



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/ Sample No.	Sample Depth / Elevation	$\sigma_{vo}'$ (kPa)	$\sigma_p'$ (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	$e_o$	$C_c$	$C_r$	$c_v^*$ (cm <sup>2</sup> /s)
H3-12/Sa 7	11.0 m / 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.

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Borehole/ Sample No.	Sample Depth / Elevation	$\sigma_{vo}'$ (kPa)	$\sigma_p'$ (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	$e_o$	$C_c$	$C_r$	$c_v^*$ (cm <sup>2</sup> /s)
H3-24/Sa 9	12.5 m / 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.



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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 Kosc 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 OPSP 203.010 (Embankments Over Swamps) prior to construction of the new embankment or OPSP 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;