table at the ground surface (about Elevation 241.2 m).

Although the existing highway embankment was not constructed with the incorporation of toe berms, it is unknown what other construction methods were used (i.e., staged construction, surcharging, etc.) to mitigate both settlement and stability. Further, the analysis of the existing embankment configuration (at the critical section at STA 12+320) indicates that for the shear strength design line considered applicable as shown on Figure A21, a Factor of Safety of about 1.1 is obtained. This implies that the existing embankment is stable, albeit with a Factor of Safety less than the current target of 1.3. It is also not known if there has been any long-term settlement that has occurred over the years and what maintenance has been carried out. It should be noted that the silty clay to clay deposit contains an approximately 1.5 m thick crust of higher shear strength, however, the design line interpreted from the undrained shear strength data does not reflect the crust due to the potential for disturbance, fissures, etc. that may be present within the crust.

The critical section (i.e., the greatest embankment height and maximum thickness of soft, compressible foundation soils) for this area is located at about STA 12+320 along the proposed Hwy 17 alignment. The proposed embankment is about 5.7 m high and the cohesive deposit is up to about 9.4 m thick.

##### **6.5.1.2.1 Stability**

The stability analyses carried out at the critical section indicates that after completion of construction (including removal and replacement of the organic deposit), the embankment will have a Factor of Safety (FoS) of less than 1.3 for a deep-seated, global failure surface that would impact the operation of the highway. Therefore, stability mitigation is required in the EBL for this High Fill Area H1.

#### **6.5.1.2.2 Settlement**

Based on the results of the settlement analysis at the critical section, for the condition where the organic deposits are removed and replaced with rock fill, the short-term settlement of the foundation soils is estimated to be about 770 mm. This estimated settlement in the Highway 17 EBL is comprised of about 110 mm of immediate settlement due to compression of the cohesionless deposits and about 585 mm of primary consolidation of the 9.4 m thick cohesive deposit.

Based on an average coefficient of consolidation ( $c_v$ ) of about  $2.0 \times 10^{-3} \text{ cm}^2/\text{s}$  estimated for the cohesive deposit, the imposed loading conditions for the approximately 5.7 m high rock fill embankment and assuming two-way drainage of the 9.4 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 3 years.

The magnitude of secondary consolidation (creep) settlement for the cohesive deposit is expected to be about 85 mm per log-cycle of time for this area corresponding to about 75 mm over a 20-year period following completion of construction.

In addition, the total settlement of the rock fill embankment itself (based on a 5.7 m high embankment at the critical section plus 0.8 m of removal of peat and replacement with rock fill) is estimated to be about 60 mm, with about 50 mm expected to occur within six months of construction of the embankment, 5 mm occurring during the next six months and about 5 mm expected to occur over the remaining design life of the embankment.

Since the total post-construction settlement (primary, creep and rock fill) exceeds the settlement criterion of 100 mm, settlement mitigation measures are required for the EBL embankment in High Fill Area H1.

#### **6.5.1.2.3 Mitigation of Stability Issues and/or Time Dependent Settlement**

The presence of the up to 9.4 m thick cohesive deposit influences both the stability and the settlement of the up to 5.7 m high embankment. In order to construct the embankments to achieve a FoS greater than or equal to 1.3, and to minimize post-construction settlements, the alternatives presented below can be considered. The alternatives described have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table A2 in Appendix A. A summary of the settlement results for each alternative is provided in Table 4.

Given the thick cohesive deposit (the bottom of which is up to about 10.4 m below ground surface) and the associated magnitude of primary and secondary consolidation settlement (about 660 mm) of the foundation soils under an up to 5.7 m high embankment, surcharging with wick drains and incorporating toe berms along the embankments into a staged construction schedule is considered the preferred mitigation alternative.

##### **6.5.1.2.3.1 Surcharging, Wick Drains and Staged Construction with Toe Berms**

We recommend a staged construction approach that incorporates wick drains into the cohesive stratum and a toe berm onto the south side of the embankment, as the preferred, most technically feasible mitigation measure to achieve a stable embankment and to minimize the post-construction settlements and subsequent maintenance of the new roadway surface. The staged construction approach relies on strength gain in the underlying cohesive deposit to enhance stability of the subsequent stages. The first stage could be constructed up to 5.7 m high with a 1.5 m high by 14 m wide toe berm along the south side of the Highway 17 EBL.

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embankment. The first stage of construction includes the installation of instrumentation to monitor for settlement and pore pressure dissipation and requires a wait period to allow for dissipation of the pore pressures and achieve a strength increase of the cohesive deposit (estimated to be about 14 kPa under the EBL embankment). The actual wait period will be dependent on the results of the monitoring.

After the first wait period is complete, the second stage of construction consisting of a 1.3 m surcharge can be completed and a second wait period will be required to allow for dissipation of the pore pressures of the cohesive deposit to satisfy the settlement criterion. The analysis assumes that the embankment will be "topped up" in the second stage to the final grade prior to placing the surcharge fill. For quantity estimation, additional fill placement of 0.5 m (i.e., second stage total 1.8 m of fill; 0.5 m top up and 1.3 m surcharge fill) for the EBL embankment should be anticipated. The results of the stability analysis for the proposed embankment geometry under the first stage and surcharge stage with the calculated strength gain of the cohesive soils is presented on Figures A24 and A25.

In this case, the up to 0.8 m thick organic deposit will be removed and replaced with granular fill to facilitate installation of the wick drains and the embankment can be constructed of rock fill.

Preliminary analysis using a wick drain spacing of 1.5 m in a triangular pattern to full depth through to the bottom of the cohesive deposit indicates that a wait period of about 10 months would be required for the first stage of embankment construction followed by a surcharge period of about 10 months for a total construction period of about 1.7 years. A detailed wick drain analysis has not been completed on this area and the preliminary analysis of the duration of the wait periods are derived from/based on undrained analyses. As such, the number of construction stages and wait period described above are approximate. The stages of construction, wait periods and estimated settlements are as follows:

| Stage         | Fill Thickness* | Wait Period | Settlement During Construction                              | Post Construction Settlement                         |
|---------------|-----------------|-------------|---|--|
| 1             | 5.7 m           | ~10 months  | 110 mm (immediate)<br>550 mm (primary)<br>40 mm (rock fill) | -  |
| 2 (surcharge) | 1.8 m**         | ~10 months  | 185 mm (primary)<br>20 mm (rock fill)                       | 0 mm (primary)<br>90 mm (creep)<br>10 mm (rock fill) |

\*After 0.8 m of peat has been sub-excavated and replaced with granular fill to the original ground surface.

\*\*Includes embankment top up.

After preloading and surcharging for about 1.7 years, the total post-construction settlement at the end of the surcharge period for the Highway 17 EBL embankment is estimated to be 100 mm (comprised of 90 mm creep settlement plus 10 mm long-term rock fill settlement). The result of the settlement analysis for staged construction for the EBL embankment is presented on Figure A26.

The main advantages of this alternative are the significant reduction in the size of the required toe berm to achieve stability compared to non-staged construction as well as the reduced delay in the schedule compared to options where wick drains are not incorporated. The main disadvantage of this option is the increased cost to install and monitor the wick drains and the associated construction wait periods due to the multiple stages.



Further, the actual wait periods are dependent on the results of the monitoring program and could be shorter or longer than estimated.

If wick drains are not utilized for this option, the first stage of construction will require a wait period of about 3 years to allow for dissipation of the pore pressures and achieve the strength increase necessary. The second wait period will require another wait period of about 3 years as the pore pressures dissipate and the necessary strength gain is achieved. The total delay time for construction would be about 6 years. The actual wait periods will be dependent on the results of the monitoring program and could be shorter or longer than estimated.

#### **6.5.1.2.3.2      Surcharging, Wick Drains with Toe Berms**

As an alternative to staged construction or the use of EPS, surcharging the Highway 17 EBL embankment, while incorporating wick drains into the clay stratum and a toe berm into the embankment, will achieve stable embankments and minimize the post-construction settlements and subsequent maintenance of the new roadway surface.

Surcharging the embankment increases the load on the underlying soils thereby reducing the delay time required to meet the settlement criterion. Wick drains provide a pathway for the dissipation of excess pore pressure in the cohesive deposit thereby further reducing the time required to meet the settlement criteria. The main disadvantages of this alternative is the large toe berm required (and thus property) on the south side of the embankment, the double handling of the surcharge material and the cost of designing and installing the wick drains, including an instrumentation and monitoring program.

A 2.0 m surcharge above the final profile grade will reduce the time required for preloading and decrease the long-term post-construction settlement including creep. In this case, the up to 0.8 m thick organic deposit will be removed and replaced with granular fill to facilitate installation of the wick drains and the embankments can be constructed out of rock fill.

The maximum stable embankment height for the Highway 17 EBL is 3.4 m. However, for the final embankment plus surcharge height of 7.7 m, a toe berm will be required to enhance stability of the surcharged embankment to a FoS equal to or greater than 1.3. The toe berm required to the south of the Highway 17 EBL embankment is 25 m wide and 2 m high above ground surface.

Preliminary analysis of consolidation of the cohesive deposit under the embankment enhanced by wick drains spaced at 1.5 m in a triangular pattern to full depth through to the bottom of the cohesive deposit indicates that 90 per cent of primary consolidation would be completed in about 10 months. After surcharging for 10 months, the total post-construction settlement is estimated to be 100 mm (90 mm creep settlement plus 10 mm long-term rock fill settlement).

To facilitate the assessment for the end of the surcharge period, instrumentation and monitoring during and after construction will be required. A detailed wick drain analysis has not been completed on this area and the preliminary analysis that the wait period is derived from in this report is based on undrained analyses. As such, the above number of stages and wait times are approximate. Further, the actual wait period is dependent on the results of the monitoring program and could be shorter or longer than estimated.





#### **6.5.1.2.3.3 Full Sub-Excavation and Preloading**

Taking into consideration the depth to the bottom of the clay deposit (i.e., up to about 10.4 m below the existing ground surface), full sub-excavation of the cohesive deposit is not considered practical for High Fill Area H1 due to: the potential requirement for specialized drag-line equipment; the substantial time and costs required to complete the sub-excavation and backfilling activities; the need for extra rock fill; the increase in long-term settlement of the thicker zone of rock fill; and the requirement for preloading for 12 months to reduce the long-term (post-construction) settlement. The full sub-excavation, if adopted, eliminates the risk associated with long-term consolidation settlement associated with the silty clay to clay deposit and the long-term settlement of the rock fill after a 12 month preload period is estimated to be 30 mm. It should be noted, however, that the cost of sub-excavation and backfilling may be of the same order of magnitude as some of the other options, including wick drains and staged construction, however, the time to conduct the sub-excavation operations and subsequent preloading may be longer.

#### **6.5.1.2.3.4 Partial Preloading and Lightweight Fill**

If there is not sufficient space to accommodate a toe berm, an alternative to reduce the post-construction settlement is to partially preload the embankment and incorporate lightweight (EPS) fill into the embankment mass. The Highway 17 EBL embankment can be constructed to the maximum stable embankment height of 3.4 m without the use of toe berms. After this partial preload period, the Highway 17 EBL embankment will be constructed to the final height by incorporating a 2.5 m thick zone of EPS into the Highway 17 EBL embankment.

After partial preloading for about 3 years (without using wick drains), the total post-construction settlement at the end of the partial preloading period for the Highway 17 EBL embankment is estimated to be 100 mm (comprised of 15 mm remaining primary settlement plus 75 mm creep settlement plus 10 mm long-term rock fill settlement).

The main disadvantage of this option is the substantial cost of EPS fill, which is typically an order of magnitude higher than other fill materials.

#### **6.5.1.2.3.5 Partial Preloading and Lightweight Fill and Wick Drains**

In order to reduce the time for consolidation for the alternative of partial preloading and lightweight fill, the use of wick drains has the added advantage of reducing the preload period from about 3 years to about 10 months. In this case, preloading for about 10 months at the maximum stable embankment height of 3.4 m (without the use of toe berms) will be required prior to installation of 2.5 m of EPS.

After partial preloading for about 3 years (using wick drains), the total post-construction settlement at the end of the partial preloading period for the Highway 17 EBL embankment is estimated to be 100 mm (comprised of 90 mm creep settlement plus 10 mm long-term rock fill settlement).

The main disadvantage of this option is the substantial cost associated with both the installation of wick drains and EPS fill.



#### **6.5.1.2.3.6      Surcharging with Toe Berms**

Surcharging the Highway 17 EBL embankment and incorporating a toe berm into the south side of the embankment will achieve a stable embankment and minimize the post-construction settlements and subsequent maintenance of the new roadway surface.

If surcharging is used without the incorporation of wick drains into the foundation cohesive deposit, a surcharge period of about 2.1 years will be required to reduce the post-construction settlement to the settlement criterion. The estimated total post-construction settlement at the end of the 2.1 year surcharge period is 100 mm (90 mm creep settlement plus 10 mm long-term rock fill settlement). To facilitate the assessment for the end of the surcharge period, instrumentation and monitoring during and after construction will be required.

As discussed in Section 6.5.1.2.3.2, the maximum stable embankment height is significantly lower than the final surcharged embankment height and toe berms are required. The toe berm required to the south of the Highway 17 EBL embankment is 23 m wide and 2 m high above ground surface.

The main disadvantages of this option are the wait time required for surcharging without the use of wick drains and the requirement of large toe berms which may require the purchase of additional property. If the length of the construction schedule is not a consideration this would be the most cost effective option.

#### **6.5.1.2.3.7      Other Mitigation Options**

The option of partial sub-excavation is considered not feasible since in this case, wick drains would not be able to be installed through the rock fill backfill and thus, full preloading to reduce post-construction settlement of the remaining clay deposit would still be required. As a minimum, a 12 month preload period would be required for rock fill compression.

The option of ground improvement, consisting of either dry/wet soil mixing or rammed aggregate piers (geopiers) is also considered not feasible since the amount of cement or aggregate required would result in costs far exceeding other options. Further, additional design and bench scale testing would be required to determine the ultimate feasibility of these options and it may be that there would be insufficient strength gain of the clay deposit to make the soil mixing option feasible.

### **6.5.2      Highway 17 WBL – STA 13+140 to 13+390 Highway 17 EBL – STA 13+140 to 13+390 St. Pothier Road – STA 9+400 9+600 High Fill H2**

Between STA 13+140 and 13+390, the WBL and EBL embankments will require a fill embankment up to 5.0 m high above the existing ground surface to achieve the vertical highway profile. The St. Pothier Road embankment will require a fill embankment up to 4.4 m high above the existing ground surface to achieve the vertical highway profile. These three alignments extend across a low lying swampy area with occasional areas of ponded water. A watercourse runs through the middle of the swamp from north to south. Considering the close proximity of these three embankments to one another, that all three of these embankments are on new alignments and the foundation benefits of building these embankments concurrently, these embankments have been combined into one high fill/swamp area (H2).

The subsurface soils along the H2 alignments generally consist of a surficial layer of peat/organics up to 3.6 m thick, underlain by a cohesive deposit of clayey silt transitioning into varved silty clay to clay transitioning to





clayey silt to silt, which in turn is underlain by deposits of silt to sand to gravelly sand. The cohesive deposit is up to about 10 m thick in some locations and extends to depths up to 13.5 m below ground surface. Resistance to dynamic cone penetration and borehole advancement was encountered at depths of up to 20.3 m below ground surface. In some boreholes, typically near the middle of the swamp, refusal was not encountered at depths of up to 25.0 m below ground surface. Details of the subsurface conditions at the Highway 17 WBL, Highway 17 EBL and St. Pothier Road embankments are presented in Sections 4.5, 4.6 and 4.7, respectively, and the stratigraphic profiles are shown on Drawings B1 to B6 in Appendix B.

The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 3. Additional details of foundation engineering parameters employed for the cohesive deposits (i.e., clayey silt/silty clay/clay) encountered in H2 are presented on Figure B25 in Appendix B. The piezometric condition used in the analyses is the water table at the existing ground surface.

The critical sections for stability and settlement are located at about STA 13+260 along the Highway 17 WBL and EBL embankments and at about STA 9+480 along the St. Pothier Road embankment, where the proposed embankment heights are 4.4 m, 4.4 m and 2.9 m high, respectively. The cohesive deposit is about 5.5 m, 9.7 m and 9.8 m thick at the WBL, EBL and St. Pothier Road embankment locations, respectively. Although the maximum embankment height was not encountered at this critical section, the thickness of the cohesive deposit governs the results of the analysis for both stability and settlement.

#### **6.5.2.1 Stability**

The stability analyses carried out at the critical section indicates that after completion of construction (including removal and replacement of the up to 3.6 m thick organic deposit with rock fill), the embankments will have a Factor of Safety (FoS) less than 1.3 for a deep-seated, global failure surface that would impact the operation of the highway. Therefore, stability mitigation is required in High Fill H2.

#### **6.5.2.2 Settlement**

The settlement analysis carried out at the critical section, estimates the short-term settlement of the foundation soils to be 690 mm. This estimated settlement of the Highway 17 EBL embankment is comprised of about 75 mm of immediate settlement due to compression of the cohesionless deposits and about 615 mm of primary consolidation of the 9.7 m thick cohesive deposit.

Based on an average coefficient of consolidation ( $c_v$ ) of about  $2.0 \times 10^{-3} \text{ cm}^2/\text{s}$  estimated for the cohesive deposit, the imposed loading conditions for the approximately 4.4 m high rock fill embankment (EBL) and assuming two-way drainage of the 9.7 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 3.2 years.

The magnitude of secondary consolidation (creep) settlement for the cohesive deposit is expected to be about 90 mm per log-cycle of time for this area corresponding to about 70 mm over a 20-year period following completion of construction.

In addition, the total settlement of the rock fill embankment itself (based on a 4.4 m high embankment at the critical section plus 3.6 m of rock fill backfill) is estimated to be about 70 mm, with about 50 mm expected to

occur within six months of construction of the embankment, 10 mm occurring during the next six months and about 10 mm expected to occur over the remaining design life of the embankment.

The estimated total settlement of the St. Pothier Road embankment is comprised of about 80 mm of immediate settlement due to compression of the cohesionless deposits and about 425 mm of primary consolidation of the 9.8 m thick cohesive deposit. These values assume that removal and replacement of the up to 3.6 m thick organic deposit with rock fill has been carried out.

Based on an average coefficient of consolidation ( $c_v$ ) of about  $2.0 \times 10^{-3} \text{ cm}^2/\text{s}$  estimated for the cohesive deposit, the imposed loading conditions for the approximately 2.9 m high rock fill embankment (St. Pothier Road) and assuming two-way drainage of the 9.8 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 3.2 years.

The magnitude of secondary consolidation (creep) settlement for the cohesive deposit is expected to be about 90 mm per log-cycle of time for this area corresponding to about 70 mm over a 20-year period following completion of construction.

In addition, the total settlement of the rock fill embankment itself (based on a 2.9 m high embankment at the critical section plus 3.6 m of rock fill backfill) is estimated to be about 60 mm, with about 45 mm expected to occur within six months of construction of the embankment, 5 mm occurring during the next six months and about 10 mm expected to occur over the remaining design life of the embankment.

Since the total post-construction settlement (primary, creep and rock fill) exceeds the settlement criterion of 100 mm for the EBL and WBL and 200 mm for the St. Pothier Road embankment, settlement mitigation measures are required for the embankments in High Fill H2.

#### **6.5.2.3     *Mitigation of Stability Issues and/or Time Dependent Settlements: Embankments Built Concurrently***

The presence of the up to about 10 m thick cohesive deposit influences both the stability and the settlement of the 4.4 m high embankments at the critical section. In order to construct the embankments to achieve a FoS greater than or equal to 1.3, and to minimize post-construction settlements, the alternatives presented below can be considered.

The mitigation options discussed in this section are all based on the assumption that all three embankments are built concurrently. Constructing the three embankments concurrently greatly improves the global stability of the embankments during construction. In particular, the St. Pothier Road embankment would essentially act as a toe berm to support the Highway 17 EBL embankment - without the St. Pothier Road embankment, a temporary toe berm extending across the width of St. Pothier Road would be required to maintain the stability of the Highway 17 EBL until the St. Pothier Road embankment is constructed. For practical purposes, we recommend building the embankments concurrently, as detailed in the alternatives below.

The alternatives described have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table B1 in Appendix B. A summary of the settlement results for each alternative is provided in Table 4.

Section 6.5.2.4 discusses the mitigation scenarios where the St. Pothier Road embankment is constructed after the Highway 17 EBL and WBL have been constructed and the mitigation alternatives are ranked in Table B2 in Appendix B.

Given the thick cohesive deposit (the bottom of which is up to 13.5 m below ground surface) and the associated magnitude of primary consolidation settlement (about 615 mm) of the foundation soils under a 4.4 m high embankment, surcharging with wick drains and incorporating toe berms along the embankments into a staged construction schedule is considered the preferred mitigation alternative, provided all three embankments are constructed concurrently.

#### **6.5.2.3.1      Surcharging, Wick Drains and Staged Construction with Toe Berms**

We recommend a staged construction approach with surcharging that incorporates wick drains installed into the cohesive stratum and toe berms supporting the embankments, as the preferred, most technically feasible mitigation measure to achieve stable embankments and to minimize the post-construction settlements and subsequent maintenance of the new roadway surface. The staged construction approach relies on strength gain in the underlying cohesive deposit to enhance the global stability of the embankment during subsequent stages. The first stage could be constructed up to 4.4 m high with 2 m high by 10 m wide toe berms along the north side of the Highway 17 WBL embankment and a 2 m high toe berm connecting the Highway 17 EBL embankment to the St. Pothier Road embankment. During the first stage, the St. Pothier Road embankment must be constructed up to 2.9 m high with a 2 m high by 10 m wide toe berm along the south side of the embankment. The first stage of construction includes the installation of instrumentation to monitor settlement and pore pressure dissipation, and requires a wait period to allow for dissipation of the pore pressures and to achieve a strength increase of the cohesive deposit (estimated to be up to about 16 kPa under the EBL embankment). The actual wait period will be dependent on the results of the monitoring.

After the first wait period is complete, the second stage of construction consisting of a 2.0 m surcharge on the Highway 17 WBL and EBL embankments and a 1.0 m surcharge on the St. Pothier Road embankment, can be completed and a second wait period will be required to allow for dissipation of the pore pressures of the cohesive deposit to satisfy the settlement criterion. The analysis assumes that the embankment will be "topped up" in the second stage to the final grade prior to placing the surcharge fill. For quantity estimation, additional fill placement of 0.7 m (i.e., second stage total 2.7 m of fill; 0.7 m top up and 2.0 m surcharge fill) for the WBL/EBL embankments should be anticipated. For the St. Pothier Road embankment, 0.5 m of top up should be anticipated for a total second stage fill of 1.5 m. The results of the stability analysis for the proposed embankment geometry under the first stage and surcharge stage with the calculated strength gain of the cohesive soils is presented on Figures B26 and B27 for the WBL embankment and on Figures B28 and B29 for the and St. Pothier Road, respectively.

In this case, the up to 3.6 m thick organic deposit will be removed and replaced with granular fill to facilitate installation of the wick drains and the embankments can be constructed out of rock fill.

Preliminary analysis using a wick drain spacing of 1.5 m in a triangular pattern to full depth through to the bottom of the cohesive deposit indicates a wait period of about 10 months would be required for the first stage of embankment construction followed by a surcharge period of about 10 months for a total construction period of about 1.7 years. A detailed wick drain analysis has not been completed for this area and the preliminary analysis to estimate the wait periods presented in this report are based on undrained analyses. As such, the

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number of stages and wait periods described above are approximate. The stages of construction, wait periods and estimated settlement are as follows:

| Embankment       | Stage         | Fill Thickness* | Wait Period | Settlement During Construction                             | Post Construction Settlement                          |
|------------------|---------------|-----------------|-------------|--|---|
| EBL/WBL          | 1             | 4.4 m           | ~10 months  | 75 mm (immediate)<br>580 mm (primary)<br>40 mm (rock fill) | -   |
|                  | 2 (surcharge) | 2.7 m**         | ~10 months  | 240 mm (primary)<br>20 mm (rock fill)                      | 0 mm (primary)<br>95 mm (creep)<br>5 mm (rock fill)   |
| St. Pothier Road | 1             | 2.9 m           | ~10 months  | 80 mm (immediate)<br>410 mm (primary)<br>40 mm (rock fill) | -   |
|                  | 2 (surcharge) | 1.5 m**         | ~10 months  | 75 mm (primary)<br>20 mm (rock fill)                       | 65 mm (primary)<br>130 mm (creep)<br>5 mm (rock fill) |

\*After 3.6 m of peat has been sub-excavated and replaced with granular fill to the original ground surface.

\*\*Includes embankment top up.

After preloading and surcharging for about 1.7 years, the total post-construction settlement at the end of the surcharge period for the Highway 17 WBL and EBL embankments is estimated to be 100 mm (comprised of 95 mm creep settlement plus 5 mm long-term rock fill settlement). The result of the settlement analysis for staged construction for the EBL embankment is presented on Figure B30.

After preloading and surcharging for about 1.7 years, the total post-construction settlement at the end of the surcharge period for the St. Pothier Road embankment is estimated to be 200 mm (comprised of 65 mm remaining primary settlement plus 130 mm creep settlement plus 5 mm long-term rock fill settlement). The result of the settlement analysis for staged construction for the St. Pothier Road embankment is presented on Figure B31.

The main advantages of this alternative are the significant reduction in the size of the required toe berms to achieve stability as compared to non-staged construction, as well as the shorter schedules for the first and second stages as compared to options where wick drains are not incorporated. The main disadvantage of this option is the increased cost to install and monitor the wick drains and the associated construction wait periods due to the multiple stages. Further, the actual wait periods are dependent on the results of the monitoring program and could be shorter or longer than estimated.

If wick drains are not utilized for this option, the first stage of construction will require a wait period of about 2.5 years to allow for dissipation of the pore pressures and achieve the strength increase necessary. The second wait period will require another wait period of about 2.5 years as the pore pressures dissipate and the necessary strength gain is achieved. The total delay time for construction would be about 5 years. The actual wait periods will be dependent on the results of the monitoring program and could be shorter or longer than estimated.



#### **6.5.2.3.2 Full Sub-Excavation and Preloading**

Taking into consideration the depth to the bottom of the clay deposit (i.e., up to about 13.5 m below the existing ground surface), full sub-excavation of the cohesive deposit is not considered practical for High Fill H2 due to: the potential requirement for specialized drag-line equipment; the substantial time and costs required to complete the sub-excavation and backfilling activities; the need for extra rock fill; the increase in long-term settlement of the rock fill; and the requirement for preloading for 12 months to reduce the long-term (post-construction) settlement. The full sub-excavation alternative, if adopted, eliminates the risk associated with long-term consolidation settlement associated with the silty clay to clay deposit and the long-term settlement of the rock fill after a 12 month preload period is estimated to be 30 mm. It should be noted, however, that the cost of sub-excavation and backfilling may be of the same order of magnitude and likely greater as some of the other options, including wick drains and staged construction as no off-site rock fill would have to be supplied to site at an added cost, however, the time to conduct the sub-excavation operations and subsequent preloading would likely be longer.

#### **6.5.2.3.3 Partial Preloading and Lightweight Fill**

If there is insufficient space to accommodate toe berms, an alternative to reduce the post-construction settlement is to partially preload the embankment and incorporate lightweight (EPS) fill into the embankment mass. The Highway 17 WBL embankment can be constructed to the maximum stable embankment height of 3.1 m and the St. Pothier Road embankment can be constructed to the maximum stable height of 2.5 m without the use of toe berms. Assuming the St. Pothier Road embankment will be built at the same time as the Highway 17 WBL and EBL embankments and as such would act as a "toe berm" for the Highway 17 EBL embankment, a 4.0 m surcharge could be constructed on the Highway 17 EBL embankment. After the partial preloading of the WBL and St. Pothier Road and the surcharging of the EBL, the Highway 17 WBL and St. Pothier Road embankments would be constructed to the final height by incorporating a 1.5 m and 1.0 m thick zone of EPS into the embankments, respectively. The EBL embankment would not require the use of EPS due to the large surcharge which could be partially removed to achieve the final embankment height.

In this case, the up to 3.6 m thick organic deposit should be removed and replaced with rock fill. In areas where EPS is required (WBL and St. Pothier Road embankments), the embankments should be constructed out of granular fill to facilitate placement of the EPS. The EBL embankment should be constructed of rock fill. The appropriate side slopes should be utilized as per Section 6.2.1.

After partial preloading for about 1.8 years (without using wick drains), the total post-construction settlement at the end of the partial preloading period for the Highway 17 WBL embankment is estimated to be 100 mm (comprised of 30 mm remaining primary settlement plus 65 mm creep settlement plus 5 mm settlement of the above ground rock fill). At the end of the partial preloading period for the St. Pothier Road embankment the total post-construction settlement is estimated to be 200 mm (comprised of 120 mm remaining primary settlement plus 75 mm creep settlement plus 5 mm long-term rock fill settlement).

After surcharging the Highway 17 EBL embankment for about 1.8 years, the total post-construction settlement for the Highway 17 EBL is estimated to be 100 mm (comprised of 20 mm remaining primary and 75 mm creep settlement and 5 mm long-term rock fill settlement).

The main disadvantage of this option is the high cost of EPS fill, which is typically an order of magnitude higher than other fill materials.



#### **6.5.2.3.4      Surcharging, Wick Drains with Toe Berms**

As an alternative to staged construction or the use of EPS, surcharging the Highway 17 WBL, EBL and St. Pothier Road embankments, while incorporating wick drains into the clay stratum and constructing toe berms to support the embankments, would achieve stable embankments and minimize the post-construction settlements and subsequent maintenance of the new roadway surface.

Surcharging the embankment increases the load on the underlying soils thereby reducing the delay time required to meet the settlement criterion. Wick drains provide a pathway for the dissipation of excess pore pressure in the cohesive deposit thereby further reducing the time required to meet the settlement criteria. The main disadvantages of this alternative is that large toe berms would be required (and thus property may need to be acquired), the double handling of the surcharge material and the cost of designing and installing the wick drains, including an instrumentation and monitoring program.

A 2.0 m surcharge above the final profile grade for the WBL and EBL embankment and a 1.0 m surcharge for the St. Pothier Road embankment would reduce the time required for preloading and decrease the long-term post-construction settlement including creep. In this case, the up to 3.6 m thick organic deposit would be removed and replaced with granular fill to facilitate installation of the wick drains and the embankments can be constructed out of rock fill.

The maximum stable embankment height without toe berms is 2.9 m for the Highway 17 WBL, for the Highway 17 EBL is 4.4 m and for the St. Pothier Road embankment it is 2.5 m. The embankment plus surcharge height for each embankment is 6.4 m, 6.4 m and 3.9 m, respectively, therefore toe berms would be required to improve stability of the surcharged embankment to achieve a FoS equal to or greater than 1.3. The toe berm required on the north side of the Highway 17 WBL embankment would be 14 m wide and 1.5 m high and the toe berm required on the south side of the St. Pothier Road embankment would be 22 m wide and 1.5 m above ground surface. Also a "toe berm" about 1.5 m above ground surface that connects the Highway 17 EBL and the St. Pothier Road embankments would be required.

Preliminary analysis of consolidation of the cohesive deposits under the three embankments enhanced by wick drains spaced at 1.5 m in a triangular pattern to full depth through to the bottom of the cohesive deposit indicates that 90 per cent of primary consolidation would be completed in about 10 months.

After surcharging for 10 months, the total post-construction settlement of the EBL embankment is estimated to be 100 mm (95 mm creep settlement plus 5 mm long-term rock fill settlement).

After surcharging the St. Pothier Road embankment for 10 months, the total post-construction settlement is estimated to be 200 mm (50 mm primary, 130 mm creep settlement plus 20 mm short and long-term rock fill settlement).

To facilitate the assessment for the end of the surcharge period, instrumentation and monitoring during and after construction would be required. A detailed wick drain analysis has not been completed for this area and the preliminary analysis used to estimate the wait periods presented in this report is based on undrained analyses. As such, the above number of stages and wait times are approximate. Further, the actual wait period is dependent on the results of the monitoring program and could be shorter or longer than estimated.



#### 6.5.2.3.5 Surcharging with Toe Berms

Surcharging the Highway 17 WBL, EBL and St. Pothier Road embankments and incorporating toe berms into the embankments will achieve stable embankments and minimize the post-construction settlements and subsequent maintenance of the new roadway surface. However, without the use of toe berms, the surcharging would take longer.

The maximum stable embankment height without toe berms is 3.3 m for the Highway 17 WBL, for the Highway 17 EBL is 2.4 m and for the St. Pothier Road embankment it is 2.9 m. The embankment plus surcharge height for each embankment is 6.4 m, 6.4 m and 3.9 m, respectively, therefore toe berms would be required to improve the stability of the surcharged embankment to achieve a FoS equal to or greater than 1.3. The toe berm required on the north side of the Highway 17 WBL embankment would be 12 m wide and 1.5 m high and the toe berm required on the south side of the St. Pothier Road embankment would be 20 m wide and 1.5 m above ground surface. Also a "toe berm" about 1.5 m above ground surface that connects the Highway 17 EBL and the St. Pothier Road embankments would be required.

In this case, the up to 3.6 m thick organic deposit will be removed and replaced with granular fill to facilitate installation of the wick drains and the embankments can be constructed out of rock fill.

After surcharging the WBL and EBL embankments for 2.5 years, the total post-construction settlement of the EBL embankment is estimated to be 100 mm (10 mm primary, 80 mm creep settlement plus 10mm long-term rock fill settlement).

After surcharging the St. Pothier Road embankment for 1.5 years, the total post-construction settlement is estimated to be 200 mm (110 mm primary, 80 mm creep settlement plus 10 mm long-term rock fill settlement).

To facilitate the assessment for the end of the surcharge period, instrumentation and monitoring during and after construction would be required.

The main disadvantages of this option are the wait time required for surcharging without the use of wick drains and the requirement for large toe berms which may require the purchase of additional property. If the length of the construction schedule is not a consideration this would be the most cost effective option.

#### 6.5.2.3.6 Other Mitigation Options

The option of partial sub-excavation is considered not feasible since in this case, wick drains would not be able to be installed through the rock fill backfill and thus full preloading to reduce post-construction settlement of the remaining clay deposit would still be required. As a minimum, a 12 month preload period would be required for rock fill compression.

The option of ground improvement, consisting of either dry/wet soil mixing or rammed aggregate piers (geopiers) is also considered not feasible since the amount of cement or aggregate required would result in costs far exceeding other options. Further, additional design and bench scale testing would be required to determine the ultimate feasibility of these options and it may be that there would be insufficient strength gain of the clay deposit to make the soil mixing option feasible.

#### **6.5.2.4     *Mitigation of Stability Issues and/or Time Dependent Settlements: Highway 17 EBL/WBL Embankments Built Independently from the St. Pothier Road Embankment***

Construction of the three embankments concurrently is critical to maintaining stability of the embankments and is also a major factor in choosing the preferred mitigation alternative. Essentially, the St. Pothier Road embankment will act as a “toe berm” for the south side of the Highway 17 EBL embankment. If the St. Pothier Road embankment is not constructed concurrently with the Highway 17 embankments, then the global stability of the proposed 4.4 m high EBL embankment would be compromised. A toe berm would be required on the south side of the Highway 17 EBL embankment with approximately the same dimensions as the proposed St. Pothier Road embankment (i.e., extending over 25 m wide) to support the EBL embankment. Therefore, the number of technically feasible options is reduced when only the Highway 17 EBL and WBL embankments are considered.

As discussed in Section 6.5.2.3, we recommend surcharging in combination with wick drains and toe berms along the embankments integrated into a staged construction schedule as the preferred mitigation alternative, provided all three embankments are constructed concurrently. We do not recommend constructing the St. Pothier Road embankment at a later time as both stability and long-term settlement of the new EBL/WBL embankments would affect roadway performance and likely construction itself. Further, the costs associated with mitigation of all the embankments would be excessive.

If, however, it is not possible to construct the St. Pothier Road embankment at the same time as the Highway 17 EBL and WBL embankments, then the alternative of partial preloading with lightweight fill (EPS) is considered the only technically feasible option to mitigate both the settlement and stability concerns related to the construction of the EBL embankment. Further, if the St. Pothier Road embankment is constructed at a later date, settlement of the cohesive deposit under the new loading would induce settlement under the EBL embankment.

##### **6.5.2.4.1     Partial Preloading and Lightweight Fill**

If the construction schedule does not allow for all three embankments to be built concurrently the only technically feasible alternative to reduce the post-construction settlement while maintaining stability is to partially preload the Highway 17 WBL and EBL embankments and to incorporate lightweight (EPS) fill into the embankment mass. The Highway 17 WBL and EBL embankments can be constructed to the maximum stable embankment height of 2.2 m without the use of toe berms. Partial preloading at this embankment height for 1.8 years would be required to allow for sufficient consolidation settlement of the cohesive deposit and settlement of the rock fill prior to the construction of the pavement structure. After the partial preloading of the WBL and EBL, the embankments could be constructed to the final height by incorporating a 3.0 m thick zone of EPS into the embankments.

After partial preloading for about 1.8 years (without using wick drains) the total post-construction settlement in the Highway 17 WBL is estimated to be 100 mm (comprised of 30 mm remaining primary and 65 mm creep settlement and 5 mm long-term rock fill settlement) and the post-construction settlement in the Highway 17 EBL is estimated to be 100 mm (comprised of 20 mm remaining primary and 75 mm creep settlement and 5 mm long-term rock fill settlement).

The St. Pothier Road embankment can subsequently be constructed to the maximum stable embankment height of 2.5 m without the use of toe berms. Partial preloading at this embankment height can then be carried out for 1.8 years to allow for sufficient consolidation settlement of the cohesive deposit and settlement of the rock fill to occur prior to the construction of the pavement structure. A 1.0 m thick zone of EPS would then be required to raise the embankment to the design grade. After completion of the preload period, the total post-construction settlement is estimated to be 200 mm (comprised of 120 mm remaining primary and 75 mm creep settlement and 5 mm long-term rock fill settlement).

The main disadvantage of this option is the substantial cost of EPS fill, which is typically an order of magnitude higher than other fill materials. The construction of toe berms could reduce the amount of EPS material currently specified. Also, wait times could be reduced by incorporating more EPS material into the embankment or utilizing wick drains.

### **6.5.3 Highway 17 WBL – STA 13+900 to 14+200 Highway 17 EBL – STA 13+900 to 14+200 High Fill H3**

Due to the close proximity and overlap of the realigned Highway 17 WBL and EBL embankments and the existing Highway 17 WBL and EBL embankments, these embankments areas have been combined into one high fill/swamp area H3.

Generally, the proposed Highway 17 WBL and EBL embankments overlap the existing Highway 17 WBL and EBL embankments throughout the high fill area with essentially a widening of the existing embankments of up to about 7 m to the north of both the WBL and EBL embankments between STA 13+950 and 14+150. Between about STA 13+900 and 13+950 and between about STA 14+150 and 14+200, there will either be a cut or minimal to no grade raise. Between about STA 13+950 and 14+150, an embankment widening up to about 7 m and a grade raise up to 3.1 m above the existing embankment will be required to achieve the horizontal and vertical highway profile.

The subsurface soils along the WBL and EBL alignments in High Fill Area H3 consist of surficial layers of peat/topsoil or asphalt and embankment fill, underlain by an upper deposit of sand to sandy silt. These upper deposits are underlain by the main cohesive deposit of clayey silt transitioning into varved silty clay to clay transitioning to clayey silt to silt underlain by a silt deposit, which is further underlain by a deposit of sand to sand and silt. The cohesive deposit is up to 16.8 m thick at some locations and extends to depths up to 19.5 m below ground surface. Resistance to dynamic cone penetration and borehole advancement was encountered at depths of up to 28.0 m ground surface. Details of the subsurface conditions for this swamp crossing area are presented in Sections 4.8 and 4.9 and shown on Drawings C1 to C4 in Appendix C.

The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 3. Additional details of foundation engineering parameters employed for the cohesive deposits (i.e., clayey silt/silty clay/clay) encountered in H3 are provided on Figure C22 in Appendix C. The piezometric condition used in the analyses is the water table just below the top of the native organic material (Elevation 241.6 m).

The critical section used in the analysis is located at approximately STA 14+100, where the existing embankments are to be widened thus requiring grade raises at the existing north crest of slope of 1.9 m and 3.1 m above the existing ground surface at the toes of the existing WBL and EBL embankments, respectively. At



STA 14+100, the grade raises will be approximately 0.8 m and 1.1 m above the existing highway grade at the new centrelines of the WBL and EBL, respectively. Boreholes drilled in the WBL and EBL embankments encountered up to 4.7 m and 5.5 m of existing fill, respectively. At this location, the cohesive deposit is up to 14.0 m thick.

#### **6.5.3.1 Stability**

At the critical section as described above, the FoS for slope stability would be greater than 1.3 for granular fill or rock fill embankments constructed on the existing fill and subsoils, provided that all organics have been removed below the widened embankment footprint, as shown on Figures C23 and C24 in Appendix C.

#### **6.5.3.2 Settlement**

To estimate the magnitude of the expected settlements due to new construction, analysis was carried out at the critical section representative of the subsurface conditions within the high fill area, at approximately STA 14+100. The critical section was chosen in the area of the largest embankment widening, which corresponds to the largest stress loading on the subsoils. At the north side of the EBL which is most critical for settlement, the widening of about 6 m creates a grade raise of 3.1 m above the existing ground surface at the crest of the slope and a grade raise of about 1.1 m above the 5.6 m of existing fill at the proposed EBL centreline.

Based on the results of the settlement analysis for the EBL embankment widening, the short-term settlement of the foundation soils under approximately the new outside edge of pavement (assumed to be the edge of the travelled lane in this analysis) is estimated to be about 115 mm. This estimated settlement in the Highway 17 WBL is comprised of about 25 mm of immediate settlement due to compression of the cohesionless deposits and about 90 mm of primary consolidation of the 14.0 m thick cohesive deposit.

Based on an average coefficient of consolidation ( $c_v$ ) of about  $2.0 \times 10^{-3} \text{ cm}^2/\text{s}$  estimated for the cohesive deposit, the imposed loading conditions for the approximately 3.1 m grade raise and assuming two-way drainage of the 14 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 6 years.

The magnitude of secondary consolidation (creep) settlement for the cohesive deposit is expected to be about 100 mm per log-cycle of time for this area corresponding to about 60 mm over a 20-year period following completion of construction (i.e., from 6 years to 20 years).

If rock fill is utilized for the embankment widening, the total settlement of the rock fill embankment itself (based on 3.1 m grade raise at the critical section) is estimated to be about 25 mm, with about 15 mm expected to occur within six months of construction of the embankment, 5 mm occurring during the next six months and about 5 mm expected to occur over the remaining design life of the embankment. If granular fill is utilized, settlement of properly placed and compacted granular fill is estimated to occur quickly during construction with no post-construction settlement.

Since the total post-construction settlement is estimated to be about 175 mm (comprised of 90 mm primary consolidation, 60 mm of creep and 25 mm rock fill settlement – if utilized), and exceeds the settlement criterion of 50 mm for an embankment widening, settlement mitigation measures are required for the Highway 17 WBL.



and EBL embankments in the High Fill Area H3. Further, the estimated differential settlement of 40 mm between the new centreline and north edge of pavement also exceeds the criterion of 200:1. It should be noted that higher magnitudes of total and differential settlement are expected to occur beyond the outside edge of pavement along the slope.

#### **6.5.3.3 Mitigation of Stability Issues and/or Time Dependent Settlements**

In order to construct the embankments in the widened areas and minimize post-construction settlements, the alternatives presented below can be considered. The alternatives described have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table C1 in Appendix C. A summary of the results of settlement analysis for each alternative is provided in Table 4.

Outside of the critical areas, there will be a minimal grade raise and little to no embankment widening and therefore no settlement or stability mitigation will be necessary. The sections listed below do not require settlement or stability mitigation:

- Highway 17 WBL and EBL – STA 13+900 to 13+950;
- Highway 17 EBL – STA 14+150 to 14+200; and
- Highway 17 WBL, – STA 14+175 to 14+200.

The areas not listed above will require settlement mitigation. Given the thick cohesive deposit (the bottom of which is up to about 19.5 m below ground surface), the associated magnitude of primary and secondary consolidation settlement (150 mm) and 40 mm of expected differential settlement of the foundation soils under a 3.1 m grade raise in the widened area, the most practical method of construction is to preload the embankment widening, allowing settlement to occur while the traffic is using the widened highway section, followed by maintenance of the roadway in the future. This method does not meet the MTO settlement criteria for post-construction settlement but is more practical from a cost and a construction standpoint compared to other technically feasible options.

#### **6.5.3.3.1 Consolidation and Maintenance**

We recommend a construction approach that involves constructing (widening and raising) the embankments to their final geometry, utilizing the embankments as the travelled highway and then conducting ongoing maintenance, as may be required to re-grade the highway to accommodate the estimated 80 mm of settlement (approximately 90 per cent of the primary consolidation settlement) expected to occur over about 6 years. While this construction option does not meet the MTO settlement criteria in the short term, it is still considered the most practical option given that the magnitude of total settlement at the new outside north edge of the EBL pavement is about 150 mm (90 mm primary and 60 mm creep). The widened embankment will remain stable while in use as a travelled lane but the expected post constructed settlements will require maintenance. This alternative relies on the fact that while the expected settlements exceed the MTO settlement criteria for a widened embankment (Figure 3, MTO 2010), the embankment can still function as a travelled lane while consolidation takes place.

While this construction approach does not strictly require the installation of instrumentation to monitor for settlement, it would be prudent and it is recommended that settlement be monitored, however, it is not required to monitor pore pressures for embankment stability. The embankment monitoring can consist of a series of settlement plates installed within the embankment at the crest of the widened portions of the embankments provided guide rail installation is not impacted and the monitoring points remain accessible. Settlement plates in the shoulder or on the slope (or nail pins in pavement) will be required to be installed along the full length of the high fill section on the south side of the widened embankment at offsets to be determined once the final cross sections are known. For a 300 m long section, settlement plates at approximately 50 m spacing would be appropriate. Monthly readings of the settlement plates are recommended for the first year of monitoring and quarterly readings afterwards until the decrease in the rate of settlement indicates that the remaining settlement is within the MTO criteria, likely up to 6 years.

The total post-construction settlement (i.e., after construction of the widened embankments is complete and traffic is using the lanes) for the Highway 17 EBL embankment expected during the consolidation period is estimated to be 150 mm (comprised of 90 mm of primary settlement plus 60 mm creep settlement plus fill settlement) which exceeds the settlement criteria. The differential settlement of 40 mm between the new centreline and new outside edge of pavement lane also exceeds the criterion of 200:1. During the approximately 6 year consolidation period, approximately 80 mm of settlement will have occurred on the new outside edge of pavement and approximately 50 mm at the new highway centreline. The estimated total and differential settlement after approximately 6 years is about 50 mm and 5 mm at the new outside edge of pavement, respectively, which meet the MTO criteria. The results of the analysis are shown on Figure C25.

The main advantages of this option are that there is neither a delay in the construction schedule nor any need to divert traffic for multiple years during staged construction. The disadvantages of this option are that it does not meet the MTO settlement criteria and future highway maintenance is likely to be required.

#### 6.5.3.3.2 Lightweight Fill

In order to meet the settlement criteria in the short term, lightweight (EPS) fill could be incorporated into the widened embankment mass. The Highway 17 WBL and EBL embankments can be widened to the north using a 1 m thick layer of EPS under the pavement structure of the new embankment (full width, not to interfere with guide rail installation) and an additional 1 m of EPS (for a total of 2 m) would be required within the remainder of the north half of the new embankment. This will require sub-excavation of the existing fill to accommodate the EPS. The EPS should be stepped in 0.3 m to 0.5 m increments across the embankment and in the taper zones longitudinally along the highway. Essentially, with the incorporation of this volume of EPS into the embankment mass, the induced settlement will be minimal and will not result in creep settlement. Total and differential post-construction settlement is estimated to be less than 10 mm.

The main advantages of this option are that it meets the post-construction settlement criteria in the short term and does not create delays in the construction schedule. The disadvantage of this option is the substantial cost of EPS fill, which is typically an order of magnitude higher than other fill materials, as well as the need to excavate a portion of the existing embankments in order to install the EPS. Generally, this option is not considered practical due to the added cost of material compared to the consolidation and maintenance option.

## 6.6 Subgrade Preparation and Embankment Construction

The following sections discuss general aspects of subgrade preparation and embankment construction for the high fill/swamp crossing areas, including: removal of organic materials; excavation and replacement of soft, cohesive deposits; groundwater control; placement of embankment fills, slope flattening and platform widening.

A summary of the recommended/preferred foundation mitigation option for each high fill/swamp crossing area is presented in Table 5. The summary contains: recommendations on embankment fill types and side slope profiles; estimated maximum depth of organic deposits encountered; the magnitude of estimated settlement (during and post-construction) for the embankment materials and the foundation soils; recommended width of platform widening as may be required to accommodate future raising of the embankment; and the recommended Ontario Provincial Standard Drawings (OPSD) excavation guideline.

### 6.6.1 Removal of Organic Deposits

Based on the subsurface information from the boreholes advanced during the field investigation, the thickness of organic deposits (i.e., peat and topsoil) in the proposed Highway 17 alignment generally ranges from about 0.1 m to 4.0 m, as presented in Table 3. After clearing and grubbing the high fill/swamp crossing areas and prior to the placement of any fill for the new construction, all organic deposits should be stripped from the plan limits of the proposed works, including toe berms, if applicable. The organic materials should be removed using construction procedures in accordance with OPSS 209 (Embankments Over Swamps and Compressible Soils) where the removal and backfilling operations are carried out simultaneously. An NSSP for excavation of organics should be included in the contract documents and an example is included in Appendix D.

In areas where the new embankments are being constructed away from existing embankments, the excavation limits should be consistent with OPSD 203.010 (Embankments Over Swamp, New Construction, modified to remove the restrictions on the height of the embankment and the depth of excavation (i.e., Note A). In areas where the existing embankments are to be widened, the organics should be removed below the toe of the widened embankment in accordance with OPSD 203.020 (Embankments over Swamp, Existing Slope Excavated to 1H:1V). If space is not sufficient for the proposed slopes (as determined by the grading), then temporary roadway protection may be required as per OPSS 539 (Temporary Protection Systems) using Performance Level 2.

All excavations must be carried out in accordance with Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (as amended). In addition, provisions for traffic control measures should be included in the Contract Documents to maintain the safe operation of Highway 17, St. Pothier Road and any associated side roads or detours that are in close proximity to the excavation operations.

### 6.6.2 Groundwater and Surface Water Control

Excavation within the plan limits of the proposed works will be required to remove organic deposits prior to embankment fill placement, which will extend below the water table. Groundwater flow into the excavations will occur due to the presence of relatively permeable deposits and relatively high groundwater levels observed in the low-lying high fill/swamp crossing areas. Unwatering is not required for the excavation and backfilling in the high fill/swamp crossing areas, however, surface water should be directed away from the excavations at all times.

### 6.6.3 Backfilling

In general, it is recommended that rock fill be used for replacement of the sub-excavated materials. However, in areas where wick drains are required to mitigate stability and/or post-construction settlements, it is recommended that OPSS.PROV 1010 (Aggregates) Granular 'B' Type II be used for the replacement of the sub-excavated materials. Where sub-excavation of organic deposits is being carried out as a foundation mitigation option, it will not likely be possible to place rock fill or granular fill in accordance with OPSS.PROV 206 (Grading), as discussed in Section 6.6.5. For placement below the water table, rock fill and granular fill will likely have to be end dumped as the excavation advances.

### 6.6.4 Embankment Fill Placement

Placement of rock fill and granular fill above the water table for construction of new embankments should be placed and compacted in accordance with OPSS 501 (Compacting) and with the requirements as outlined in OPSS.PROV 206 (Grading). The rock fill should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging should be minimized by blading, dozing and 'chinking' the rock to form a dense, compacted mass. Side slopes for rock fill embankments should be no steeper than 1.25H:1V.

Where a surcharge fill or EPS levelling pad is required, granular fill should be placed in regular lifts with loose thickness not exceeding 300 mm and compacted to at least 95 per cent of the standard Proctor maximum dry density. Side slopes for granular fill should be no steeper than 2H:1V.

Where a large thickness of EPS is required in the embankment and a partial preload is recommended, consideration should be given to constructing the preload embankment of OPSS.PROV 1010 (Aggregates) Granular 'B' Type II at side slopes of 2H:1V.

Where the existing embankments are to be widened, the new fill should be "keyed-in" or benched into the existing embankment fills, in accordance with OPSD 208.010 (Benching of Earth Slopes).

The EPS fill should be installed in accordance with the manufacturer's requirements. It is recommended that a levelling pad comprised of a minimum 300 mm thick layer of OPSS.PROV 1010 (Aggregates) Granular 'A' be placed prior to the installation of the EPS. The EPS should be encapsulated by a 10 mil thick polyethylene sheet, and a minimum 125 mm thick reinforced concrete pad (designed by others) should be constructed on top of the EPS, followed by the placement of a protective cover/pavement structure over the EPS (for a minimum thickness of 1 m including the concrete pad, compacted granular materials and asphalt). The EPS on the side slopes of the embankments should be covered with a 2 m thick layer of conventional soil. The EPS should be placed to avoid conflict with guide rail installation, if any. Specifications to supply and install the EPS should be incorporated into an NSSP in the Contract (an example is included in Appendix D).

## 6.7 Slope Flattening

We understand that consideration is being given to flattening the proposed embankment rock fill slopes using surplus excavated materials, which is typically considered for all embankments under 4.5 m high as per OPSD 203.010 (Embankments Over Swamp) and OPSD 203.020 (Embankments over Swamp, Existing Slope Excavated to 1H:1V). However, depending on the type of material used, and the timing of placement of the surplus material, slope flattening may adversely affect the long-term performance of the roadway by inducing



further post-construction settlement. Considerations with respect to settlement and stability are discussed below. It is assumed that the rock fill embankment side slopes will be constructed at an inclination of 1.25H:1V and that the flattened side slopes will be constructed at 3H:1V or flatter. It is also understood that the material used for the slope flattening will likely consist of the excavated organic material or other excess earth material, excavated elsewhere or locally.

#### **6.7.1.1 Stability**

In general, global stability is enhanced when side slopes are flattened, hence the FoS of the flattened embankment slopes would be greater than the FoS of the existing embankment slopes.

#### **6.7.1.2 Settlement**

Post-construction settlement of the embankments will occur as a result of placement of the excess material in the slope flattening areas of the embankments. Therefore, the timing of placement of the additional/excess material load should be considered in determining whether slope flattening should be implemented. Three scenarios are presented below for different stages of placement of the additional slope flattening material as well as the corresponding settlement implications.

- 1) Concurrently with construction of the embankment (in stages where required). This construction method would produce the least amount of post-construction settlement of the roadway embankment.
- 2) After construction of the preload embankment and prior to placement of the final surcharge and/or prior to the full preload/surcharge period. Any settlement induced prior to construction of the final roadway could be accommodated by grading operations.
- 3) After the preload/surcharge period is complete. This construction method imposes additional loads from the slope flattening material, which will cause immediate and long-term settlement beneath both the embankment side slopes and the roadway and is the least preferred construction method. The magnitude of the settlement could be significant, depending on the embankment geometry and subsoil conditions in the area.

### **6.8 Embankment Platform Widening**

In accordance with the requirements of MTO Northern Region Engineering Directive NRE 98-200, Northern Region Embankment Design Guidelines, the construction of the embankments should include an allowance for platform widening (in 0.5 m increments) to accommodate settlements during construction, as well as post-construction settlements, so that the minimum standard shoulder widths are maintained if future grade raises on the embankments are required. According to NRE 98-200, the need for future raises in road grade could occur due to settlement/compression of the embankment fill, settlement of the foundation soils and to accommodate future pavement overlays up to 200 mm thick. We understand that this directive applies to all rock fill embankments, as well as for granular fill embankments, where widening restrictions are present (such as the presence of a sensitive body of water or due to space/property issues). It is further understood that the minimum required platform widening on major highways (i.e., including Highway 17) over swamp crossings is 2 m per side, unless the preferred mitigation option eliminates uncertainty regarding embankment





settlement/performance (i.e., full sub-excavation to bedrock and backfilling with granular material). For non-major highways and roadways (i.e., St. Pothier Road) over swamp crossings, the minimum required platform widening is 1 m per side.

The minimum required embankment platform widening (per embankment side) is calculated based on the estimated consolidation settlement of the foundation soils (including creep) and the settlement/compression of the embankment fill plus an additional 200 mm for the future pavement overlay, multiplied by the horizontal component of the side slope of the pavement structure (4H:1V), but cannot be less than the minimum platform widening requirements as described above.

For the proposed embankments in these swamp crossing/high fill areas, the minimum platform widening values are summarized in Table 5. The initial platform widening is required to account for settlement during and post construction. The final platform widening is required to account for post-construction settlement and future overlay.

## **7.0 CLOSURE**

This report was prepared by Mr. Evan Childerhose, P.Eng., and the technical aspects were reviewed by Ms. Sarah E. M. Poot, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M.A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal of Golder, conducted an independent quality control review of the report.

## Report Signature Page

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**FOUNDATION REPORT – FOUR-LANING HIGHWAY EXTENSION  
HIGH FILL EMBANKMENTS OVER SWAMPS GWP 156-98-00**

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**ASTM International**

|            |   |
|------------|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
| ASTM D1587 | Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes          |
| ASTM D2573 | Standard Test Method for Field Vane Shear Test in Cohesive Soil                             |

**Commercial Software:**

GeoStudio (Version 7.19) by Geo-Slope International Ltd.

Settle<sup>3D</sup> (Version 2.0) by Rocscience Inc.

**Ministry of Transportation Ontario:**

Embankment Settlement Criteria for Design. March 2010.

MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates. September 2010.

Northern Region Engineering Directive NRE 98-200. Northern Region Embankment Design Guidelines. October 1998.

Northeastern Region Geotechnical Section Memorandum. "Use of Mid-Slope Berms for Rockfill Embankments, Northeastern Region" dated February 8, 2005.

**Ontario Occupational Health and Safety Act:**

Ontario Regulation 213/91 Construction Projects (as amended)

**Ontario Provincial Standard Drawings:**

|              |  |
|--------------|--|
| OPSD 203.010 | Embankments Over Swamp – New Construction                  |
| OPSD 203.020 | Embankments Over Swamp – Existing Slope Excavated to 1H:1V |
| OPSD 208.010 | Benching of Earth Slopes                                   |

**Ontario Provincial Standard Specifications:**

|                 |  |
|-----------------|--|
| OPSS.PROV 209   | Construction Specification for Embankments Over Swamps and Compressible Soils                |
| OPSS.PROV 501   | Construction Specification for Compacting  |
| OPSS.PROV 539   | Construction Specification for Temporary Protection Systems                                  |
| OPSS PROV. 206  | Construction Specification for Grading   |
| OPSS PROV. 1010 | Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material |

**Ontario Water Resources Act:**

Ontario Regulation 903/90 Wells (as amended)





## LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

### I. GENERAL

|             |                                       |
|-------------|---------------------------------------|
| $\pi$       | 3.1416                                |
| $\ln x$     | natural logarithm of x                |
| $\log_{10}$ | x or log x, logarithm of x to base 10 |
| g           | acceleration due to gravity           |
| t           | time                                  |
| FoS         | factor of safety                      |

### II. STRESS AND STRAIN

|                                |  |
|--------------------------------|--|
| $\gamma$                       | shear strain   |
| $\Delta$                       | change in, e.g. in stress: $\Delta \sigma$                                 |
| $\epsilon$                     | linear strain  |
| $\epsilon_v$                   | volumetric strain  |
| $\eta$                         | coefficient of viscosity   |
| $\nu$                          | Poisson's ratio  |
| $\sigma$                       | total stress   |
| $\sigma'$                      | effective stress ( $\sigma' = \sigma - u$ )                                |
| $\sigma'_{vo}$                 | initial effective overburden stress  |
| $\sigma_1, \sigma_2, \sigma_3$ | principal stress (major, intermediate, minor)                              |
| $\sigma_{oct}$                 | mean stress or octahedral stress<br>$= (\sigma_1 + \sigma_2 + \sigma_3)/3$ |
| $\tau$                         | shear stress   |
| u                              | porewater pressure   |
| E                              | modulus of deformation   |
| G                              | shear modulus of deformation   |
| K                              | bulk modulus of compressibility  |

### III. SOIL PROPERTIES

#### (a) Index Properties

|                    |  |
|--------------------|--|
| $\rho(\gamma)$     | bulk density (bulk unit weight)*   |
| $\rho_d(\gamma_d)$ | dry density (dry unit weight)  |
| $\rho_w(\gamma_w)$ | density (unit weight) of water   |
| $\rho_s(\gamma_s)$ | density (unit weight) of solid particles   |
| $\gamma'$          | unit weight of submerged soil<br>( $\gamma' = \gamma - \gamma_w$ )                                   |
| $D_R$              | relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ ) |
| e                  | void ratio   |
| n                  | porosity   |
| S                  | degree of saturation   |

#### (a) Index Properties (continued)

|             |  |
|-------------|--|
| w           | water content  |
| $w_l$ or LL | liquid limit   |
| $w_p$ or PL | plastic limit  |
| $I_p$ or PI | plasticity index $= (w_l - w_p)$   |
| $w_s$       | shrinkage limit  |
| $I_L$       | liquidity index $= (w - w_p) / I_p$  |
| $I_C$       | consistency index $= (w_l - w) / I_p$  |
| $e_{max}$   | void ratio in loosest state  |
| $e_{min}$   | void ratio in densest state  |
| $I_D$       | density index $= (e_{max} - e) / (e_{max} - e_{min})$<br>(formerly relative density) |

#### (b) Hydraulic Properties

|   |   |
|---|---|
| h | hydraulic head or potential                             |
| q | rate of flow  |
| v | velocity of flow  |
| i | hydraulic gradient                                      |
| k | hydraulic conductivity<br>(coefficient of permeability) |
| j | seepage force per unit volume                           |

#### (c) Consolidation (one-dimensional)

|             |   |
|-------------|---|
| $C_c$       | compression index<br>(normally consolidated range)    |
| $C_r$       | recompression index<br>(over-consolidated range)      |
| $C_s$       | swelling index  |
| $C_\alpha$  | secondary compression index                           |
| $m_v$       | coefficient of volume change                          |
| $c_v$       | coefficient of consolidation (vertical direction)     |
| $c_h$       | coefficient of consolidation (horizontal direction)   |
| $T_v$       | time factor (vertical direction)                      |
| U           | degree of consolidation                               |
| $\sigma'_p$ | pre-consolidation stress                              |
| OCR         | over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$ |

#### (d) Shear Strength

|                  |  |
|------------------|--|
| $\tau_p, \tau_r$ | peak and residual shear strength                         |
| $\phi'$          | effective angle of internal friction                     |
| $\delta$         | angle of interface friction                              |
| $\mu$            | coefficient of friction $= \tan \delta$                  |
| $c'$             | effective cohesion                                       |
| $c_u, s_u$       | undrained shear strength ( $\phi = 0$ analysis)          |
| p                | mean total stress $(\sigma_1 + \sigma_3)/2$              |
| $p'$             | mean effective stress $(\sigma'_1 + \sigma'_3)/2$        |
| q                | $(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$ |
| $q_u$            | compressive strength $(\sigma_1 - \sigma_3)$             |
| $S_t$            | sensitivity  |

\* Density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1  
2

$\tau = c' + \sigma' \tan \phi'$   
shear strength = (compressive strength)/2

## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

|    |                     |
|----|---------------------|
| AS | Auger sample        |
| BS | Block sample        |
| CS | Chunk sample        |
| DS | Denison type sample |
| FS | Foil sample         |
| RC | Rock core           |
| SC | Soil core           |
| SS | Split-spoon         |
| ST | Slotted tube        |
| TO | Thin-walled, open   |
| TP | Thin-walled, piston |
| WS | Wash sample         |

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), $N_d$ :

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

#### Dynamic Cone Penetration Resistance; $N_d$ :

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

|            |   |
|------------|---|
| <b>PH:</b> | Sampler advanced by hydraulic pressure        |
| <b>PM:</b> | Sampler advanced by manual pressure           |
| <b>WH:</b> | Sampler advanced by static weight of hammer   |
| <b>WR:</b> | Sampler advanced by weight of sampler and rod |

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### III. SOIL DESCRIPTION

#### (a) Non-Cohesive (Cohesionless) Soils

| Density Index    | N                        |
|------------------|--------------------------|
| Relative Density | Blows/300 mm or Blows/ft |
| Very loose       | 0 to 4                   |
| Loose            | 4 to 10                  |
| Compact          | 10 to 30                 |
| Dense            | 30 to 50                 |
| Very dense       | over 50                  |

#### (b) Cohesive Soils Consistency

|            | $c_u, s_u$ |                |
|------------|------------|----------------|
|            | kPa        | psf            |
| Very soft  | 0 to 12    | 0 to 250       |
| Soft       | 12 to 25   | 250 to 500     |
| Firm       | 25 to 50   | 500 to 1,000   |
| Stiff      | 50 to 100  | 1,000 to 2,000 |
| Very stiff | 100 to 200 | 2,000 to 4,000 |
| Hard       | over 200   | over 4,000     |

### IV. SOIL TESTS

|                 |   |
|-----------------|---|
| w               | water content   |
| $w_p$           | plastic limit   |
| $w_l$           | liquid limit  |
| C               | consolidation (oedometer) test  |
| CHEM            | chemical analysis (refer to text)   |
| CID             | consolidated isotropically drained triaxial test <sup>1</sup>                                       |
| CIU             | consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup> |
| $D_R$           | relative density (specific gravity, $G_s$ )   |
| DS              | direct shear test   |
| M               | sieve analysis for particle size  |
| MH              | combined sieve and hydrometer (H) analysis  |
| MPC             | Modified Proctor compaction test  |
| SPC             | Standard Proctor compaction test  |
| OC              | organic content test  |
| SO <sub>4</sub> | concentration of water-soluble sulphates  |
| UC              | unconfined compression test   |
| UU              | unconsolidated undrained triaxial test  |
| V               | field vane (LV-laboratory vane test)  |
| $\gamma$        | unit weight   |

**Note: 1** Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

### V. MINOR SOIL CONSTITUENTS

| Per cent by Weight | Modifier   | Example   |
|--------------------|--|---|
| 0 to 5             | Trace  | Trace sand  |
| 5 to 12            | Trace to Some (or Little)                            | Trace to some sand  |
| 12 to 20           | Some   | Some sand   |
| 20 to 30           | (ey) or (y)  | Sandy   |
| over 30            | And (non-cohesive (cohesionless)) or With (cohesive) | Sand and Gravel<br>Silty Clay with sand / Clayey Silt with sand |

**FOUNDATION REPORT – FOUR-LANING HIGHWAY EXTENSION  
HIGH FILL EMBANKMENTS OVER SWAMPS GWP 156-98-00**

**Table 1: Summary of High Fill Areas**

| Foundation Investigation Area                                 | Foundation Investigation High Fill Area Designation | Maximum Proposed Embankment Height <sup>1</sup> | Boreholes/DCPTs   | Appendix |
|---|---|---|---|----------|
| Highway 17 WBL<br>STA 12+220 to 12+570<br>Township of Louise  | H1  | 5.7 m   | 16 Boreholes (H1-1 to H1-9, H1-11 to H1-15, C1-1 and C1-2) and 6 DCPTs (H1-DC1 to H1-DC6) | A        |
| Highway 17 EBL<br>STA 12+220 to 12+570<br>Township of Louise  | H1  | 5.7 m   | 17 Boreholes (H1-16 to H1-30, C1-3, C1-4) and 6 DCPTs (H1-DC7 to H1-DC12)                 | A        |
| Highway 17 WBL<br>STA 13+140 to 13+390<br>Township of Louise  | H2  | 5.0 m   | 21 Boreholes (H2-1 to H2-21) and 10 DCPTs (H2-DC1 to H2-DC10)                             | B        |
| Highway 17 EBL<br>STA 13+140 to 13+390<br>Township of Louise  | H2  | 5.0 m   | 22 Boreholes (H2-22 to H2-43) and 11 DCPTs (H2-DC11 to H2-DC21)                           | B        |
| St. Pothier Road<br>STA 9+400 to 9+600<br>Township of Louise  | H2  | 4.4 m   | 17 Boreholes (H2-44 to H2-60) and 8 DCPTs (H2-DC22 to H2-DC29)                            | B        |
| Highway 17 WBL<br>STA 13+900 to 14+200<br>Township of Denison | H3  | 3.8 m   | 13 Boreholes (H3-1 to H3-13) and 6 DCPTs (H3-DC1 to H3-DC6)                               | C        |
| Highway 17 EBL<br>STA 13+900 to 14+200<br>Township of Denison | H3  | 3.8 m   | 13 Boreholes (H3-14 to H3-26) and 6 DCPTs (H3-DC7 to H3-DC12)                             | C        |

Note: 1. Based on centreline of highway and existing ground surface profiles, dated February 2013. Prepared by: EC Checked by: SEMP

**FOUNDATION REPORT – FOUR-LANING HIGHWAY EXTENSION  
HIGH FILL EMBANKMENTS OVER SWAMPS GWP 156-98-00**

**Table 2: Summary of Consolidation Test Parameters**

| Foundation Investigation Area  | Borehole/<br>Sample No. | Elevation<br>(m) | $\sigma_{vo}'$<br>(kPa) | $\sigma_p'$<br>(kPa) | $\sigma_p' - \sigma_{vo}'$<br>(kPa) | OCR | $e_o$ | $C_c$ | $C_r$ | $c_v^*$<br>(cm <sup>2</sup> /s) | Appendix |
|--------------------------------|-------------------------|------------------|-------------------------|----------------------|-------------------------------------|-----|-------|-------|-------|---------------------------------|----------|
| Highway 17 EBL<br>High Fill H1 | H1-19/Sa 6              | 236.4            | 31                      | 256                  | 225                                 | 8.3 | 1.86  | 1.04  | 0.02  | $2.1 \times 10^{-3}$            | A        |
| Highway 17 EBL<br>High Fill H1 | H1-25/Sa 8              | 234.6            | 56                      | 135                  | 79                                  | 2.4 | 1.52  | 0.66  | 0.02  | $3.8 \times 10^{-3}$            |          |
| Highway 17 EBL<br>High Fill H2 | H2-26/Sa 7              | 233.3            | 39                      | 128                  | 89                                  | 3.2 | 1.67  | 0.48  | 0.01  | $2.0 \times 10^{-3}$            | B        |
| Highway 17 EBL<br>High Fill H2 | H2-36/Sa 8A             | 232.7            | 47                      | 132                  | 85                                  | 2.8 | 2.13  | 0.48  | 0.08  | $3.8 \times 10^{-4}$            |          |
| Highway 17 WBL<br>High Fill H3 | H3-12/Sa 7              | 231.4            | 97                      | 129                  | 32                                  | 1.3 | 1.13  | 0.41  | 0.02  | $1.3 \times 10^{-3}$            | C        |
| Highway 17 EBL<br>High Fill H3 | H3-24/Sa 9              | 231.3            | 149                     | 149                  | 0                                   | 1.0 | 1.14  | 0.36  | 0.05  | $1.2 \times 10^{-3}$            |          |

Note: For the normally consolidated stress range.

where:  $\sigma_{vo}'$  is the in situ vertical effective overburden stress in kPa  
 $\sigma_p'$  is the preconsolidation stress in kPa  
OCR is the over consolidation ratio  
 $e_o$  is the initial void ratio  
 $C_c$  is the compression index  
 $C_r$  is the recompression index  
 $c_v$  is the coefficient of consolidation in cm<sup>2</sup>/s

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