

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/m³ and 16.6 kN/m³ 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/ Sample No.	Sample Depth / Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	e_o	C_c	C_r	c_v^* (cm ² /s)
H2-26/Sa 7	6.4 m / 233.3 m	39	128	89	3.2	1.67	0.48	0.01	2.0×10^{-3}
H2-36/Sa 8A	7.1 m / 232.7 m	47	132	85	2.8	2.13	0.48	0.08	3.8×10^{-4}

*For the normally consolidated stress range.

where: σ_{vo}' is the effective overburden stress in kPa
 σ_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 cm²/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/m³ and a specific gravity of 2.78 was measured on the consolidation test sample. The

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	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	e_o	C_c	C_r	c_v^* (cm ² /s)
H3-12/Sa 7	11.0 m / 231.4 m	97	129	32	1.3	1.13	0.41	0.02	1.3×10^{-3}

*For the normally consolidated stress range.

where: σ_{vo}' is the effective overburden stress in kPa
 σ_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 cm²/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/m³ 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	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	e_o	C_c	C_r	c_v^* (cm ² /s)
H3-24/Sa 9	12.5 m / 231.3 m	149	149	0	1.0	1.14	0.36	0.05	1.2×10^{-3}

*For the normally consolidated stress range.

where: σ_{vo}' is the effective overburden stress in kPa
 σ_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 cm²/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

GOLDER ASSOCIATES LTD.



Adam Core, E.I.T.
Geotechnical Engineering Intern



Sarah E. M. Poot, P. Eng.
Senior Geotechnical Engineer, Associate



Jorge M. A. Costa, P. Eng.
Designated MTO Contact, Principal

AC/SEMP/JMAC/kp

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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 kN/m^3 and an effective friction angle of 40° . The granular fill (assume Granular B Type II) modelled in the analyses is assumed to have a unit weight of 20 kN/m^3 (uncompacted below the water table) and 21 kN/m^3 (above the water table) and an effective friction angle of 35° . 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)
σ'_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)
μ	=	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*^{3D} (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 (cm^2/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