

**FOUNDATION DESIGN REPORT
PROPOSED HIGHWAY 17
TRUNK ACCESS ROAD BRIDGES
OVER THE BLACK CREEK
SAULT STE. MARIE, ONTARIO
G.W.P. 406-01-00
GEOCRES NO. 41K-66**

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5. DISCUSSION AND RECOMMENDATIONS

The proposed Trunk Access Road (Link Road), to be constructed from Trunk Road north easterly for 1.6 km to Highway 17 (New), crosses Black Creek about midway along the proposed alignment. At this crossing, the north bank of the creek is only about 0.5 to 1 m above the normal creek water level of El. 181.5 ±m and then rises gradually to about El. 184 m within a horizontal distance of about 30 m, while on the south side, the bank of the creek first rises sharply to about El. 186 ±m and then more gently to about El. 189 m. In 2003, an investigation was carried out by our firm at the present creek location for 20 m long single span, twin bridges which would carry the EBL and WBL of the proposed highway. The boreholes showed below a veneer of topsoil, the presence of silty fine sand with sandy silt zones to about El. 175 m, which is underlain by an extensive clay deposit to about El. 140 ±m. Interbeds/lenses of silty clay, clayey silt and silt as well as occasional layers/lenses of silty fine sand to sandy silt were also found within the clay deposit in the upper zones (between about El. 170-159 m). The silty fine sand to sandy silt deposit was found to be generally very loose to loose. The underlying clay deposit is generally of firm to stiff consistency in the upper zones becoming stiff to very stiff with increasing depth. The clay extends to about El. 140 ±m (i.e. more than 40 m below the prevailing ground surface) and is underlain by a lower silty fine sand with sandy silt zones (see inferred subsurface profiles in Appendix B). This lower silty fine sand with sandy silt zones is considered to be compact to dense and was found to be under excess hydrostatic pressure.

Groundwater table at the time of our 2003 investigation was found at depths ranging between 0.6 and 4.5 m below the ground surface or at El. 182.8 to 180.6 m, but would be subject to seasonal changes and in response to water level in the creek. Subsequently, in 2004, it was noted one of the piezometers showed artesian conditions emanating from one of the silty sand layers in the clay deposit at about 12 m below the ground surface.

Subsequent to our investigation, the design team decided to relocate the creek at the proposed crossing by about 70 m to the north, which would entail the re-channeling the creek and filling of the existing creek bed.

Our firm carried out an additional foundation investigation in January 2005 at the proposed new bridge locations. As agreed by MTO, this investigation consisted of two deep boreholes (Boreholes 101 and 102) and one shallow borehole (Borehole 103), plus two dynamic cone

penetration tests (DCPT Boreholes 101A and 102A), adjacent to the deep boreholes. In addition, Borehole F of the previous investigation is located close to the present site. A subsurface profile based on these four boreholes is given in Drawing No. 1 (see Foundation Investigation Report). Briefly, these boreholes showed in general the presence of a surficial clay layer to a depth of about 1.2 m, beneath a veneer of topsoil. The surficial clay layer is underlain by an approximately 2.0 m thick organic silt layer followed by a very loose to loose sandy silt to silty fine sand deposit to a depth between 7.5 and 10.0 m below the ground surface or to about an average Elevation of 175 m. Below this, an extensive deposit of clay of soft to very stiff consistency extends to a depth of about 41 m/EI. 143 m. The upper zones of this clay deposit are interbedded sandy silt to silty fine sand layers to 16 m depth/EI. 168 m. The clay is underlain a compact sandy silt to silty fine sand deposit under artesian pressure. Borehole 101 was terminated in this deposit at a depth of 49 m/EI. 135 m. Borehole 102, which was extended deeper, contacted a lower clay deposit of stiff consistency at 54 m below the ground surface or at EI. 130.5 m.

We understand that the new twin bridges, which will carry the EBL and WBL of the connecting link, will be single span structures and integral type abutments are preferred. At the time of the preparation of this report, the conceptual design incorporated 27 m long span and approximately 13 m wide twin structures (see Figures G-1 and G-2 in Appendix G). The presently proposed finished grades are about Elevation 185.3 m and 185.7 m for the north and south end of the bridges, respectively and as such minor regrading (i.e. fills of about 1.0 to 1.2 m) is required on the north side of the proposed bridge. As shown in Figure H-1 in Appendix H, on the south side, the grade will be raised by about 1.3 m at the south abutment location but the existing ground surface elevation drops rapidly starting about 4 m south of the south abutment location towards the flood plain of the creek and the height of the fill reaches to about 4 m about 25 m towards the south, increasing to about 5.0 to 5.5 m height about 20 m further south, within the existing creek bed location. Further south from the creek bed, the existing grade rises sharply and the fill changes into a cut about 80 m south of the proposed south abutment locations.

We understand that the highway will be opened to traffic in 2006 and therefore, there are limited opportunities for the duration of preloading, surcharging, etc.

5.1 FOUNDATION CONDITIONS

5.1.1 GENERAL

The organic silt and the underlying loose to very loose upper sandy silt to silty fine sand are considered unsuitable for the support of normal shallow spread footing foundations, including the use of footings on engineered fill (i.e. on compacted Granular 'A' pad). The relative density (i.e. compactness condition) of the sandy silt to silty fine sand can be improved by surcharging and/or by means of various in-situ densification methods but such

operations are considered to be impractical immediately adjacent to the creek, particularly in view of environmental aspects and duration of the operations. In addition, the organic silt will need to be replaced. As well, excessive settlements can be expected due to the consolidation of the underlying clay deposit. The bridges will, therefore, need to be supported on deep foundations.

The use of drilled and cast-in-place concrete (caisson) foundations to support the structures is considered impractical due to the presence of water-bearing sandy silt to fine sand deposits and the lack of a well-defined bearing stratum to support the caissons within the clay. Auger press piles can be extended in the cohesionless (sand) soils below the groundwater table, but these offer little resistance to lateral loads and will not be economical. They are, therefore, not recommended based on reliability and cost.

Expanded base (Franki-type) concrete piles and driven concrete piles are not considered to represent a practical, cost-effective and reliable solution for this project and as such they are not recommended.

It is our opinion that for this project the use of driven steel piles represents the best choice. Low displacement steel H-piles are better suited with the prevailing subsurface conditions, in comparison with steel tube piles, because H-piles will more easily penetrate through the thick clay deposits in excess of 40 m depth, as well since integral abutments are preferred.

The following table presents the merits and disadvantages of various foundation alternatives.

Table 5.1.1.1
Summary of Foundation Alternatives

Foundation Type	Comments	Recommendations
<ul style="list-style-type: none">Normal spread footingsSpread footings on compacted Granular 'A' pad	Excessive settlements due to loose to very loose sandy silt to silty sand to depths of about 7 to 11 m and the underlying firm to very stiff clay. The organic soils will need to be replaced.	Not recommended based on reliability and practicability.
<ul style="list-style-type: none">Footings on compacted Granular 'A' after the removal and replacement of organic silts and surcharging	Surcharging is impractical due to time restraints.	Impractical due to time restraints. Not recommended based on reliability and practicality.
<ul style="list-style-type: none">Footings on compacted Granular 'A' pad after removal of organic silt deposit and in-situ densification of foundation soils and surcharging	Impractical due to time restraints and adverse environmental effects during construction. As well, the underlying clays can be expected to undergo excessive settlements unless sufficiently surcharged.	Not recommended based on practicability of construction, economics, time restraints and reliability.

Foundation Type	Comments	Recommendations
○ Footings on Expanded Base (Franki-type) concrete piles after the removal of organic silt and surcharging	Impractical due to time restraints and adverse environmental effects during construction. As well, the underlying clays can be expected to undergo excessive settlements.	Not recommended based on practicality of construction, economics, time restraints and reliability.
○ Drilled caissons	Impractical due to water-bearing sands and a lack of well-defined bearing zone in the clay.	Not recommended based on cost and reliability.
○ Auger press concrete piles	Do not provide lateral support and are costly.	Not recommended based on cost and reliability.
○ Driven concrete piles	Considered uneconomical.	Not recommended based on cost.
○ Timber piles	Short piles will not provide adequate axial resistance.	Not recommended based on reliability.
○ Steel H-piles	Being low displacement piles, represent the best option to reach a suitable end-bearing and adhesion/friction depths at about 50 m depths, as well as being necessary for integral abutments.	Considered best choice based on reliability and suitability.
○ Steel tube piles	Less reliable than steel H-piles; are not suitable for integral abutments.	Can be considered an alternative to steel H-piles.

From the foregoing discussion and table presented, it is evident that driven steel piles represent the best alternative for the support of the proposed twin bridges.

It should, however, be pointed out that the organic silt, the very loose to loose sandy silt to silty fine sands and the firm to very stiff clay deposits encountered can be expected to undergo considerable settlements due to the stresses imposed by the approach and abutment fills to be placed, especially on the south side where the existing grades are relatively low and further drop towards the flood plain of the creek. These aspects and their influence on the staging of the construction of the foundations (e.g. piles will need to be driven after the surcharging period is completed) will be discussed in further sections of the report.

5.1.2 STEEL H-PILES

The boreholes show that with the prevailing subsurface conditions the use of a low displacement pile, such as a steel H-pile with a heavy section (e.g. HP 310 x 110), would be better suited than other pile types (e.g. steel tube piles, steel H-piles with lighter sections or precast concrete piles).

The following table summarizes the approximate average tip elevations that may be utilized for design purposes.

Table 5.1.2.1
Estimated Pile Tip Elevations for Steel H-Piles

Support Location	Reference Borehole	Estimated Pile Tip Elevation	Estimated Approximate Pile Length Below Existing Ground Surface	Soil Deposit
South Abutments	101	136.0 m	48.2 m*	sandy silt to silty fine sand
North Abutments	102	135.0 m	49.5 m*	sandy silt to silty fine sand

*Actual effective steel length of the piles below the flex zone will likely be about 6 m less than the figures given here.

In view of the fact that since the pile capacities will be based on friction and adhesion in addition to end-bearing, as well as observed artesian conditions, reinforcing flanges should not be used.

The following axial resistances are estimated for HP 310 x 110 steel piles driven to tip elevations documented in Table 5.1.2.1.

Factored Axial Resistance at U.L.S. = 1100 kN/pile
Axial Resistance at S.L.S. = 750 kN/pile

Approximately two-thirds of these resistances are attributed to adhesion and one-third to end-resistance.

The piles should be driven using a suitably heavy hammer capable of delivering a rated energy of at least 50 kilojoules/blow, but not more than 70 kilojoules/blow. The driving of the piles in the field should be controlled by a recognized pile driving formula, such as the Hiley Formula. Normally, in accordance with MTO practice, the estimated ultimate resistance of the piles by the Hiley Formula can be calculated by multiplying the recommended axial resistance at U.L.S. by a factor of 2 (i.e., 1100 x 2), giving an ultimate geotechnical resistance of 2200 kN.

In accordance with the above criterion, we recommend that the piles be driven to about 2 m above the quoted design elevations (i.e. El. 138 m south abutments and El. 137 m north abutments), and driving should then be monitored and controlled by employing the Hiley Dynamic Pile Driving Formula in accordance with MTO Standard SS103-11, using an ultimate geotechnical resistance of 2200 kN per pile, but not below El. 131.5 m without the approval of the Engineer.

If the piles encounter refusal before sufficiently penetrating into the compact sandy silt to silty sand deposit underlying the clay, then pile capacities will need to be revisited, as they are based on a friction adhesion and end-bearing factors, or measures of extending the piles to the required depths will need to be sought. It is also possible that the piles may drive some distance below the estimated pile tip elevations. All pile driving should be carried out in accordance with SP903S01. Re-striking should be done as per SP903S01. As well, it may be necessary to stagger the driving of the piles, if heaving is observed. As mentioned in Section 5.4 of this report, the piles will need to be driven after surcharging. The use of light-weight (e.g. HP 310 x 79) piles is not recommended due to the energy required to extend the piles to relatively deep tip elevations (i.e. piles may be damaged). We recommend in contract documents the possibility of pile load test(s) be allowed for.

For frost protection, all pile caps should have a permanent earth cover of at least 1.8 m, or equivalent artificial insulation.

In cohesionless soils the coefficient of horizontal subgrade reaction can be estimated from:

$$k_s = n_h z / d$$

Where k_s = coefficient of horizontal subgrade reaction
 z = depth
 d = pile width
 n_h = coefficient related to soil density as given in Table 5.1.2.2

Also as presented in the same table are estimated values for angle of internal friction and bulk unit weights.

Where the soil is primarily cohesive, the undrained shear strength of the soil is given.

Table 5.1.2.2
Recommended Soil Parameters

Area Reference/ Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m ³)	Angle of Internal Friction (ϕ) Degrees	Recommended n_h Value (MN/m ³)	Recommended Undrained Shear Strength (kPa)	
EBL- North Abutment/101	184.0-183.1	silty clay	18.0	20	0.2	30	
	183.1-181.2	organic silt*	14.0			25	
	181.2-180.9	clay	16.0				
	180.9-176.4	sandy silt to silty fine sand	18.5	29	1.2	25	
	176.4-175.4	clay	16.0	29	1.3		
	175.4-172.7	sandy silt to silty fine sand	18.5		25		
	172.7-170.5	clay	16.0			29	1.3
	170.5-168.2	sandy silt to silty fine sand	18.5	60 120			
	168.2-160.0	clay	16.0				
	160.0-142.7	clay	16.5				
	142.7-135.0	sandy silt to silty fine sand	19.0		30	4.0	

Area Reference/ Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m ³)	Angle of Internal Friction (φ) Degrees	Recommended n _h Value (MN/m ³)	Recommended Undrained Shear Strength (kPa)
WBL-South Abutment/102	184.3-183.2	silty clay	18.0			30
	183.2-181.3	organic silt*	14.0	20	0.2	
	181.3-176.0	sandy silt to silty fine sand	18.5	29	1.2	
	176.0-173.0	clay	16.0			25
	173.0-169.5	clay	16.0			30
	169.5-168.5	sandy silt to silty fine sand	18.5	29	1.3	
	168.5-158.0	clay	16.0			80
	158.0-143.5	clay	16.5			120
	143.5-130.5	sandy silt to silty fine sand	18.5	30	4.0	
	130.5-129.2	clay	16.5			50

*If replaced with Granular 'B' type material, the following values can be used:

Bulk unit weight = 21 kN/m³, Angle of Internal Friction (φ) = 32 degrees, n_h value = 3.0 MN/m³

For preliminary estimating purposes, the recommended horizontal resistances for HP310x110 steel H-piles are as follows:

Factored Horizontal Resistance at U.L.S. = 120 kN/pile

Horizontal Resistance at S.L.S. = 50 kN/pile

If integral abutments are not constructed then the lateral resistance of the piles can be supplemented, if desired, by the horizontal components of battered piles. In this instance, we recommend that the batter be limited to no more than 5:1, as in practice greater batter is difficult to install, especially considering the length of the piles. The minimum spacing between piles should be in accordance with Clause 6.8.9.2 of the CAN/CSA-S6-00, Canadian Highway Bridge Design Code (CHBDC).

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven.

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flex zone. In essence, the current MTO standard for the flex zone consists of an annular space in between two concentric corrugated steel pipes (CSP's). One of the CSP's surrounds the H-pile (i.e. has a diameter of about 600 mm surrounding the pile, while the second CSP has a somewhat larger diameter; typically 800 mm for a 310 mm H-pile). The annular space in between the CSP's is the 3 m long flex zone. In accordance with current MTO practice, this space between the CSP's can be left void. After the pile is driven, the space between the H-pile and the inner CSP is filled with sand.

Alternatively, in accordance with MTO structural office requirements (Report SO-96-01), the flex zone can be provided by augering a 600 mm diameter hole 3000 mm deep and filling

with uniform sand. A special provision should be included in the contract specifying the gradation of the sand as follows:

Sieve Size	Percentage Passing
2 mm	100 %
600 µm	80-100 %
425 µm	40-80 %
250 µm	4-25 %
150 µm	0-6 %

5.1.3 STEEL TUBE PILES

Tube piles will provide lower resistances in comparison with H-piles as they will not drive as deep, but it is possible that the lower resistances may be somewhat compensated by the relatively shorter pile lengths and material costs. These piles may not drive into a well-defined competent layer across the site and, therefore, geotechnical resistances will be based on both friction and end-bearing. Steel tube piles have the advantage that they can be inspected after driving and prior to pouring the concrete for possible damage that may have incurred while driving. They should have sufficient wall thickness and base plate thickness to minimize potential damage caused by the expected hard driving conditions, due to depths to be driven. The end plates should not be wider than the base area of the piles (i.e. should not project beyond the circumference of the pile) so that adhesion/friction is not adversely affected. Tube piles will need to be filled with concrete after their installation and inspection for possible damage.

Steel tube piles of 300 mm nominal diameter (e.g. 324 mm x 12.5 mm) driven at least 2 m into the sandy silt to silty sand deposit underlying the clay can be expected to provide a Factored Axial Resistance at U.L.S. of 900 kN and an Axial Resistance at S.L.S. equal to 600 kN at about El.139.0 m.

The piles will need to be driven using a suitably heavy hammer capable of delivering a rated energy of at least 50 kilojoules/blow, but not more than 70 kilojoules/blow. The driving of the piles in the field should be controlled by a recognized pile driving formula, such as the Hiley Formula. The estimated ultimate resistance of the piles based on the Hiley Formula can be calculated by multiplying the recommended axial resistance at U.L.S. by a factor of 2.0. With this criterion, the estimated ultimate resistance required would be $900 \times 2.0 = 1800$ kN.

The piles should be driven to about 2 m above the design elevation and driving should then be monitored and controlled by employing Hiley Dynamic Pile Driving Formula in accordance with MTO Standard SS103-11. Steel tube piles are less likely to reach the required depths in comparison with steel H-piles and are therefore considered to be a less desirable choice.

The driving of the piles should be conducted in accordance with SP903S01. As was mentioned for steel H-piles re-striking and staggering should be allowed for, if necessary.

Pile lengths may be different than the estimated values and, therefore, this aspect will need to be considered in the contract documents and when ordering piles. As mentioned before, piles may not reach the required pile tip elevations and pile load test(s) may need to be conducted.

The minimum pile spacing should be in accordance with CHBDC and with due consideration of the pile lengths.

Suggested soil parameters for the calculation of the lateral resistance/deflection of the piles were given in the previous section of this report. If battered piles are required to sustain horizontal loads, then the batter should be limited to 5:1 in view of the lengths of the piles as was discussed before.

The piles will need to be driven after the surcharging period, as discussed in Section 5.4.

5.2 LATERAL EARTH PRESSURES

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free-draining granular materials in accordance with the Ontario Ministry of Transportation Standards and the requirements of OPSD 3501.00 and OPSD 3505.00.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B') and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with C.H.B.D.C.. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A' and Granular 'B' Type II

Angle of Internal Friction, $\phi = 35^\circ$ (unfactored)

Unit Weight = 22 kN/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.27$

$K_b = 0.35$

$K_o = 0.43$

$K^* = 0.45$

Compacted Granular 'B' Type I

Angle of Internal Friction, $\phi = 32^\circ$ (unfactored)

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

$$K_a = 0.31$$
$$K_o = 0.47$$

$$K_b = 0.41$$
$$K^* = 0.57$$

Rock Fill

Angle of Internal Friction, $\phi = 40^\circ$ (unfactored)

Unit Weight = 18 kN/m^3

Coefficient of Lateral Earth Pressure:

$$K_a = 0.27$$

$$K_b = 0.32$$

$$K_o = 0.36$$

$$K^* = 0.42$$

Where K_b is the 'intermediate' earth pressure coefficient for a partially restrained structure.

K^* is the earth pressure coefficient for a soil loading a fully-restrained structure, including compaction surcharge effects.

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest pressures should be used in accordance with C.H.B.D.C.. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9 of C.H.B.D.C..

For unrestrained wing walls (if any), the intermediate earth pressure coefficient K_b may be adopted. In the determination of degree of wall displacement or rotation to mobilize the fully active earth pressure state, Section C6.9 of the C.H.B.D.C. Commentary can be consulted. We understand, however, that the present design of the bridge structures does not incorporate any wing walls.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice.

If rock fill is used for backfill, special care is required to prevent damage to the retaining structures. In such a case, a cushion of Granular 'A' material or finely-graded rock fill (e.g. less than 200 mm normal diameter) should be placed between the structure and the rock fill. This cushion should be at least 0.45 m wide and if Granular 'A' is used, proper filtering should be provided to prevent the loss of finer particles from the Granular 'A' cushion into the coarse rock fill. As was mentioned, however, coarse materials (e.g. rock fill) should not be used in areas through which piles will be driven.

5.3 CUT CHANNEL

The relocated creek will entail a new cut channel between about Stations 10+824 and 10+851. The proposed channel profile is shown in Figure G-2 in Appendix G. The boreholes drilled at this location showed the presence of a 2.0 m thick organic silt layer which extends to a depth of about 3 m below the o.g.. This deposit was not encountered in the previously drilled boreholes to the south of the proposed channel location, except in Borehole F where the presence of organics was noted to 1.0 m depth. We recommend that these organic soils be removed and replaced with Granular 'B' type materials. Based on our slope stability analyses, this configuration is stable except for the 1½H:1V side slopes in the lower portion which may not be stable, when the water level in the creek shows a rapid drawdown condition, creating seepage forces. We understand that the meandering of the creek does not present a problem; rather it may be desirable from an environmental point of view. However, such meandering is undesirable during the construction and surcharge period before the bridges are built and therefore we recommended that these slopes be inspected bi-weekly during this period and rectified, if necessary.

5.4 APPROACH EMBANKMENTS

At present, it is expected that the grades at the proposed north embankment locations will be raised by about 1.2 m on the south side at the proposed abutment location the grade will be raised by 1.3 m. As shown in Figure H-1 in Appendix H, further south, the existing grade (o.g.) gradually falls towards the existing creek valley and then into the existing creek bed, some 50 m south of the south abutment location of the proposed bridges. At the existing creek bed location, the grade raise will be a maximum of about 5.6 m, with an average of about 5.2 m between Stations 10+766 and 10+780. Further south, o.g. rises rather rapidly and the proposed grade diminishes to zero at about Station 10+750, with cut being proposed to the south of this area.

Based on the borehole results, no foundation failures are anticipated for up to 6 m high embankments with normal (2H:1V) side slopes, assuming that all organic or otherwise unsuitable materials will be removed as per MTO standards prior to placing the embankment fills.

All organic and other unsuitable soils should be removed within an envelope area given by an imaginary slope not steeper than 1:1 from the toe of the proposed embankment. Based on the available borehole data, for preliminary estimating purposes, the average thickness of unsuitable soils to be stripped can be assumed to be 0.2 m. However, the thickness of organic or otherwise organic soils can be variable, especially near watercourses. We have no geotechnical/foundation information within the existing creek bed but for preliminary estimating purposes, allowance should be made to remove about 1 m of unsuitable sediments.

In addition, the presence of organic soils was noted in Borehole F, to a depth of 1.0 m. Organic soils were also encountered in Boreholes 101, 102 and 103 from about 1.2 m to about 3.0 m below the ground surface. This aspect is further discussed in this section of the report. After stripping, the exposed subgrade should be inspected, approved and properly compacted (i.e. proof rolled) from the surface, using a suitably heavy compactor. The existing site conditions (i.e. high watertable and fine-grained granular soils) could influence the choice of compaction equipment. Proof rolling and application of compaction within the first 0.5 to 1 m thick zone of the fill layer may not be feasible in the low-lying areas below the groundwater level.

Assuming properly compacted, acceptable inorganic earth fill materials are utilized, 2 horizontal in 1 vertical side slopes can be used for the construction of the approach fills. However, local flattening will be required depending on the geometry and especially where surcharge is required, as will be discussed later. Proper erosion control measures should be implemented by seed and cover (OPSS 572) or sodding (OPSS 571).

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill. We understand that Granular 'B' (Type I) is readily available and this material would be recommended especially below the groundwater level as well as within the upper zones of the embankment fill (i.e. within the frost zones). Oversize materials (having a nominal diameter in excess of 75 mm) should not be used in embankment fills through which piles would be driven. Fill used for construction of the embankments should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of OPSS 501 and OPSS 206. In general, the fills should be placed in lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density.

The settlement of embankment fills under their own weight, prepared as described above, should not exceed 30 mm for embankment heights of up to about 5.5 m. The time-rate of settlement will depend on the materials used. For example, granular soils will settle more rapidly than finer soils. These settlements will be substantially completed and can be ignored if a surcharge period of six months is effected.

In addition to settlement under self-weight, considerable foundation settlements can be expected under the weight of the approach fills to be placed. With the present design, on the north side the approach embankments will be approximately 1.2 m to 1.0 m high while on the south side the height of the embankment starts at about 1.3 m at the south abutment location and gradually increases to about 5.5 m at the existing creek bed location as shown on Figure H-1, Appendix H. The total settlements anticipated under the 1.0 to 5.2 m of embankment fill range from about 0.2 m to in excess of 1.0 m, as summarized in Table 5.4.1, depending on the height of the fill and nearest borehole results. These figures include settlements during the construction period (i.e. fill placement to full height). A significant aspect of these settlements is the rate of settlements. The silty fine sands encountered in

the previous investigation within the upper $8\pm$ m can be expected to settle relatively rapidly, while the sandy silt to silty fine sand deposits encountered in 100-series boreholes (i.e. recent boreholes) can be expected to settle somewhat more slowly, but can be expected to be completed within a preload period of several months. The underlying clay with silty fine sand and silt layers can be expected to settle more slowly. These extend to a maximum depth of about 16 m (except in Borehole D in which we did not encounter such fine-grained granular layers, which enhance drainage and the rate of consolidation). Settlement of these layers can be expected to take from about one-half year to about two years, depending on the thickness of clay in relation to silty sand/sandy silt layers.

The consolidation settlement of the underlying thick clay deposit (generally 24 to 28 m except in Borehole D where it is 33 m thick), will take place in excess of 30 years. Within the window of opportunity of six months available for preload/surcharge, the anticipated degree of consolidation of the clay can be expected to be about 10% (about 8% at Borehole D location) of the total consolidation settlement, based on an average coefficient of consolidation (c_v) value of 8×10^{-4} cm²/s from seven consolidation tests performed during the previous and present investigations.

The fills which will be placed within the creek flood plain will increase the settlements near the south abutment of the proposed bridges due to stress superimposition and this was taken into consideration when calculating the magnitude of settlements near the south abutments of the proposed bridges.

Organic soils undergo long-term secondary settlements and therefore, it is our recommendation that all the organic soils encountered to a depth of about 3.0 m below the o.g. in Boreholes 101, 102 and 103 be removed and replaced with granular soils (e.g. Granular 'B') in the vicinity of the proposed bridge structure. Assuming that conditions further north of Borehole 103 is similar to those encountered in Borehole 103, it is recommended that this removal and replacement process be continued for a distance of about 25 m from the north abutments to about Station 10+875, reducing gradually to zero about 20 m beyond at about Station 10+895.

On the south side in the previously drilled boreholes, organic soils were only encountered in Borehole F at about Station 10+810 to a depth of about 1.0 m. From this, it can be surmised that the organic soils on the south side extend to about Station 10+810, and this soil exchange can be assumed to be carried to about Station 10+815 to a depth of 3.0 m, reducing to zero about 10 m beyond this station (i.e. at about Station 10+805), for preliminary estimating purposes.

A better assessment of the organic soil removal can be made by drilling additional boreholes, particularly on the north side of the bridge. Actual stripping depths and the horizontal extent of this process should be controlled in the field during construction, under the supervision of the Geotechnical Engineer.

It should be pointed out that the unit weight of organic soils is less than Granular 'B' type material. When organic soils are removed and replaced with granular materials, therefore, the additional weight increases the stresses on the underlying soils. This increases the future settlements of the underlying compressible soils. This aspect was taken into consideration when carrying out settlement calculations.

After a six-month preload period, the calculated settlements would be reduced but up to about 1.0 m settlements could occur, where the fill is in excess of 5 m. Settlements of these magnitude are considered excessive, especially immediately adjacent to the bridges, as well for piled foundations where settlements should normally not exceed 25 mm. Away from the bridge structures, (e.g. beyond about 15 m to 30 m beyond the bridge abutments) settlements of the order of 200 to 300 mm may be tolerable depending on the view of local MTO personnel.

Table 5.4.1 shows the results of settlement calculations based on the available subsurface and laboratory data. These predict a settlement of about 1.0 m remaining after a six-month preload period at the creek location while at Station 10+920 where the fill height is only 1.0 m the anticipated remaining settlements are of the order of 0.1 m, after a preload period of six months.

To reduce these settlements, measures such as surcharging over sufficient periods of time, the use of light-weight fill, wick drains or a combination of these may be considered.

The following discussion is based on the premise that a period of only 6 months is available to effect some of these foundation settlements prior to the construction. If, however, a longer time duration should be available, the recommendations could be revised.

Speeding the rate of consolidation by means of wick drains was considered but in our opinion this is not a desirable option due to the fact that the wick drains will have to extend to considerable depths to be effective. Further, artesian conditions were observed both in the lower sandy silt/silty sand deposits underlying the extensive clay deposit below 40 m depth, as well as within these fine-grained granular soils interbeds in the upper zones of the clay. Time restraints for the installation and the anticipated difficulties in the installation of the wick drains are other considerations which lead us to conclude that the use of wick drains is not a good choice for this project.

Within the time restraints imposed (i.e. six months) surcharging will have only limited success in effecting consolidation settlements in the thick clay layer but it will be effective in ensuring that all the settlements within the silty sand/sandy silt deposit are realized. As well, a significant portion of the settlements within the upper zones of the clay where it is interbedded with silty fine sand/sandy silt layers will also be effected in addition to effecting some of the settlements (i.e. about 10%) in the underlying thick clay deposit. Because of

this, our recommendation is to apply surcharge as well as using light-weight fill (i.e. expanded polystyrene blocks) where required after surcharging.

Figures I-1 and I-2 in Appendix I show the proposed surcharge heights across the site. In essence, this consists of a surcharge height of 1.5 m on the north side of the proposed bridges. On the south side, the proposed surcharge height is 2.0 m at the abutment location, increasing to 4.0 m at Station 10+800, about 24 m south of the south abutment location. The proposed surcharge height remains at 4.0 m for a horizontal distance of about 40 m to Station 10+760, from thereon gradually decreasing to zero at Station 10+746.

If a 4 m high surcharge is considered excessive, then lesser surcharge heights can be considered. Table 5.4.1 provides, in addition to 4.0 m high surcharge, surcharge heights of 3.0, 2.0 and 1.5 m. Calculated foundation settlements remaining after a six-month surcharge period at the existing creek area range from about 840 mm for a 4.0 m surcharge height to 980 mm for 1.5 m high surcharge. As these figures represent excessive settlements, surcharging will need to be considered with the use of expanded polystyrene blocks (EPS), where required. At other locations where the proposed embankment heights are relatively less the calculated settlements are also less, but in most cases, still excessive and surcharging will need to be combined with the use of EPS. For example, at Station 10+794 area where the maximum height of the proposed embankment is 4.4 m, settlements of up to 600 mm could occur after surcharging 4.0 m for a period of six months.

In addition, when considering surcharge heights, the foundation stability of the embankments would need to be considered.

TABLE 5.4.1
CALCULATED SETTLEMENTS

LOCATION	Embankment Height Used for Calculations	Calculated Total Settlement (mm)	Remaining Settlement After 6 month Preload Period (mm)	Remaining Settlement After 6 month Surcharge Period (mm)	Remaining Settlement After 3 Years* (mm)	Remaining Settlement After 10 Years * (mm)	Remaining Settlement After 30 Years * (mm)
Existing Creek Bed Area Sta 10+760 to 10 + 780	5.2 m fill (no surcharge)	1250	1020	n/a	770	580	300
	5.2 m fill plus 1.5 m surcharge for 6 months	1650	n/a	980	740	560	280
	5.2 m fill plus 2.5 m surcharge for 6 months	1940	n/a	950	720	550	270
	5.2 m fill plus 3.0 m surcharge for 6 months	2000	n/a	890	715	540	260
	5.2 m fill plus 4.0 m surcharge for 6 months	2250	n/a	840	675	525	250
	5.2 m fill plus 4.0 m surcharge for 6 months plus 1.5 m EPS	n/a	n/a	480	375	290	145
	5.2 m fill plus 4.0 m surcharge for 6 months plus 2.0 m EPS	n/a	n/a	340	270	205	100
	5.2 m fill plus 3.0 m surcharge for 6 months plus 1.5 m EPS	n/a	n/a	500	395	300	150
	5.2 m fill plus 3.0 m surcharge for 6 months plus 2.0 m EPS	n/a	n/a	360	280	215	105
	5.2 m fill plus 3.0 m surcharge for 6 months plus 2.7 m EPS	n/a	n/a	195	150	115	60
	5.2 m fill plus 2.5 m surcharge for 6 months plus 2.0 m EPS	n/a	n/a	400	290	220	110
	5.2 m fill plus 2.5 m surcharge for 6 months plus 2.7 m EPS	n/a	n/a	225	165	120	65

LOCATION	Embankment Height Used for Calculations	Calculated Total Settlement (mm)	Remaining Settlement After 6 month Preload Period (mm)	Remaining Settlement After 6 month Surcharge Period (mm)	Remaining Settlement After 3 Years* (mm)	Remaining Settlement After 10 Years * (mm)	Remaining Settlement After 30 Years * (mm)
	*** 5.2 m fill plus 1.5 m surcharge for 6 months plus 2.7 m EPS	n/a	n/a	250	180	135	70
	*** 5.2 m fill plus preload for 6 months (no surcharge) plus 2.7 m EPS	n/a	n/a	290	200	150	75
Sta 10 + 794 **	4.4 m fill (no surcharge)	1100	760	n/a	560	420	210
	4.4 m fill plus 4.0 m surcharge for 6 months	2100	n/a	600	440	310	165
	4.4 m fill plus 3.0 m surcharge for 6 months	1960	n/a	630	455	320	170
	4.4 m fill plus 4.0 m surcharge for 6 months plus 1.3 m EPS	n/a	n/a	315	250	185	95
	4.4 m fill plus 3.0 m surcharge for 6 months plus 1.3 m EPS	n/a	n/a	340	270	200	100
	4.4 m fill plus 4.0 m surcharge for 6 months plus 2.0 m EPS	n/a	n/a	150	115	90	45
	4.4 m fill plus 3.0 m surcharge for 6 months plus 2.0 m EPS	n/a	n/a	165	130	100	50
South Abutment Area Station 10+820 ** ****	1.3 m fill plus 3.0 m soil replacement due to 2.0 m thick organic layer (soil replacement) no surcharge	520	300	n/a	180	130	60
	1.3 m fill plus 2.0 m surcharge plus soil replacement	1100	n/a	330	135	100	45

LOCATION	Embankment Height Used for Calculations	Calculated Total Settlement (mm)	Remaining Settlement After 6 month Preload Period (mm)	Remaining Settlement After 6 month Surcharge Period (mm)	Remaining Settlement After 3 Years* (mm)	Remaining Settlement After 10 Years * (mm)	Remaining Settlement After 30 Years * (mm)
	1.3 m fill plus 2.0 m surcharge plus soil exchange plus 1.2 m EPS	n/a	n/a	40	30	20	10
	1.3 m fill plus 2.0 m surcharge plus soil exchange plus 1.5 m of EPS	n/a	n/a	25	20	15	8
North Abutment Area Sta 10 + 852 ****	1.2 m fill (no surcharge) plus 3.0 m soil replace- ment due to 2.0 m thick organic layer	280	150	n/a	100	75	35
	1.2 m fill plus 1.5 m surcharge plus soil replacement	570	n/a	130	85	65	30
	1.2 m fill plus 1.5 m surcharge plus soil replacement plus 1.2 m EPS	n/a	n/a	18	14	10	7
Sta 10 + 920 Area	1.0 m fill (no surcharge) plus soil replacement to about 3.0 m due to 2.0 m thick organic layer (soil replacement)	230	115	n/a	85	60	30
	1.0 m fill plus 1.5 m surcharge plus soil replacement	480	n/a	85	65	50	20
	1.0 m fill (no soil replacement) (no surcharge)	260	180	n/a	95	60	30

LOCATION	Embankment Height Used for Calculations	Calculated Total Settlement (mm)	Remaining Settlement After 6 month Preload Period (mm)	Remaining Settlement After 6 month Surcharge Period (mm)	Remaining Settlement After 3 Years* (mm)	Remaining Settlement After 10 Years * (mm)	Remaining Settlement After 30 Years * (mm)
	1.0 m fill plus 1.5 m surcharge (no soil replacement)	500	n/a	110	80	50	25

* after the completion of 6 mo. preload/surcharge period

** assumes no organic soils underlying the site

*** we are not recommending less than 2.5 m of surcharge, as less than 2.5 m of surcharge is unlikely to be sufficient to effect the consolidation settlement of the upper zones of the clay with sand and silt seams

**** surcharge will be placed some distance from the abutment location as shown in Figures J-1 and K-1.

EPS expanded polystyrene blocks

Stability analysis was carried out by means of limit state equilibrium, utilizing the computer program Slope/W. In the analysis Bishop's Simplified method was utilized, for both short-term (undrained) and long-term (drained) analyses calculations.

The calculated safety factor for a 9.2 m high embankment (i.e. 5.2 m grade raise plus 4.0 m surcharge fill) with 2H:1V side slopes is about 1.26 in the short-term and about 1.29 in the long-term, as shown in Figures I-3 and I-4 in Appendix I. These figures are considered acceptable, due to the temporary nature of the fill, as well since the pore pressures will be monitored during and after the fill placement.

For 3H:1V side slope, the calculated safety factors increase as shown in Figures I-5 and I-6. In our calculations, the groundwater table was assumed at the original ground surface (o.g.) level. The calculated factors of safety range from 1.32 to 1.74.

Based on the calculations, surcharge heights of up to 4 m are considered feasible.

We recommend the installation of geotechnical foundation instrumentation to monitor pore pressures and settlements.

The following construction sequence is recommended.

- Strip the site of all unsuitable soils (including the approximately 2 m thick organic silt to about 3 m depth in the vicinity of the cut channel).
- Install instrumentation (i.e. piezometers and settlement monitoring plates) to monitor and regulate the placement of surcharge fills.
- Construct the new channel for the creek.
- Place embankment fills in accordance with MTO/OPSS. The use of granular soils such as Granular 'B' is recommended below the groundwater table (e.g. about El. 182 m). Above this level, SSM or suitable earth fill can be used, but since Granular 'B' (Type I) material is readily and economically available in the general area, the use of Granular 'B' for the entire embankment fill would be desirable. On site excavated silty sands or the surficial silty clay (encountered within the upper 1.2 m in Boreholes 101, 102 and 103) can also be used, provided that their moisture contents at the time of excavation are suitable and weather conditions during construction are favourable.
- Place the lower 1.5 m of the surcharge fills. The surcharge fills can consist of inorganic suitable earth fill. Topsoil, organic silt or other on-site excavated unsuitable soils should not be used nor should any materials that are too wet (or too dry) to achieve a suitable degree of compaction. The placement of surcharge fills should be

carried out with the aid of instrumentation results with regards to generated pore pressures. When defining the interface between the surcharge and the top of embankment fill, the magnitude of anticipated settlements may need to be taken into consideration.

- A period of about one month should be allowed, depending on the results of monitoring by the instruments before and during the placing of the remainder of the surcharge fills. The surcharge fills should be left in place for a period of six months but preferably longer, if feasible.
- The instrumentation should be monitored during the surcharge period and after this period for as long as practically feasible.
- The piles which would support the bridge structures must be driven after the fill surcharge period and the removal of the surcharge fills. A sketch of surcharge configuration at the bridge and cut channel is given in Figure J-1 in Appendix J.
- After the removal of surcharge fills, in the area of proposed expanded polystyrene (EPS) blocks, the embankment fill would be removed to the bottom of the light-weight fill layer and replaced with EPS, prior to the construction of the road pavement structure.

Table 5.4.1 shows that the anticipated settlements after a six-month surcharge period are between 85 mm (Station 10+920; 1.0 m fill + 1.5 m surcharge + soil replacement) and 840 mm (existing creek bed area; 5.2 m fill + 4.0 m surcharge), depending on the location and height of embankment fill and surcharge. When looking into these settlement figures, it should be remembered that the anticipated time frame for the settlements is also important, in addition to the magnitudes. For example, under an average 5.2 m embankment fill and 4.0 m high surcharge, a total settlement of 840 mm can be anticipated (within the next 50 years or so) after a six-month surcharge period, but the calculated settlement within the first three years after surcharge is only 165 mm (i.e. 840 minus 675 mm). For the ten-year period after surcharge, the anticipated settlement is 315 mm (i.e. 840 minus 525 mm), etc. With this in mind, the following are our specific recommendations:

- (a) South Abutment Area – The anticipated residual settlements after surcharging (2.0 m) for a period of six months is 330 mm (including settlements due to stress increase from the replacement of the 2 m thick organic soil layer, as well as stress superimposition due to filling of the existing creek bed and valley). The predicted settlements after 30 years are about 285 mm (i.e. 330 mm minus 45 mm – see Table 5.4.1 row labeled 1.3 m fill plus 2.0 m surcharge plus soil replacement), which in our opinion are excessive for pile foundations and bridge abutments, particularly since the surcharge may not be totally effective at the immediate abutment locations. This is because a 5.5 m set-back from edge of

the deeper portion of the cut slopes, which is required in order to prevent a slope failure, as shown in Figure J-1 in Appendix J. This was based on slope stability analyses carried out by means of limit state equilibrium, utilizing the computer program Slope/W. In the analysis Bishop's Simplified method was utilized, for both short-term (undrained) and long-term (drained) analyses calculations. Typical stability analyses results are given in Appendix J.

In view of predicted settlements, which are in excess of normally accepted values of about 25 mm, it is recommended that after the surcharging (and driving of the piles), the 1.5 m thick EPS layer be placed at a depth of about 1.2 m below the finished grade (see Figure K-1). For this purpose, the embankment fill should be sub-excavated to a depth of 2.7 m (i.e. 1.2 m + 1.5 m) and then EPS block geofoam be placed. This should extend laterally to a horizontal distance not less than 10 m beyond the abutment wall. With this approach, the predicted foundation settlements due to the weight of the fill within a 30-year period after surcharging are less than 25 mm.

- (b) Stations 10+810 to 10+800 – In this area, the height of the embankment fill ranges from about 2 to 4 m and the recommended surcharge height also range from 2 to 4 m. It is recommended that a minimum 1.0 m thick EPS layer be provided (see Figure K-1 in Appendix K). The predicted maximum settlements within a period of 30 years range from about 30 mm to about 100 mm.
- (c) Stations 10+800 to 10+780 – In this area, the height of embankment fill generally ranges from 4.0 to 4.4 m and the recommended surcharge height is 4.0 m. It is recommended that the thickness of the EPS layer be gradually increased to 2.0 m at Station 10+780. With this approach, the predicted settlement after a 30-year period after surcharging at Station 10+794 is about 220 mm (i.e. 315 minus 95 mm).
- (d) Stations 10+780 to 10+767 – This area includes the existing creek bed where the height of the embankment fill reaches 5.6 m. In this area, in spite of surcharging excessive settlements can be expected unless a thick layer of EPS is placed. A 2.0 m thick EPS can be expected to reduce future settlements within 30 years after surcharging to about 255 mm (i.e. 360 minus 105 mm) and this is recommended. A thicker EPS layer will further reduce future settlements. If in the opinion of MTO personnel, the cost is unwarranted, the thickness of the EPS layer can be kept at 1.5 m in which case a total settlement of about 335 mm can be expected after 30 years.

The thickness of the EPS layer can be gradually decreased to zero at Station 10+758, as shown in Figure K-1.

It should be pointed out that these recommendations are based on centerline profile. As the original ground (o.g.) level varies considerably in this area, the thickness of the fill also varies. Therefore, the recommended thickness of EPS would be different at WBL and EBL, especially to the south of Station 10+770, where the embankments on the left side are higher (i.e. filling of old creek bed) while on the right side they may be shallower and less EPS would suffice. The variations in the thickness of embankment fill at any given cross-section may lead to differential settlements across the width of the roadway. We will be pleased to discuss these details further, if required. This may apply to a lesser extent between Stations 10+770 and 10+800 where the o.g. is somewhat more level.

- (e) North Abutment – At the north abutment location, the height of the embankment fill will be about 1.2 to 1.3 m and the recommended height of the surcharge is 1.5 m. A recommended surcharge configuration immediately adjacent to the cut channel is given in Figure J-1 (Appendix J) as was discussed for the south abutment.

The anticipated settlements under this fill (including stress due to organic soil replacement), after 6 months surcharging by 1.5 m, are about 100 mm after a 30-year period (i.e. 130 mm minus 30 mm, Table 5.4.1). Such settlements are unlikely to be within tolerable limits for bridge foundations and the placement of 1.2 m thick EPS layer, similar to south abutment is recommended. In any event, we understand that the placement of a similar EPS layer is desirable adjacent to the north abutment similar to south abutment, from structural engineering point of view. We recommend that this 1.2 m thick EPS layer be extended to at least 6 m beyond the abutment wall, its thickness gradually reducing to zero about another 10 m beyond. If this is done, the settlements at the abutment location (and to 6 m beyond) should be less than 25 mm.

The predicted settlements beyond this point (i.e. to the north of Station 10+867) is about 100 mm within the 30 years following the completion of the surcharge and this is considered acceptable for a flexible pavement structure, without causing any visible distress in the pavement structure itself.

- (f) Stations 10+890 to 11+000 – As was discussed before, there is no foundation subsurface data in this area. We have the following comments and recommendations to make assuming subsurface conditions encountered in Boreholes 101 and 103 are representative. As was also mentioned before, we recommend that the 2.0 m thick organic silt be removed to a distance of about 25 m beyond the north abutment (i.e. to about Station 10+875) gradually reducing to zero about 20 m beyond at about Station 10+895.

We recommend that this area (i.e. Stations 10+890 to 11+000) be surcharged by placing about 1.5 m of fill above the final grades for as long as possible but not less than six months. In this manner, the anticipated settlements within the next 30 years are calculated to be less than 100 mm, but would continue at a slower pace beyond this time frame, due to the organic nature of the 2.0 m thick silt layer (i.e. secondary consolidation) which will be left in place. In our opinion, settlements of this magnitude are within acceptable limits for flexible pavements and this is a reasonable approach. If this is not acceptable to MTO then all the organic soils can be replaced. An EPS layer of suitable thickness can also be used.

More detailed recommendations can be provided if subsurface conditions are investigated prior to construction.

As was mentioned before, across the site a combination of surcharge heights and EPS thickness may be selected to provide the most cost-effective solution. It should, however, be pointed out a surcharge of less than 2.5 m may be somewhat risky as lesser amounts of surcharge may not be fully effective in effecting the settlement of the upper zones of the clay interbedded with sand/silt layers within the anticipated surcharge period.

It should also be pointed out that in our recommendations we have assumed that a minimum cover of 1.2 m will be provided above EPS (i.e. the EPS will be placed 1.2 m below finished grade). If, however, hydrological conditions permit, it should be placed lower to provide more cover against a potential uplift condition.

The design and construction of EPS should be in accordance with MTO Special Provision entitled "Expanded Polystyrene Embankment." The design and construction is the responsibility of the contractor. Rebound of the soil is not a concern at this site. Embankments may need to be provided with a widened cross-section to allow for settlements of the underlying soils and any possible future minor grade raises.

5.5 CONSTRUCTION

The high watertable in the floor of the valley will likely necessitate some drainage and/or surficial dewatering during stripping, subsequent proofrolling (where practical) and fill placement. For this reason, it is our opinion that a granular fill will likely be necessary in the low-lying areas, until the fill reaches the existing ground surface level or even slightly higher, depending on the construction season and site conditions. The dewatering will likely consist of gravity drainage and pumping from strategically placed filtered sumps.

It should be pointed out the surficial fine-grained granular soils (i.e. silty fine sand to sandy silt) are highly erodible and frost susceptible materials. These aspects should be considered in the design.

The side slopes should be protected against erosion during construction and permanently, including rock protection placed to the high water level, as per hydrological considerations. The rock protection should be separated from the native soils or embankment material with a geotextile filter fabric or a filter zone of granular material. The filter fabric should have a filtering opening size (F.O.S.) not larger than 120 microns.

As was mentioned before, artesian conditions were observed in the boreholes, emanating from the lower sandy silt/silty sand and also in the sand and silt interlayers in the upper zones of the clay deposit. It is likely that the clay will seal the piles and prevent water and soil particles from migrating upwards along the pile. It is, however, recommended that conditions at the site be carefully observed to detect such occurrence during and several weeks after the driving of the piles. In addition, an NSSP be provided in this contract to deal with this eventuality should this happen. This could consist of standard MTO inverted filter at the base of the piles, as shown in Appendix L.

We understand that it is impractical to place subdrains in the inverted filter, since there is no place to drain these. In this case, if desired, the drains can be omitted. We understand that the flex zone may be provided by augering a 600 mm diameter hole 3000 mm deep and filling the hole with uniform sand, as was discussed in Section 5.1.2. In this case, care should be exercised so that the geotextile of the inverted filter will not be unduly damaged. In addition, since the hole will be extended below the water table, the employment of a temporary casing will likely be required to prevent the caving-in of the silty fine sand subgrade. It is also recommended that an approximately 0.2 to 0.3 m thick bentonite layer be placed at the bottom of the hole (before placing the sand fill) to reduce the risk of silt being forced up (by possible artesian condition) through the uniform sand backfill.

5.6 FROST PROTECTION

Design frost protection depth for the general area is 1.8 m. Therefore, a permanent soil cover of 1.8 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps. In case of rip-rap (rock fill), only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

6. CLOSURE

We recommend that during finalizing of the details of the twin bridges, close liaison be maintained with the foundation (geotechnical) consultant to select optimum solutions regarding settlement, fill stability, surcharging, etc. issues, as well as reviewing recommendations contained in this report for their specific applicability.

The Limitations of Report, as quoted in Appendix M, are an integral part of this report.

SHAHEEN & PEAKER LIMITED



Z.S. Ozden, P.Eng.



Ramon Miranda, P. Eng.



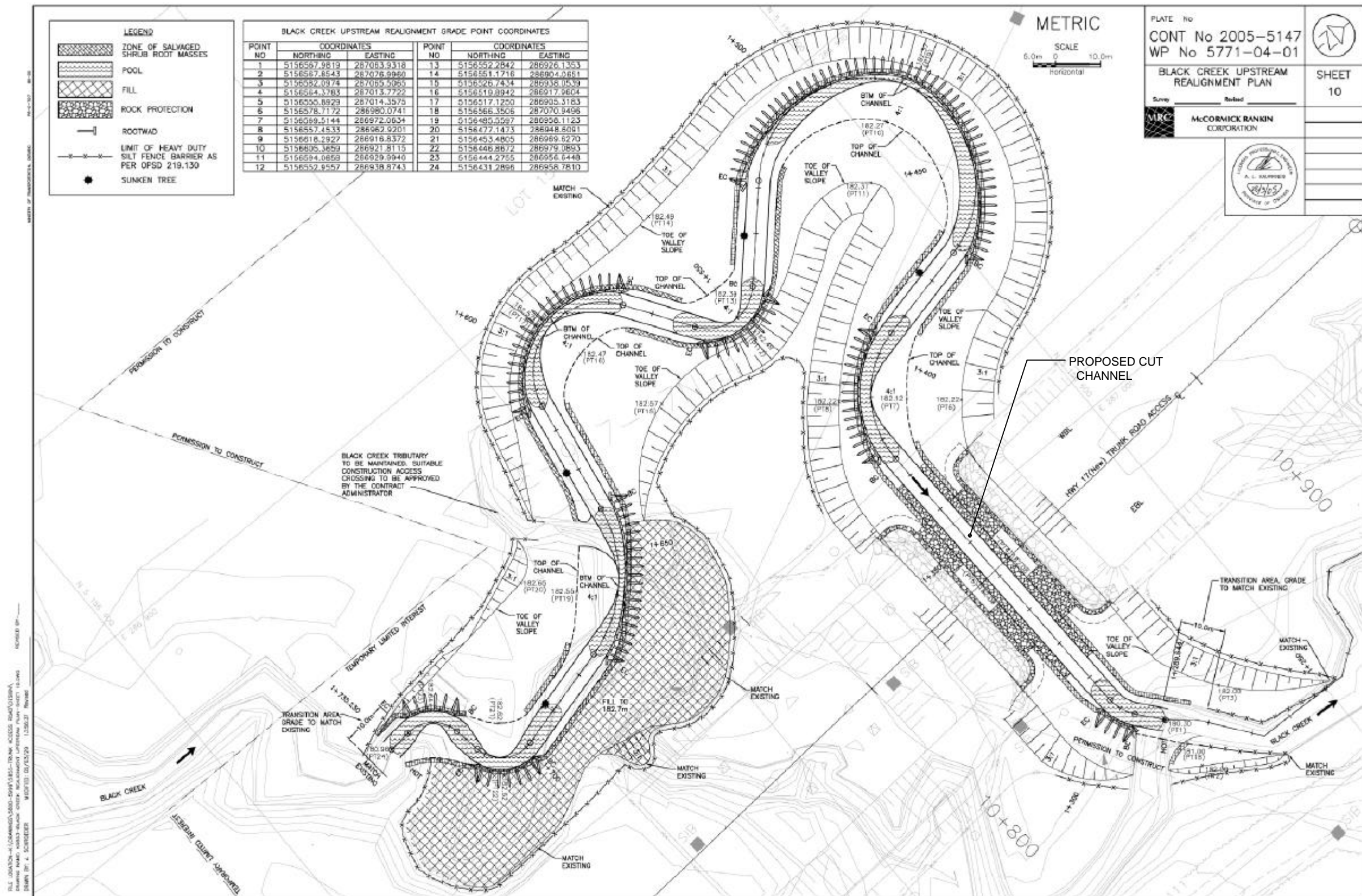
K. R. Peaker, Ph.D., P.Eng.



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Appendix G

Cut Channel



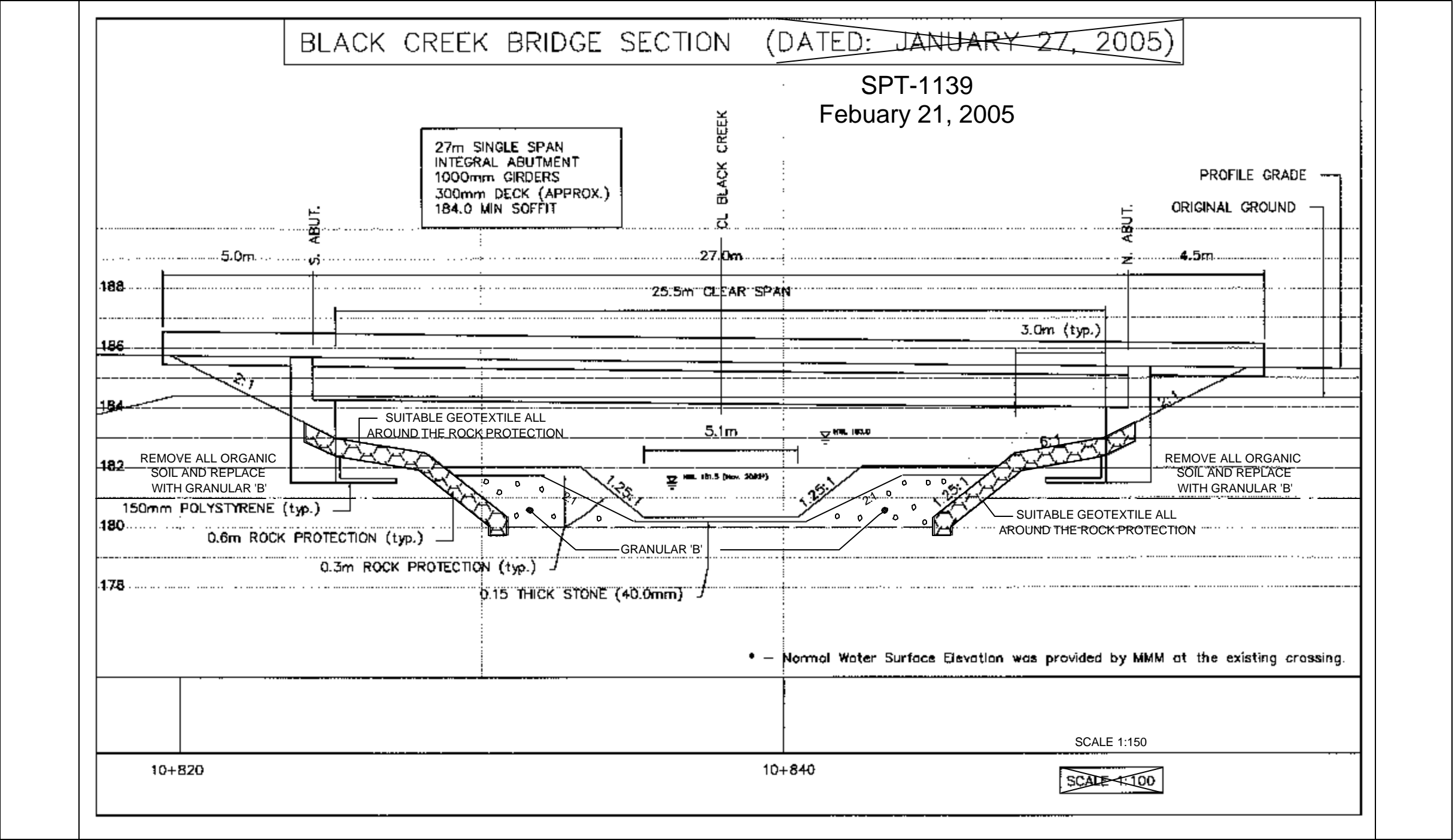


FIGURE G-2
PROPOSED CHANNEL CONFIGURATION

Appendix H

Proposed Profile

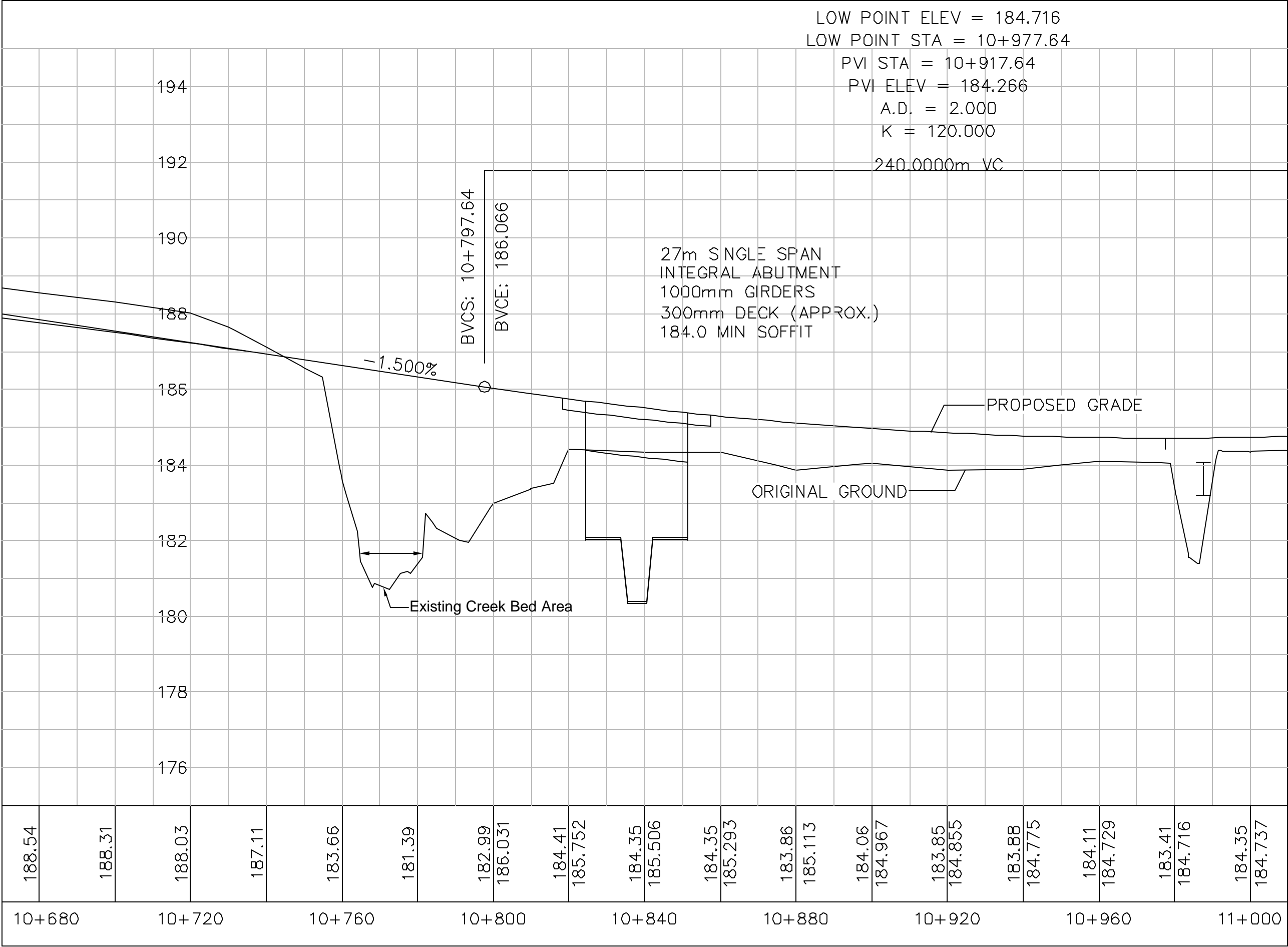


FIGURE H-1

N.T.S.

Appendix I

Surcharge Profile and Typical Slope Stability Analyses Results

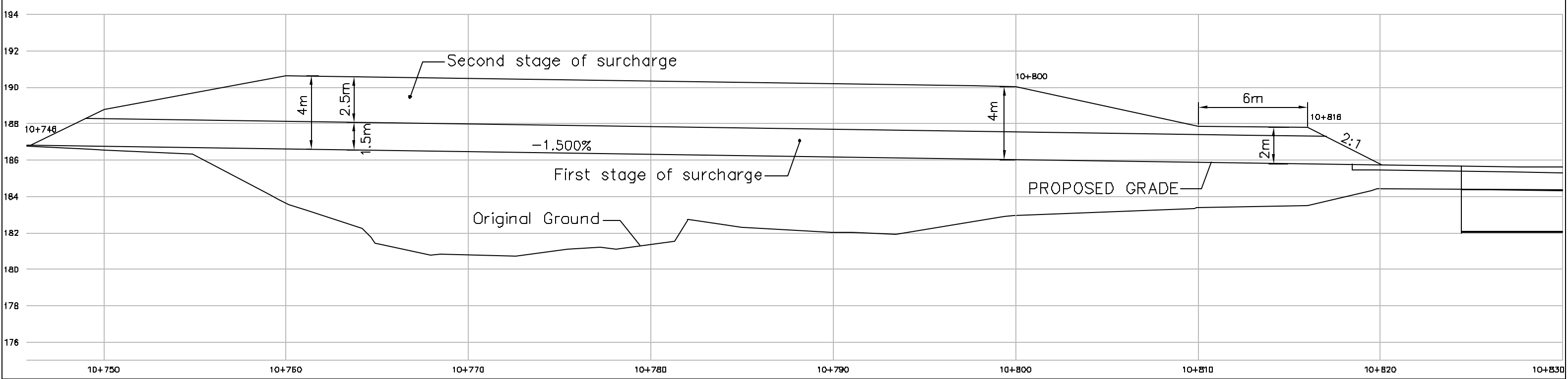


FIGURE I-1
PROPOSED SURCHARGE PROFILE
SOUTH SIDE OF PROPOSED BRIDGES

N.T.S.

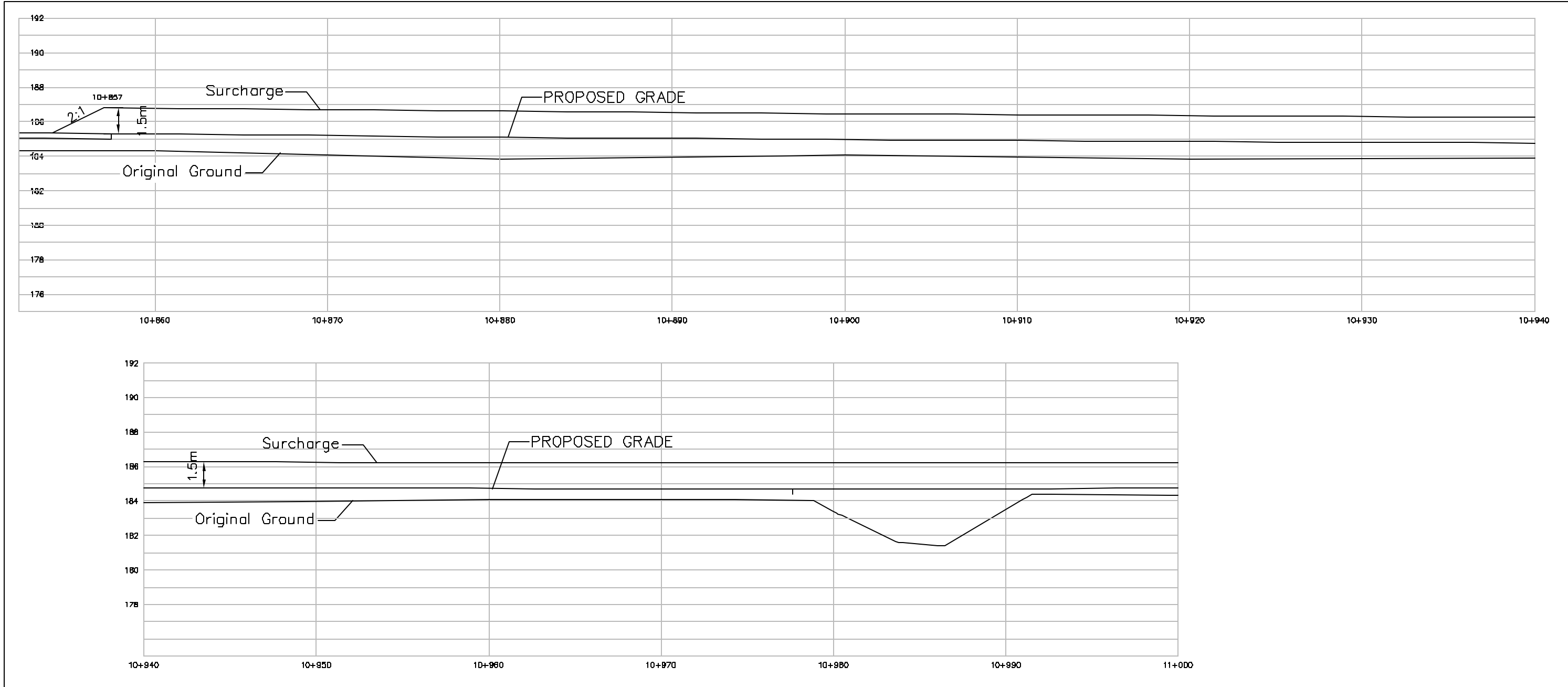


FIGURE I-2
PROPOSED SURCHARGE PROFILE
NORTH SIDE OF PROPOSED BRIDGES

N.T.S.

Highway 17, Trunk Access Road
Sault Ste. Marie, ON

Undrained Case (Short-Term Analysis)

Analysis Method: Bishop

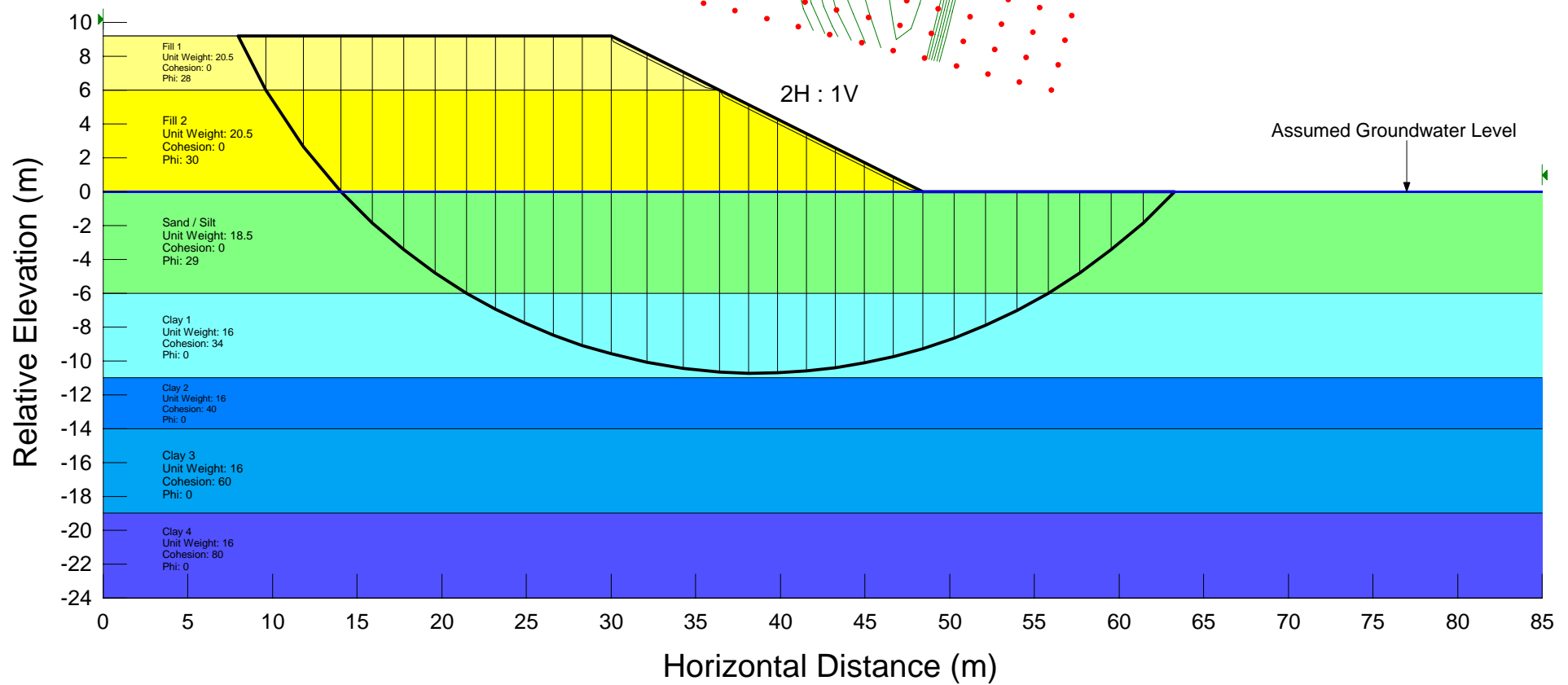


Figure I-3

Highway 17, Trunk Access Road
Sault Ste. Marie, ON

Drained Case (Long-Term Analysis)

Analysis Method: Bishop

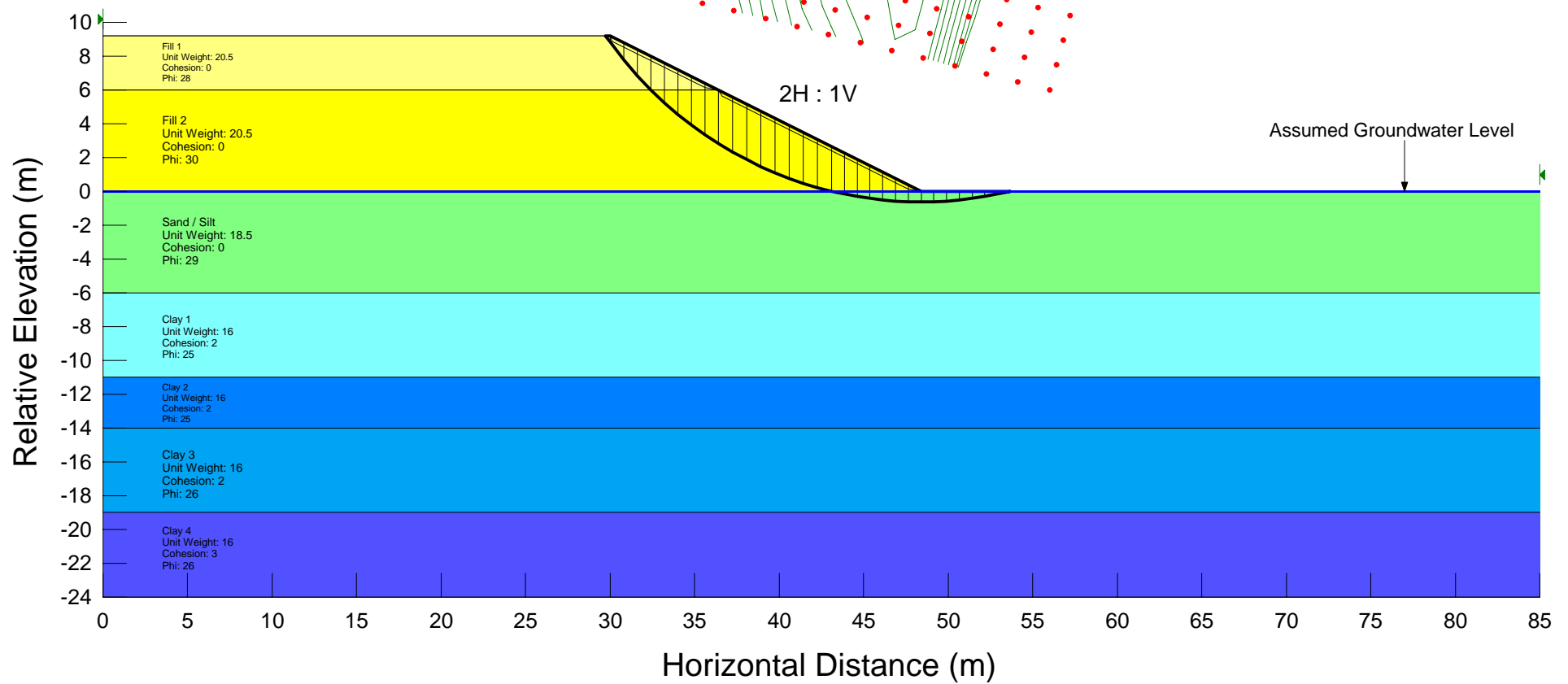


Figure I-4

Highway 17, Trunk Access Road
Sault Ste. Marie, ON

Undrained Case (Short-Term Analysis)

Analysis Method: Bishop

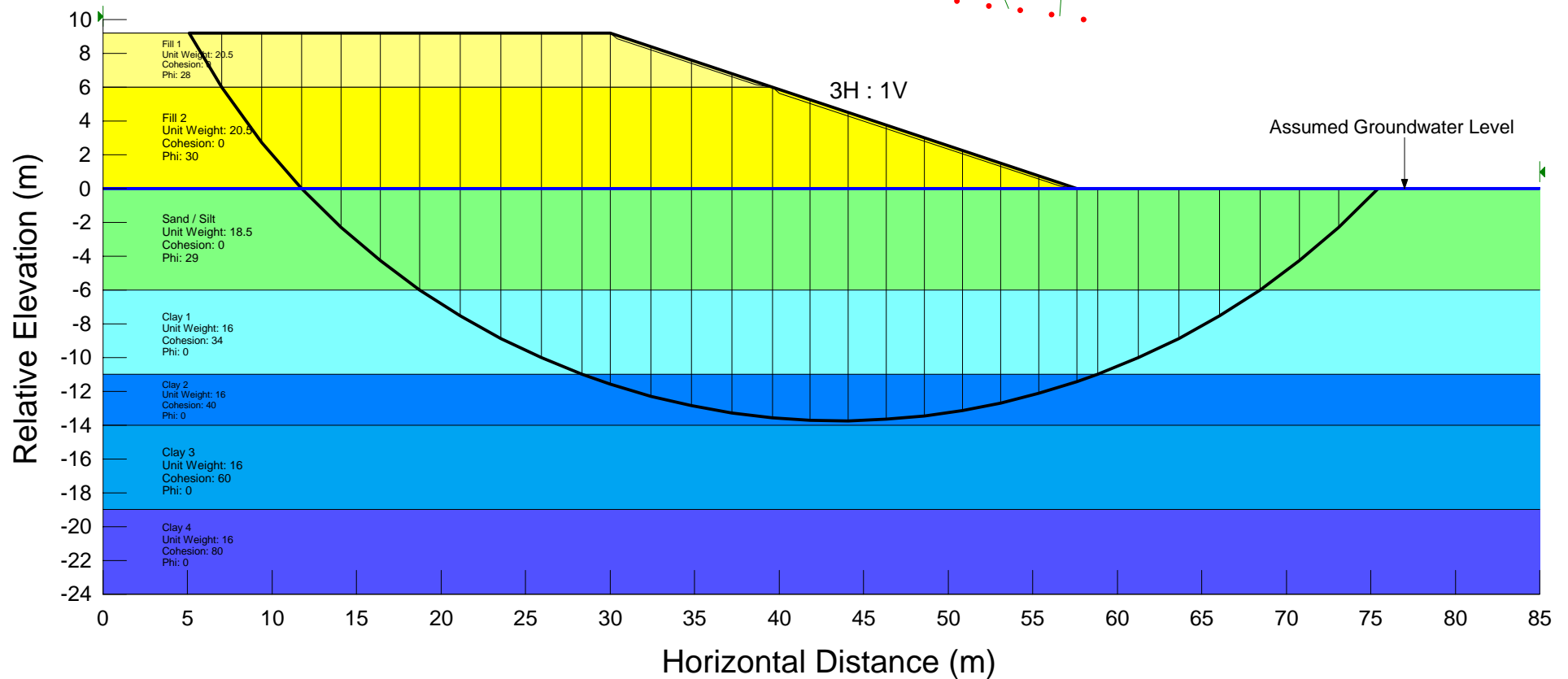
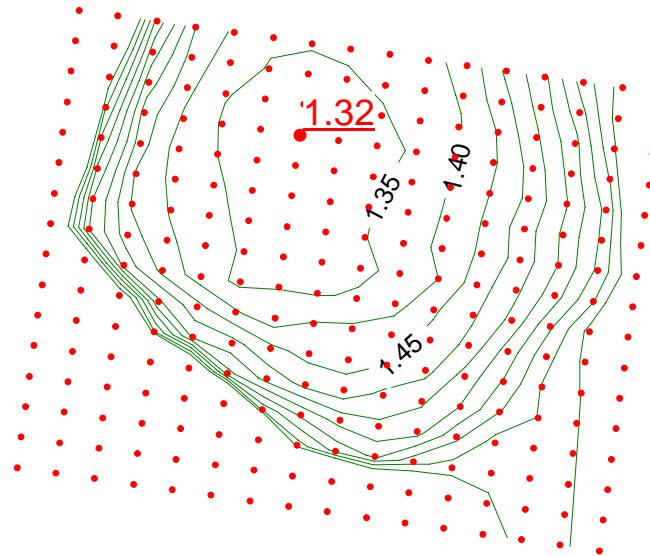


Figure I-5

Highway 17, Trunk Access Road
Sault Ste. Marie, ON

Drained Case (Long-Term Analysis)

Analysis Method: Bishop

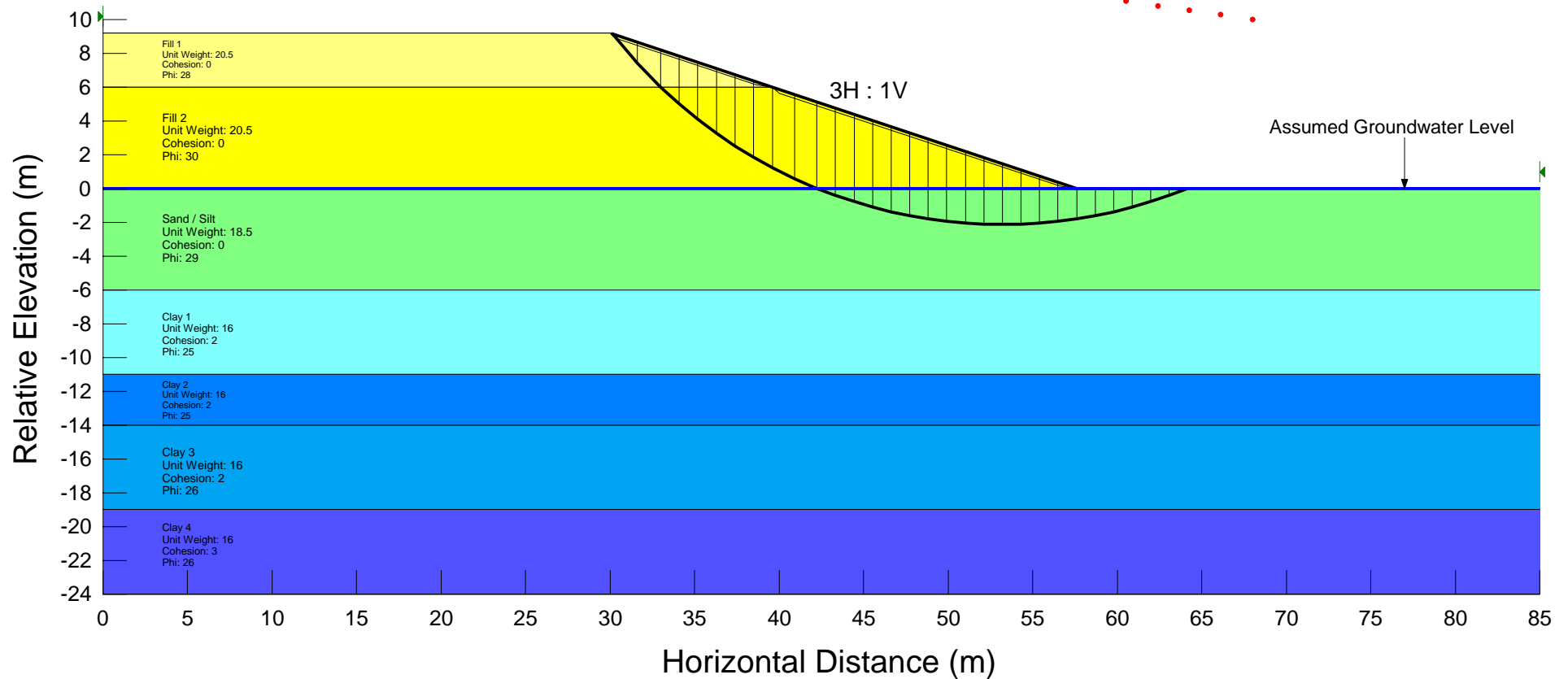
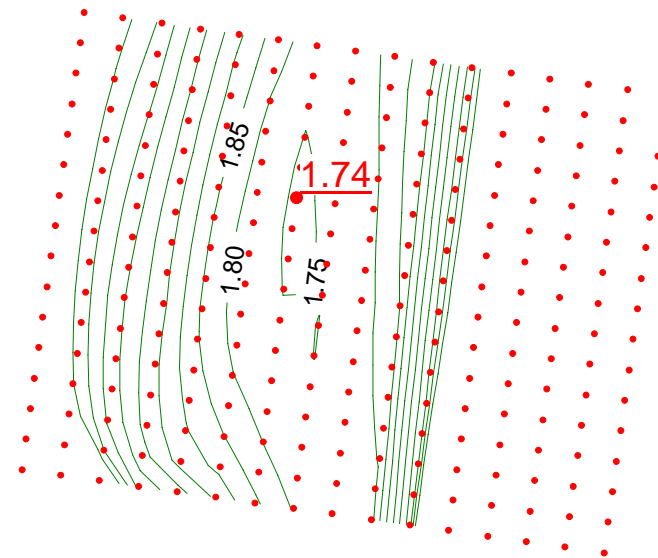


Figure I-6

Appendix J

Surcharge Configuration at Cut Channel and Typical Slope Stability Analyses Results

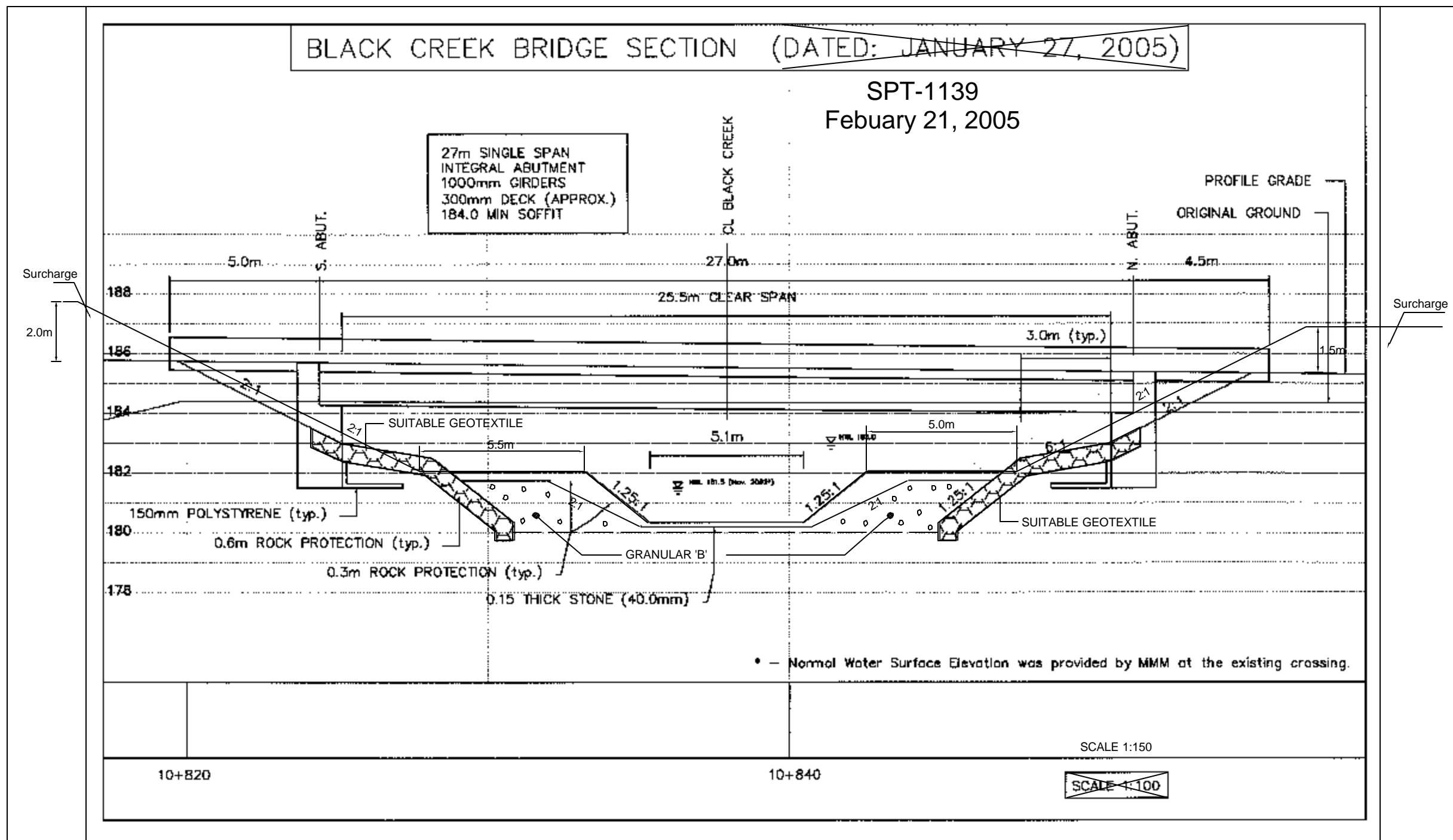


FIGURE J-1

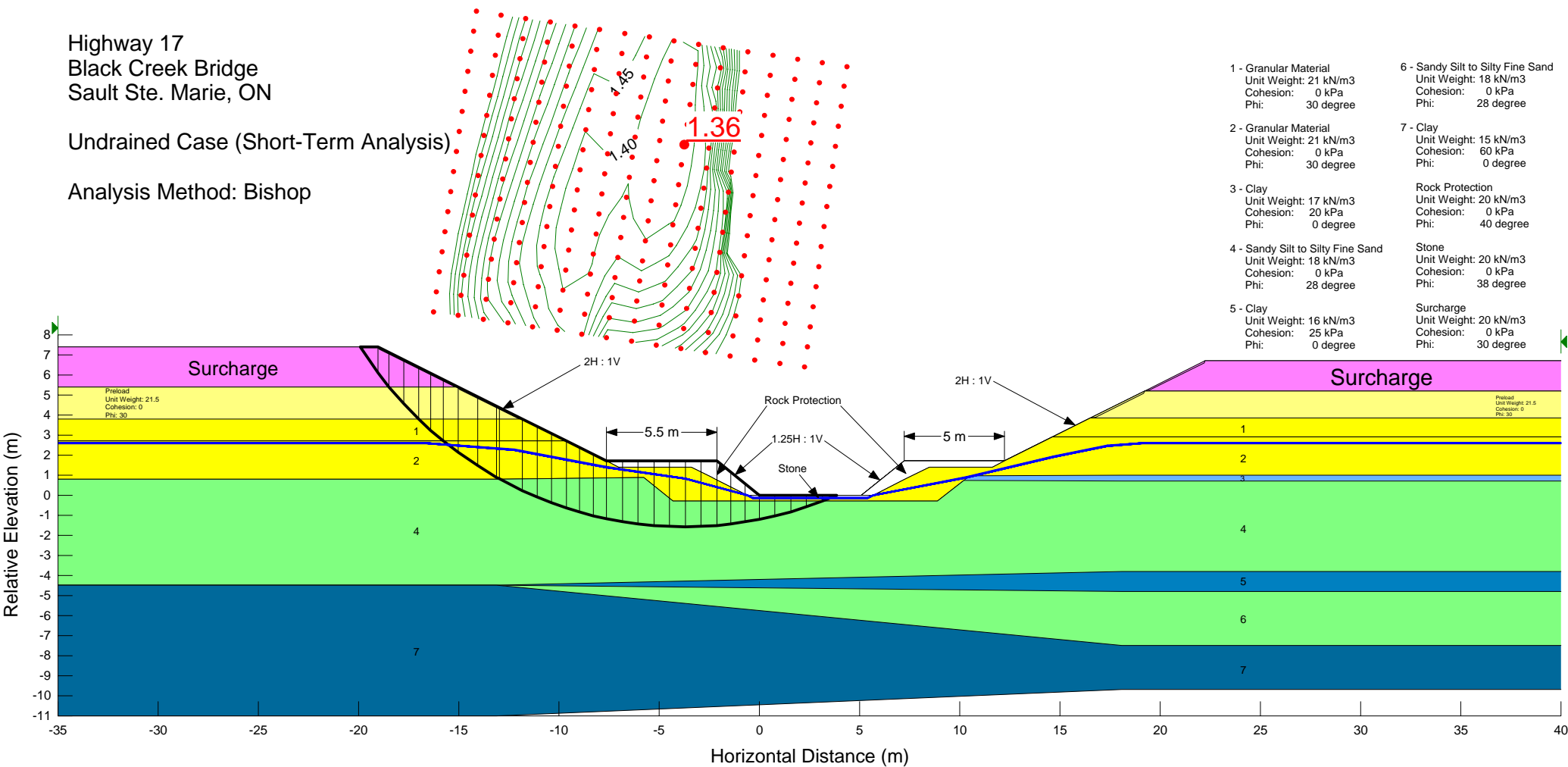
PROPOSED SURCHARGE AT CUT CHANNEL

Highway 17
Black Creek Bridge
Sault Ste. Marie, ON

Undrained Case (Short-Term Analysis)

Analysis Method: Bishop

- | | |
|---|---|
| 1 - Granular Material
Unit Weight: 21 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 6 - Sandy Silt to Silty Fine Sand
Unit Weight: 18 kN/m ³
Cohesion: 0 kPa
Phi: 28 degree |
| 2 - Granular Material
Unit Weight: 21 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 7 - Clay
Unit Weight: 15 kN/m ³
Cohesion: 60 kPa
Phi: 0 degree |
| 3 - Clay
Unit Weight: 17 kN/m ³
Cohesion: 20 kPa
Phi: 0 degree | Rock Protection
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 40 degree |
| 4 - Sandy Silt to Silty Fine Sand
Unit Weight: 18 kN/m ³
Cohesion: 0 kPa
Phi: 28 degree | Stone
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 38 degree |
| 5 - Clay
Unit Weight: 16 kN/m ³
Cohesion: 25 kPa
Phi: 0 degree | Surcharge
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree |



Highway 17
Black Creek Bridge
Sault Ste. Marie, ON

Drained Case (Long-Term Analysis)

Analysis Method: Bishop

- | | |
|---|---|
| 1 - Granular Material
Unit Weight: 21 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 6 - Sandy Silt to Silty Fine Sand
Unit Weight: 18 kN/m ³
Cohesion: 0 kPa
Phi: 28 degree |
| 2 - Granular Material
Unit Weight: 21 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 7 - Clay
Unit Weight: 15 kN/m ³
Cohesion: 2 kPa
Phi: 25 degree |
| 3 - Clay
Unit Weight: 17 kN/m ³
Cohesion: 2 kPa
Phi: 25 degree | Rock Protection
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 40 degree |
| 4 - Sandy Silt to Silty Fine Sand
Unit Weight: 18 kN/m ³
Cohesion: 0 kPa
Phi: 28 degree | Stone
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 38 degree |
| 5 - Clay
Unit Weight: 16 kN/m ³
Cohesion: 2 kPa
Phi: 25 degree | Surcharge
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree |

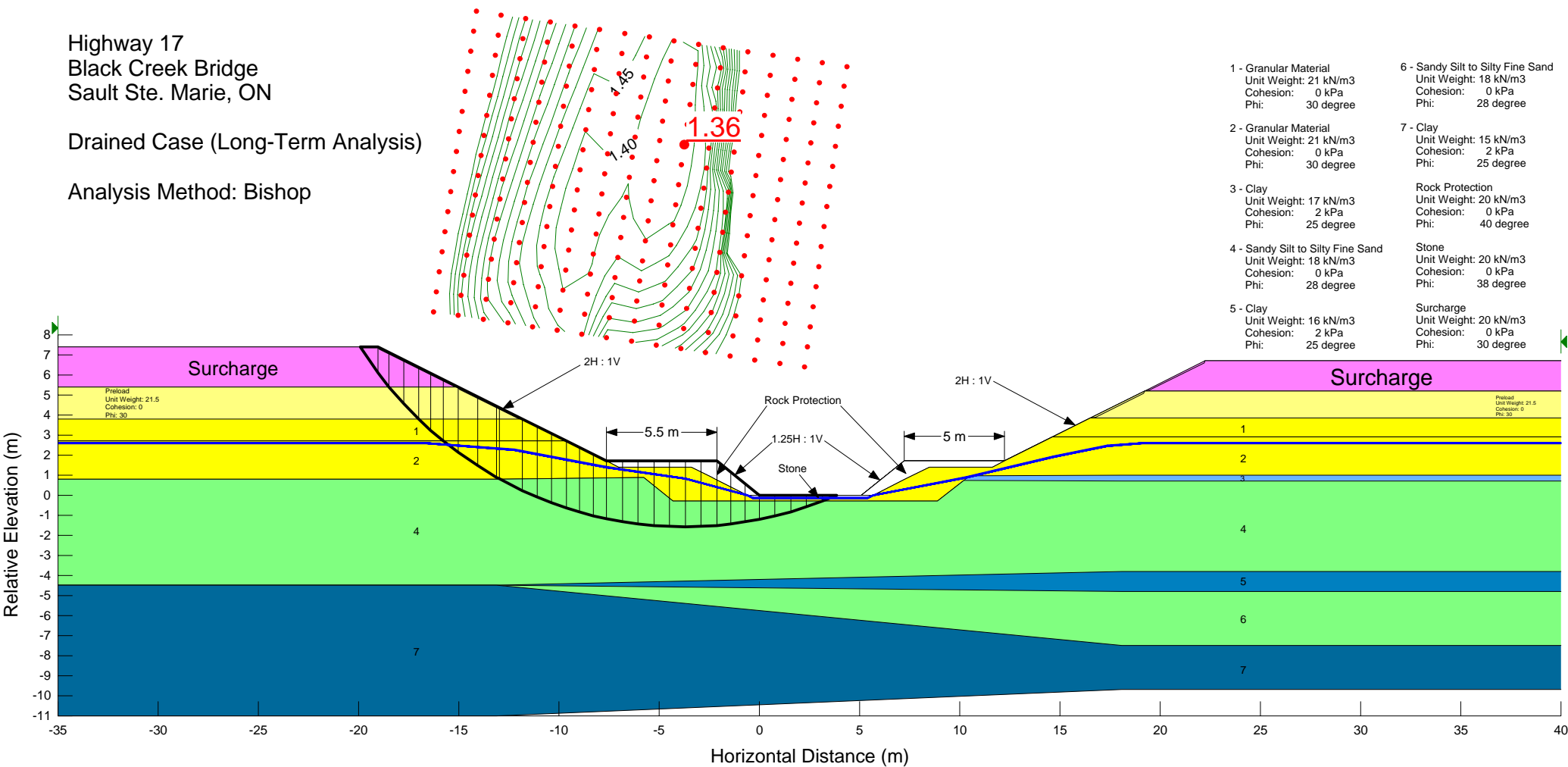
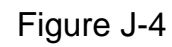


Figure J-3

Analysis Method: Bishop



Highway 17
Black Creek Bridge
Sault Ste. Marie, ON

Drained Case (Long-Term Analysis)

Analysis Method: Bishop

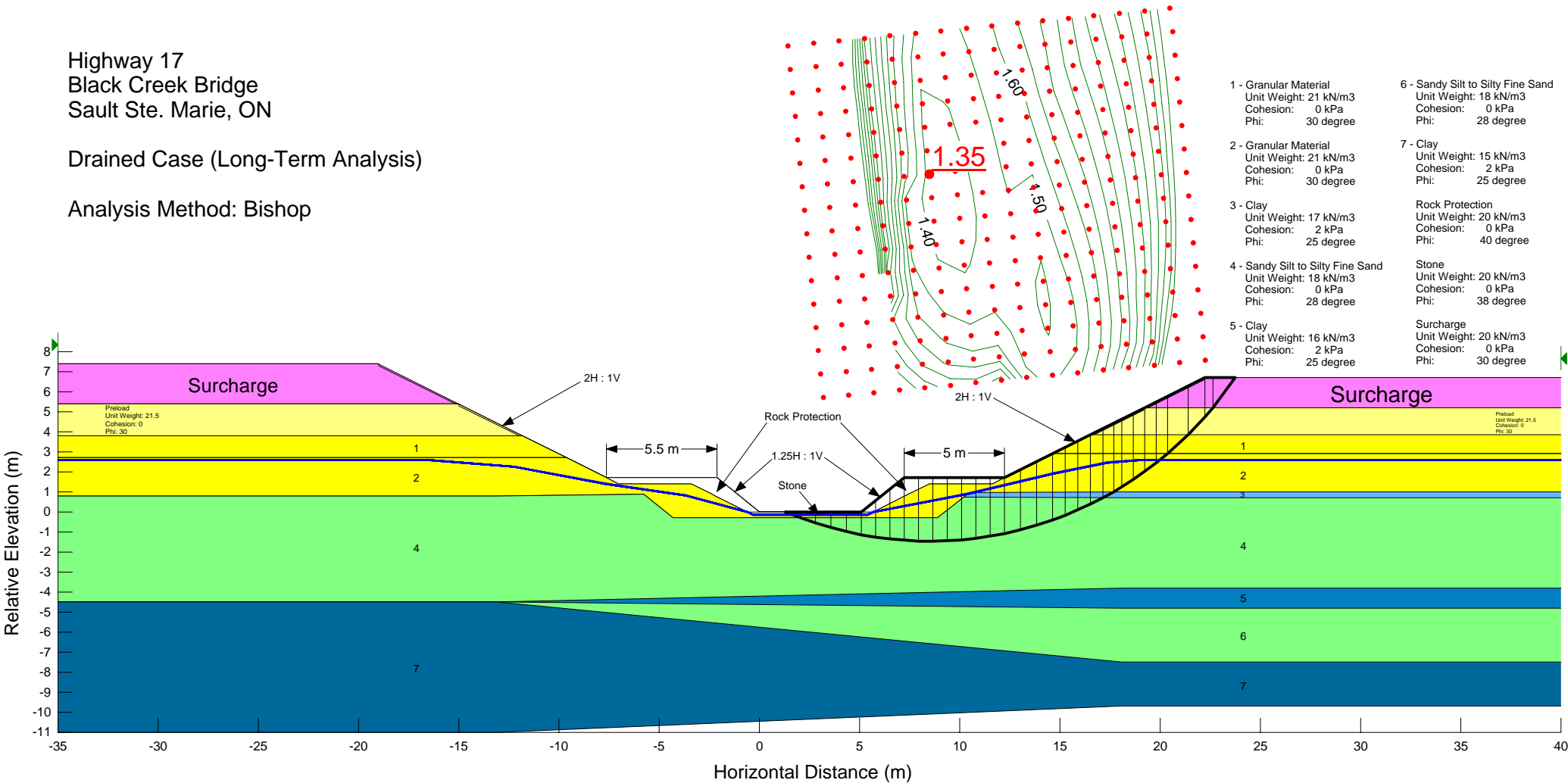


Figure J-5

Appendix K

Proposed EPS Profile

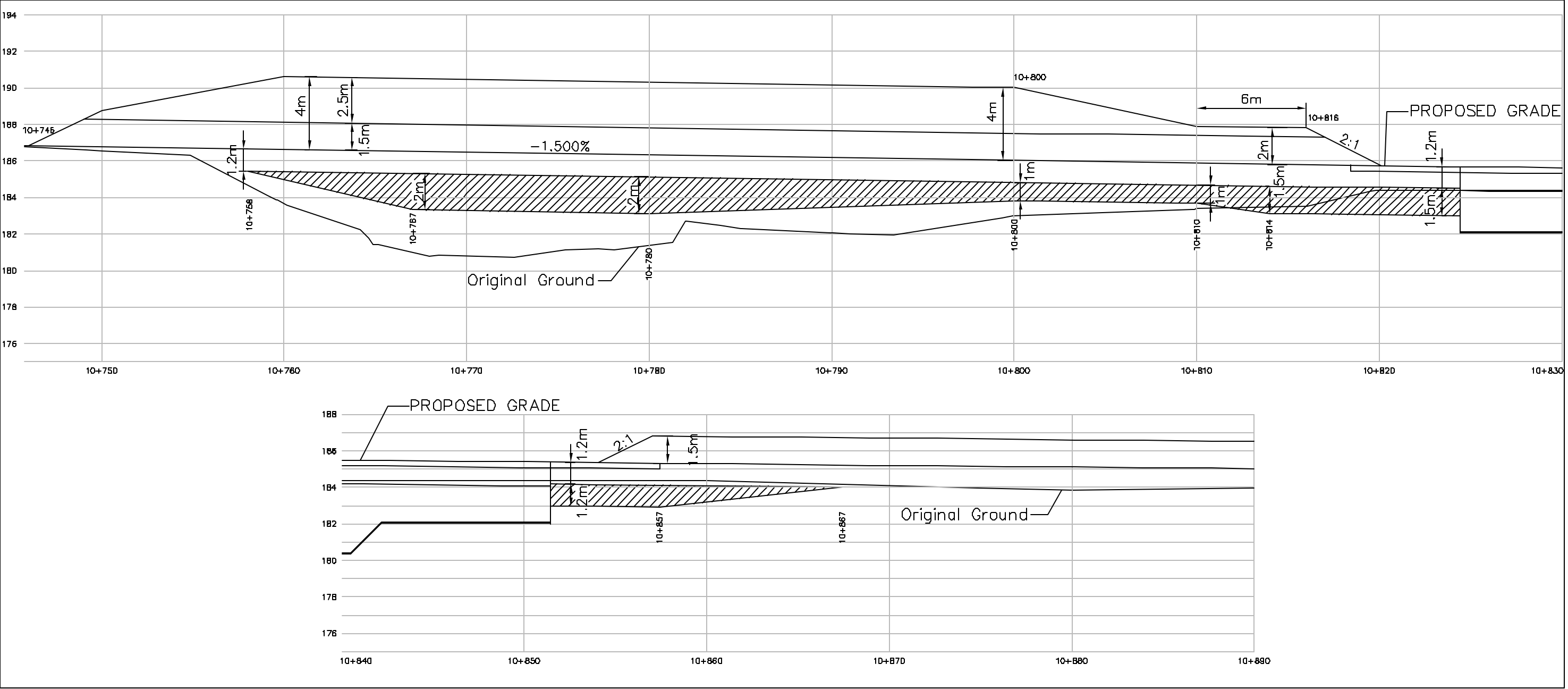


FIGURE K-1
PROPOSED EPS PLACEMENT PROFILE

NOTE: To the south of station 10+770, the thickness of EPS may be less on the right side, but may need to be increased on the left side, depending on the cross sections.

Legend

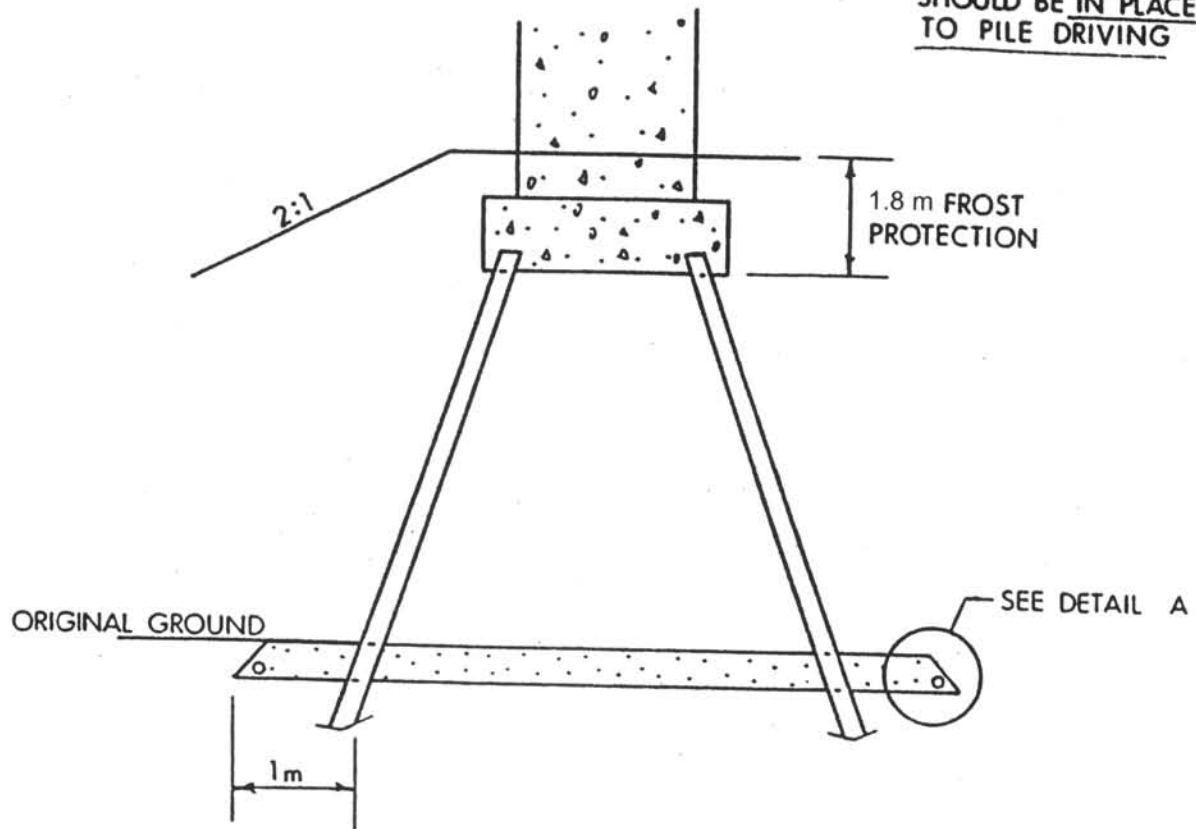
 Proposed EPS

N.T.S.

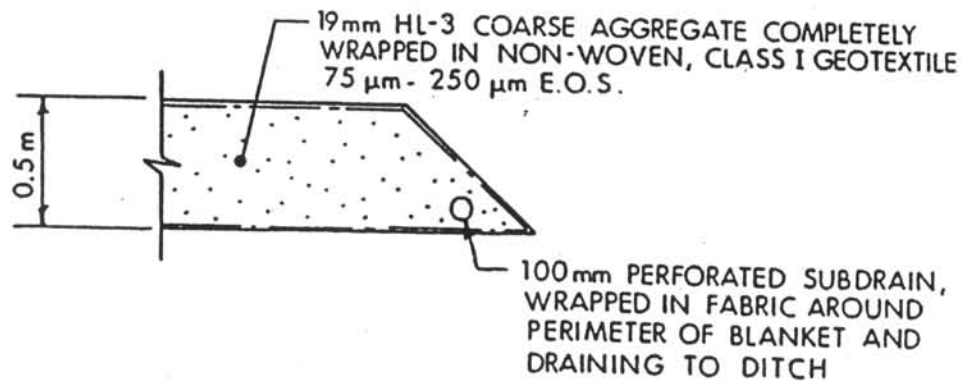
Appendix L

MTO Inverted Filter

NOTE : THE DRAINAGE BLANKETS
SHOULD BE IN PLACE PRIOR
TO PILE DRIVING



ABUTMENT SECTION (TYP)



DETAIL A

DRAINAGE BLANKET DETAILS FOR
ABUTMENTS AND PIERS

Appendix M

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Shaheen & Peaker Limited at the time of preparation. Unless otherwise agreed in writing by Shaheen & Peaker Limited, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Shaheen & Peaker Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.