FOUNDATION DESIGN REPORT
PROPOSED HIGHWAY 17 (NEW)
FROM ECHO RIVER TO BAR RIVER ROAD
DISTRICT 62, SAULT STE. MARIE, ONTARIO
G.W.P. 354 AND 352-94-00

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5. DISCUSSION AND RECOMMENDATIONS

5.1 SITE NO. 1 : HIGHWAY 17 (NEW) CUT SECTION BETWEEN STATIONS 10+210 AND 10+410 EASTBOUND LANES, AND BETWEEN STATIONS 10+210 AND 10+350 WESTBOUND LANES

5.1.1 EARTH CUT

5.1.1.1 Station 10+210 to 10+270 EBL

Between Station 10+210 and 10+270 EBL, the profile drawing indicates that up to about 7 m of overburden will be excavated to the subgrade level (considering about 1 m of pavement structure). The anticipated cut material consists of surficial topsoil, sand and gravel layer with occasional cobbles and boulders, and firm to very stiff clay.

From the cross-section in Drawing 1C, the cut slope to the right of the EBL could consist mainly of clay (as shown for Section 10+240) or a combination of rock cut with a clay wedge over the bedrock (as presented for Section 10+250). Slope stability analyses were conducted on a 5 m high cut slope consisting of clay on the right side of Station 10+240 and on a 6.5 m high cut slope consisting of clay wedge over sloping rock on the right side of Station 10+250. For the undrained (short-term) stability analyses, undrained shear strengths (c-values) of the clay were utilized based on the field vane tests results, assuming angle of internal friction (ϕ) of the soil being zero. For the drained (long-term) analyses, a residual ϕ-angle and a small value of shear strength (c) were used.

The analyses were performed using limit state equilibrium (Bishop's Simplified Method by the computer program Slope/W) and the following soil parameters were used:

Table 5.1.1.1 Soil Parameters Used in Slope Stability Analyses

Material Type	Short-Term Analysis			Long-Term Analysis		
	ϕ (degrees)	c (kPa)	γ (kN/m^3)	ϕ' (degrees)	c' (kPa)	γ (kN/m^3)
Sand and Gravel	35	0	21.5	35	0	21.5
Clay	0	35	16 –19	24	2 - 3	16 - 19

Due to the variability of the shear strengths of the clay and the presence of highly erodible silt seams/layers, the recommended minimum factor of safety for a stable slope in this cut area is 1.40. Typical cut slope stability sections are presented in Appendix E1.

Based on the above and our stability analysis, 3H : 1V side slopes on earth cut are considered stable. In this case, however, the groundwater is assumed to be at least 1.5 m below the surface of the cut slope by the provision of a toe drain, as indicated in Appendix E1. Due to the presence of the highly erodible silt seams or layers, erosion protection should be provided with the placement of an erosion control blanket.

In the case of a soil wedge over the bedrock, our analyses indicate that the soil wedge is not stable at normal 2H : 1V side slopes and therefore should be flattened. It is considered that 3H : 1V side slope is stable. It is anticipated that the underlying bedrock is sloping at a rate of about 3H : 1V and therefore, it is most likely that all overburden will have to be removed. Consideration should be given to the right of way limit; it is possible that this limit may have to be extended, or at least a temporary easement beyond the limit may have to be provided.

5.1.1.2 Station 10+220 to 10+350 WBL

Between Station 10+220 and 10+350 WBL, the profile drawing indicates that up to about 1.5 m of earth cut is anticipated. The cut material consists of surficial topsoil and peat, sand and gravel layer with some organics and occasional cobbles and boulders, and firm to very stiff clay.

Based on the height of cut (maximum 1.5 m high cut slope) and the anticipated cut materials, the cut slope is considered stable at normal 2H : 1V side slopes. In areas where thick organic soils (e.g., 0.7 m thick peat at Borehole 10+315 21m Lt), flatter side slopes of up to 4H : 1V may be required. Alternatively, these organic soils could be stripped and replaced with compacted inorganic soils, within the road right of way.

The groundwater table throughout much of the site along the WBL was near the existing ground surface and could be under artesian condition. This aspect should be taken into consideration when carrying out stripping and backfilling. If necessary, dewatering by gravity drainage and pumping from strategically positioned sumps and/or relief wells may be required to facilitate these tasks.

5.1.2 ROCK CUT

Between about Station 10+270 and 10+410 EBL, the anticipated subgrade is likely to be bedrock and the cut materials could consist of thin layer of topsoil, sand and gravel layer extending to a depth of about 0.7 m, clay and the underlying bedrock. In this area, the anticipated cut slope will consist of rock. The quartzite bedrock in this section is considered stable at steep cut slope of 0.25 H : 1 V, provided that the rock within the cut zone is not steeply jointed; or if steeply jointed, the joints are not are not dipping towards the highway. It is recommended that the exposed rock should be inspected by a Rock Engineer during construction to assess its stability and provide any corrective measures, if required. It should be ensured that no rocks are overhanging along the steep slope. If this rock was found to be fractured and/or unfavourably jointed or unstable, corrective measures such as grouting and/or rock anchors should be implemented.

If a Rock Engineer or Geologist will not be utilized to inspect the exposed rock, it is recommended that grouting of joints with dowels or rock bolts should be provided in the Contract Documents through an NSSP. Any rock bolt design and/or grouting should be illustrated on the contract drawings in addition to including specifications in the contract documents. Alternatively, the rock could be cut at a flatter slope such as 0.5 H : 1 V and/or clear zone could be increased.

5.1.3 CONSTRUCTION

Considering that the groundwater table is generally high at this site, groundwater will most likely be encountered during the excavation. Groundwater seepage may have to be controlled especially within the pervious sand and gravel layer and possibly along silt seams or layers within the clay deposit. Groundwater seepage in the surficial sand gravel could be controlled by interceptor trench from the top of the slope or strategically located filtered sumps and/or trench around the perimeter of and within the excavation.

Since the groundwater table is high, the cut materials are expected to be generally wet and are therefore not suitable for re-use, unless the materials are dried off prior to re-use as fills.

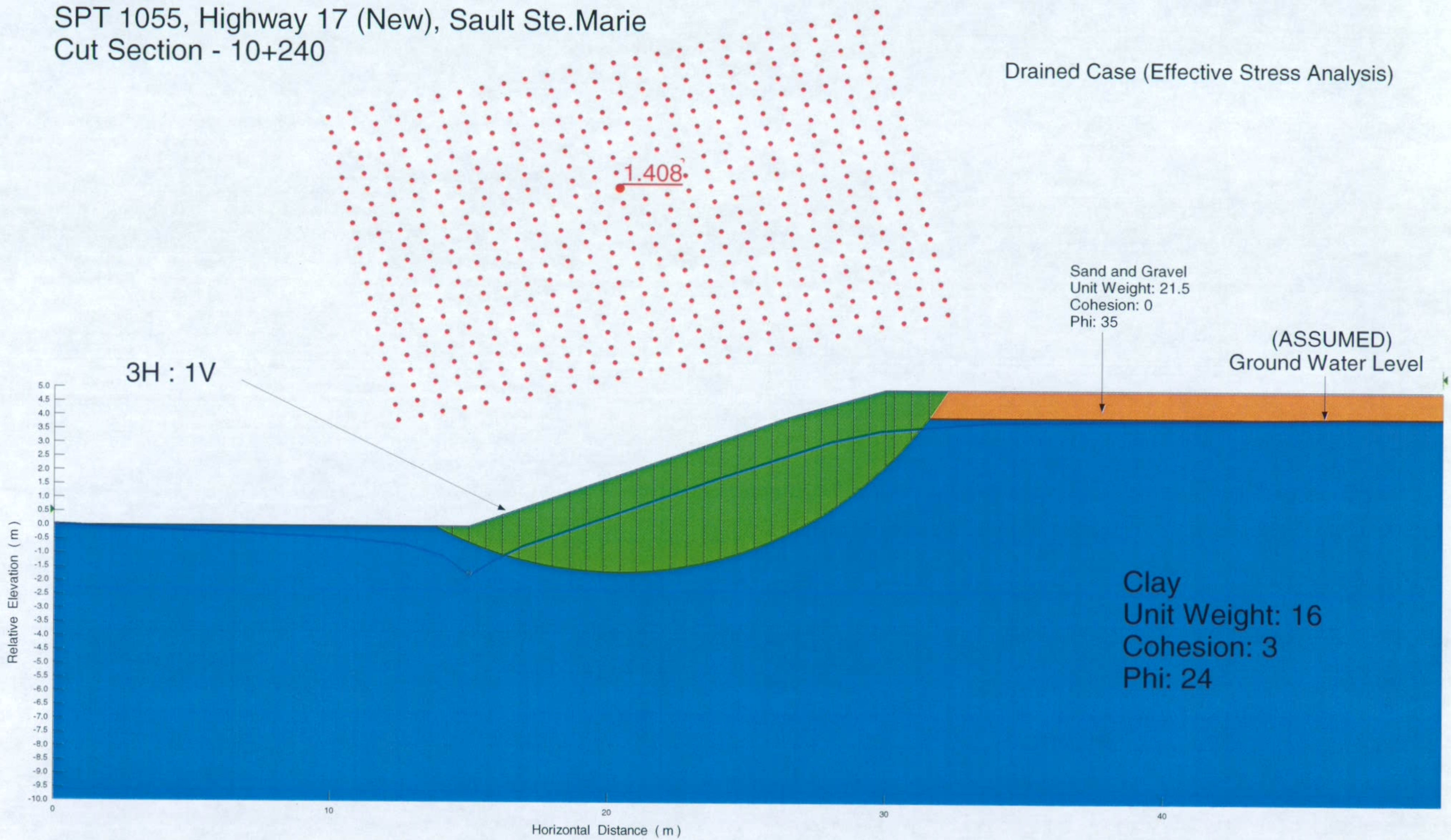
Blasting of the rock should be carefully controlled so as not to produce an overbreak which could cause instability of the rock face. As mentioned before, the stability of the rock should be assessed by an experienced Rock Engineer during construction.

APPENDIX E1

Slope Stability Analysis Results

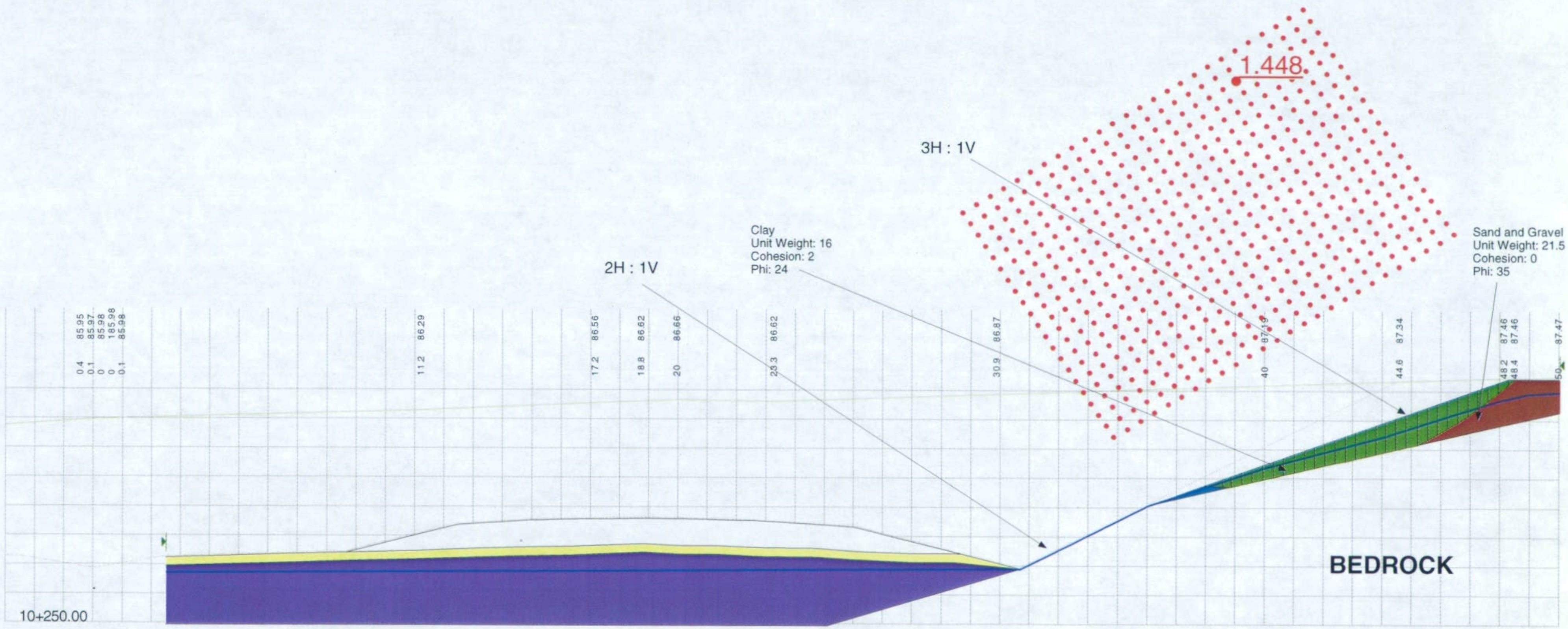
SPT 1055, Highway 17 (New), Sault Ste.Marie
Cut Section - 10+240

Drained Case (Effective Stress Analysis)



SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 10+250, Cut Section
Drained Case (Effective Stress Analysis)

Subsurface Conditions: Inferred from Borehole 10+255 Rt
and Borehole 10+255 CL



5.2 SITE NO. 2 : HIGHWAY 17 (NEW) CUT SECTION BETWEEN STATIONS 10+670 AND 10+825 EASTBOUND LANES, AND BETWEEN STATIONS 10+700 AND 10+780 WESTBOUND LANES

5.2.1 EARTH CUT

5.2.1.1 Station 10+740 to 10+825 EBL

Between about Stations 10 +740 and 10+825 EBL, the profile drawing indicates that up to about 8 to 10 m of earth cut is anticipated along the centerline of the road, but up to about 10 to 12 m of cut could be expected to the right of the road. The cut material consists of surficial topsoil, sand and gravel and/or silty sand till with cobbles and boulders, silty clay to clay and bedrock at some locations.

From the cross-section in Drawing 2D, the cut slope to the right of the EBL could consist mainly of very dense sand and gravel, cobbles and boulders and silty sand till (as shown for Section 10+790) or could consist of combination of rock cut with a silty sand till or sand and gravel wedge over the bedrock (as shown for Section 10+750).

Slope stability analyses were conducted on a 10 m high cut slope consisting of sand and gravel / silty sand till with cobbles and boulders on the right side of Station 10+790, and on a 12 m high cut slope consisting of sand and gravel/cobbles and boulders wedge over sloping rock on the right side of Station 10+750. The bedrock surface is estimated to be at 3H : 1V slope, dipping to the left.

The analyses were performed using limit state equilibrium (Bishop's Simplified Method by the computer program Slope/W) and the following soil parameters were used:

Table 5.2.1.1 Soil Parameters Used in Slope Stability Analyses

Material Type	Short-Term Analysis			Long-Term Analysis		
	ϕ (degrees)	c (kPa)	γ (kN/m^3)	ϕ' (degrees)	c' (kPa)	γ (kN/m^3)
Sand and Gravel or Silty Sand Till	35	0	21.5	35	0	21.5

Due to the height of cut, the recommended minimum factor of safety for a stable slope in this cut area is 1.40. Typical cut slope stability sections are presented in Appendix E2.

Based on the above and our stability analysis, 2H : 1V side slopes on earth cut with a 2 m wide mid-height bench are considered stable. In this case, the groundwater is estimated to be about 4 m below existing grade. Since groundwater table is within the cut slope consisting of pervious materials, it is recommended that a 0.5 m thick rip-rap be provided for erosion protection. The rip-rap (0.5 m thick) should be underlain with a suitable geotextile (e.g., Class II, non-woven with FOS 75 to 150 μm).

In the case of a soil wedge over the bedrock, our slope stability analysis shows that the soil wedge is stable at 2.5H : 1V side slope. As mentioned before, the slope of the bedrock is probably dipping at 3H : 1V, and therefore, some of the overburden could be removed.

It is anticipated that the clay lense within this cut area will likely thin out to nothing along the proposed cut slope to the right of the EBL. Analysis with clay layer was therefore not considered.

Cut slope higher than 6 m should have an at least 2 m wide mid-height bench to control drainage and to improve surficial stability.

5.2.1.2 Station 10+700 to 10+780 WBL

Between Station 10+700 and 10+780 WBL, the profile drawings indicate that up to about 4 m of earth cut is anticipated along the centerline of the road, but up to about 3 m of cut could be expected on the left side of the road. The cut material consists of surficial topsoil, sand and gravel and/or silty sand till with cobbles and boulders.

Similarly, based on the above and our analysis, a 2H : 1V side slopes on earth cut are considered stable on the left side of the WBL.

The groundwater table throughout much of the site along the WBL was near the existing ground. This aspect should be taken into consideration when carrying out stripping and backfilling. If necessary, dewatering by gravity drainage and pumping from strategically positioned sumps and/or relief wells may be required to facilitate these tasks.

5.2.2 ROCK CUT

Between Station 10 +670 and 10+740 EBL, the profile drawing indicates that up to about 7.5 m of rock will be excavated to the subgrade level (considering about 0.43 m of pavement structure). Along the right side of the EBL, up to about 12 m of rock cut is anticipated. The cut material consists of surficial topsoil, sand and gravel and/or silty sand till with cobbles and boulders, and bedrock.

In this area, the anticipated cut slope will consist of rock. The quartzite bedrock in this section is considered stable at steep cut slope of 0.25H:1V, provided that the rock within the cut zone is not steeply jointed; or if steeply jointed, the joints are not are not dipping towards the highway. It is recommended that the exposed rock should be inspected by a Rock Engineer during construction to assess its stability and provide any corrective measures, if required. It should be ensured that no rocks are overhanging along the steep slope. If this rock was found to be fractured and/or unfavourably jointed or unstable, corrective measures such as grouting and/or rock anchors should be implemented.

If a Rock Engineer or Geologist will not be utilized to inspect the exposed rock, it is recommended that grouting of joints with dowels or rock bolts should be provided in the Contract Documents through an NSSP. Any rock bolt design and/or grouting should be illustrated on the contract drawings in addition to including specifications in the contract documents. Alternatively, the rock could be cut at a flatter slope such as 0.5 H : 1 V and/or clear zone could be increased.

5.2.3 CONSTRUCTION

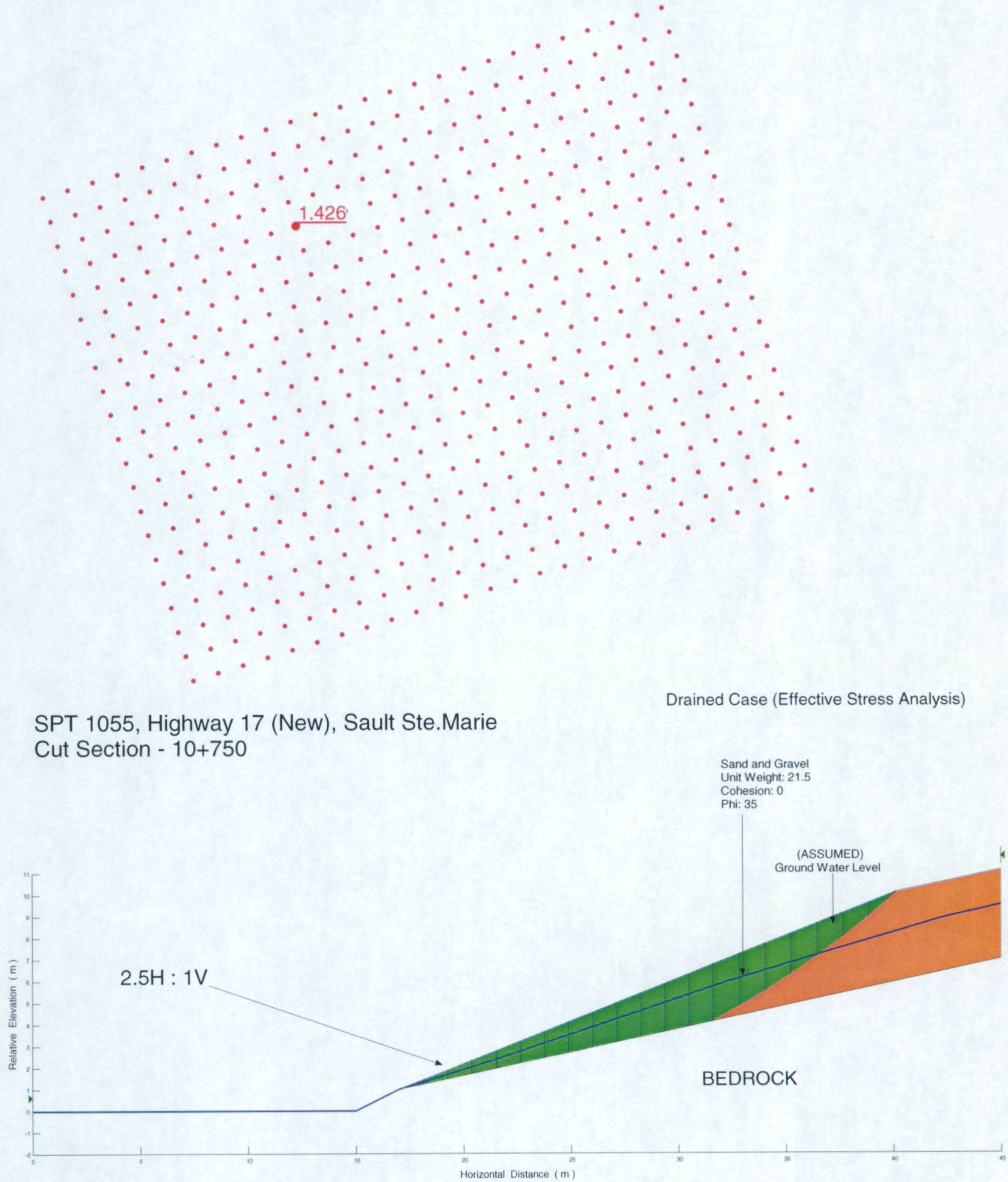
Considering that the groundwater table is relatively high at this site, groundwater will most likely be encountered during the excavation. Groundwater seepage may have to be controlled especially within the pervious sand and gravel layer and possibly along silt seams or layers within the clay deposit. Groundwater seepage in the surficial sand gravel could be controlled by interceptor trench from the top of the slope or strategically located filtered sumps and/or trench around the perimeter of and within the excavation.

In this case, the cut materials are expected to be generally wet and are therefore not suitable for re-use, unless the materials are dried off prior to re-use as fills.

Blasting of the rock should be carefully controlled so as not to produce an overbreak which could cause instability of the rock face. As mentioned before, the stability of the rock should be assessed by an experienced Rock Engineer during construction.

APPENDIX E2

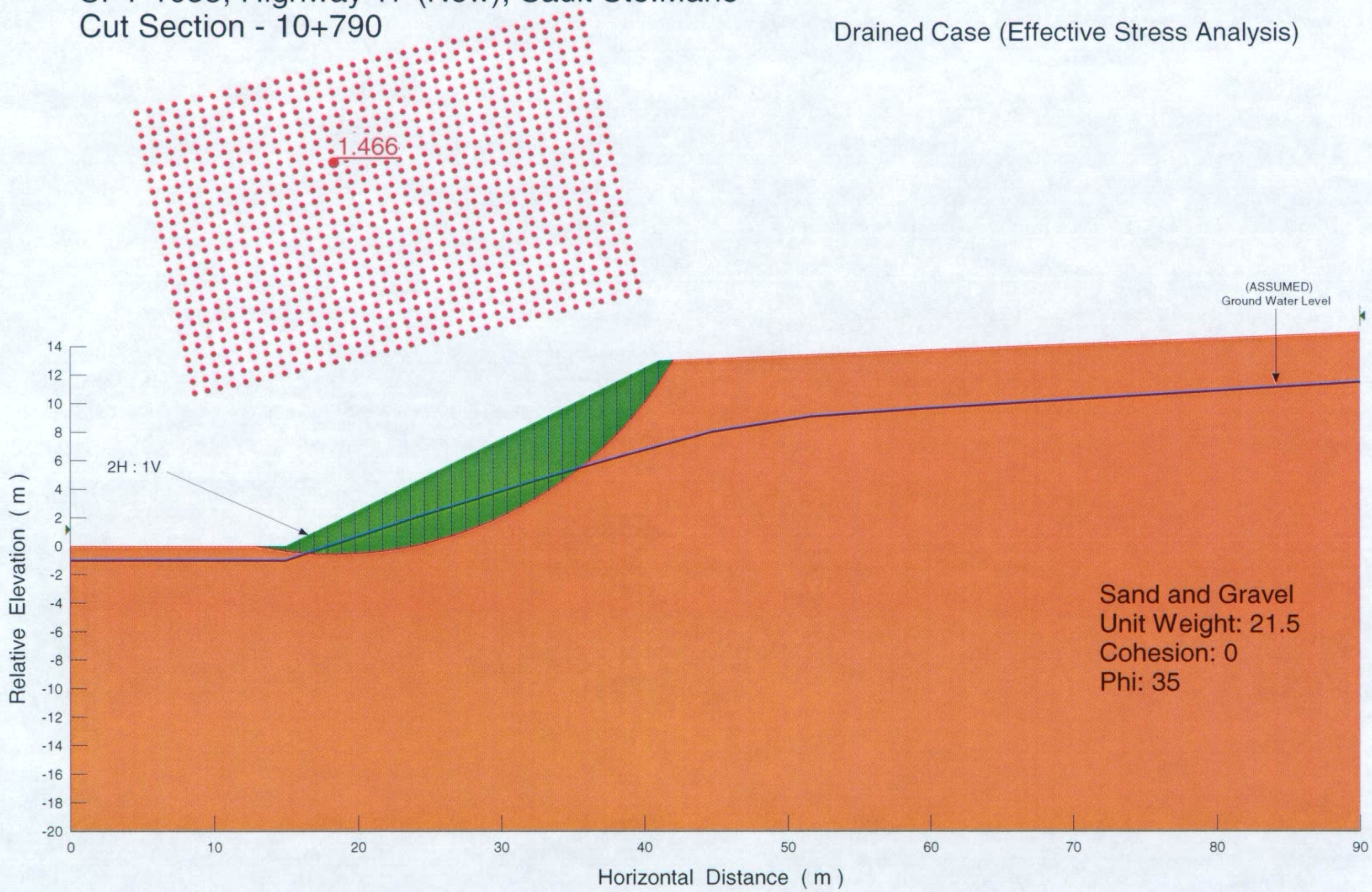
Slope Stability Analysis Results



1.426
Drained Case (Effective Stress Analysis)
SPT 1055, Highway 17 (New), Sault Ste.Marie
Cut Section - 10+750
Sand and Gravel
Unit Weight: 21.5
Cohesion: 0
Phi: 35
(ASSUMED)
Ground Water Level
2.5H : 1V
BEDROCK
Relative Elevation (m)
Horizontal Distance (m)

SPT 1055, Highway 17 (New), Sault Ste.Marie
Cut Section - 10+790

Drained Case (Effective Stress Analysis)



5.3 SITE NO. 3 : HIGHWAY 17 (NEW) FILL SECTION BETWEEN STATIONS 10+825 AND 11+000 EASTBOUND LANES, AND BETWEEN STATIONS 10+780 AND 11+000 WESTBOUND LANES (BOREHOLES 10+840 Lt, 10+860 CL, 10+885 Lt, 10+900 CL, 10+923 Rt, 10+940 CL and 10+980 Rt)

Seven boreholes explored this fill area and these show that below some topsoil of up to 0.3 m in thickness, the site is underlain by a deep deposit of clay to at least 10 m below existing ground surface. The upper 1.5 $\pm$ m of the clay deposit is considered stiff to very stiff with undrained shear strengths of 58 to greater than 100 kPa, and then soft to firm below this depth with shear strengths between 18 and 49 kPa. A DCPT was conducted in Borehole 10+900 CL to a refusal depth of about 29 m and this inferred that a somewhat "stiffer" layer at a depth of approximately 24 m, and a hard or very dense stratum at a depth of about 28 m below existing grades. From the colour of the soil and from site observations, the groundwater table is believed to be close to the ground surface.

5.3.1 EMBANKMENT STABILITY

Based on the profile drawings, the grades along this section of the proposed Highway 17 (New) are expected to be raised to a maximum height of about 2.5 m along EBL and 3.5 m along WBL. The existing grades are rising towards the south (towards increasing stations) and the required fill is decreasing in height towards this direction.

Embankment stability analyses were performed using limit state equilibrium (Bishop's Simplified Method by the computer program Slope/W). For the undrained (short-term) stability analyses, undrained shear strengths (c-values) of the clay were utilized based on the Field Vane Test results, and assuming angle of internal friction (ϕ) of the soil being zero. No correction factor to the Field Vane tests was applied. Because of this and due to the variable strength characteristics and thickness of the clay deposit, a minimum factor of safety of 1.40 was considered as being necessary.

Drained (long-term) analyses were also conducted at some locations, and these indicated stable conditions.

The following soil parameters were used for the stability analyses:

Table 5.3.1 Soil Parameters Used in Slope Stability Analyses

Material Type	Short-Term Analysis			Long-Term Analysis		
	ϕ (degrees)	c (kPa)	γ (kN/m^3)	ϕ' (degrees)	c' (kPa)	γ (kN/m^3)
Embankment Fill (Select Subgrade Material)	30	0	21.5	30	0	21.5
Embankment Fill (Rock Fill)	43	0	18.0-20.0	43	0	18.0-20.0
Sand Backfill (used to replace existing peat and other surficial unsuitable soils)	30	0	20.0	30	0	20.0
Rock Backfill (used to replace existing peat and other surficial unsuitable soils)	43	0	18.0-20.0	43	0	18.0-20.0
Clay	0	18-80	15.0-17.0	24	2-3	15.0-17.0

Typical embankment slope stability sections and summary of the calculations are presented in Appendix E3.

Based on the above and our analyses, no foundation failures are anticipated for earthfill embankments up to 2.5 to 3.5 m high and with normal 2H : 1V side slopes, provided that all organic, weak or otherwise unsuitable materials are removed as per MTO standards before placing the fill, and that the fill consists of properly compacted, acceptable inorganic material (e.g., non frost susceptible soils or SSM). If necessary and to keep the embankment slopes uniform across this area, the slopes could be flattened to 4H : 1V using inorganic unsuitable soils (e.g., wet and weak soils).

Alternatively, if available, embankments could be constructed using rockfill. Based on our stability analysis, 1.25H : 1V side slopes, as per MTO standard procedures, would be stable. Similarly, If necessary and to keep the embankment slopes uniform across this area, the slopes could be flattened to 4H : 1V using any inorganic unsuitable earthfill.

5.3.2 SETTLEMENT OF EMBANKMENTS

From the profile drawings, the maximum height of fill along the EBL is about 2.5 m, while along the WBL, the maximum height is about 3.5 m. The actual heights of fills along at stations along the borehole locations are tabulated and are presented in Table 5.3.2 below.

Table 5.3.2.1 Fill Heights at Station Centreline

Station Number	Proposed Embankment Height (m) **WBL**	Proposed Embankment Height (m) **EBL**
10+860	3.41	2.17
10+885	3.42	2.40
10+900	3.36	2.43
10+923	3.06	2.21
10+940	2.77	2.10

Our analyses show that the expected settlements of the founding clay under the weight of the proposed embankments are in the range of 200 to 700 mm. These settlements are in addition to the settlement of the embankment fills, which are estimated not to exceed 25 to 35 mm.

The highest settlement estimate values were along Station 10+860 and 10+885 WBL where the height of fill is at its maximum (3.4 m). It is anticipated that these settlements will probably take place over a period of five to more than twenty years.

To expedite the settlements, we recommend surcharging. Our stability analyses indicate that the maximum height of surcharge that can be placed over the embankment is 1.7 m. In this case, a surcharge period of 2 years or longer is required. The time-settlement curves in the area of Station 10+860 and 10+885 WBL , using earthfill with 2H:1V and 4H:1V side slopes, are presented in Appendix F3.

Surcharging should start along the EBL (with lower embankment height) at Station 10+825 and gradually increase to 1.7 m at Station 10+840. It should remain at this height to Station 10+960, gradually reducing to zero at Station 10+970. Along the WBL, surcharging should start at Station 10+800 and gradually increase to 1.7 m at Station 10+810 and remain at this level to Station 10+960, and gradually reducing to zero at Station 10+970.

With this approach, the anticipated settlements after 2 years of surcharging are reduced to a maximum of about 300 mm along the WBL. After the surcharge is removed (after 2 years), about 40 mm of settlements are expected to occur within the next two years along the WBL. The remaining settlements are expected to occur gradually to about 170 mm over a period of 15 years (from year 4 to year 19 from start of construction), and a further 90 mm thereafter. Alternatively, wick drains could be considered if faster consolidation period is desired.

In any event, surcharging should be carried out with proper instrumentation for field monitoring. It is furthermore recommended that the surcharge be placed gradually (i.e., preferably at least 3 layers, starting from one end of the site and proceeding to the other end), to allow excess pore pressures to dissipate.

Fills at the above section should be provided with a widened cross-section to allow for settlements of the underlying soils and a future grade raise. In this case, we recommend that the road platform should be widened by at least 3 m on each side of the centreline (total of 6 m). This is also in accordance with the Northern Region Engineering Directive NRE 98-200.

5.3.3 CONSTRUCTION

For embankment construction, all organic and other unsuitable soils should be removed within an envelope given by an imaginary slope no steeper than 1:1 from the toe of the proposed embankment. Based on the available borehole data, the average values of the unsuitable soils to be stripped can be expected to be variable but for preliminary estimating purposes, it can be assumed to be about 0.3 m on the average.

After stripping, the exposed subgrade should be inspected, approved and properly compacted from the surface, using a suitable compactor. This should, however be done at the discretion of the Quality Verification Engineer, as it may be feasible to effect surface compaction due to site conditions such as high water table or soft clay. If necessary, high water table may necessitate some dewatering by pumping from open sumps in low-lying areas in order to achieve proper compaction of earth fills. In addition, the first one to two lifts of the fill may consist of granular materials and may need to be thicker than normal (i.e., thicker than 300 mm lifts) to be able to compact the fill.

The materials used for the construction of the embankment should consist of approved, acceptable earth or rock fill. The earth fills should be placed in lifts not exceeding 300 mm before compaction and each lift of earth fill should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density (SPMDD) (OPSS 206 and OPSS 501).

In case of rockfill, the rockfill should be placed in lifts with thickness not exceeding 1.5 m. The rockfill should be compacted by overlapping track prints of the construction equipment. Depending on the size and type of equipment used, six to eight passes along each path should be required. The surface voids of each layer of rock fill materials should be filled with fragments of rock before the next layer is deposited. The final surface of rock fill material should be compacted by at least two additional passes and should be blinded with compacted fine fill material (or chinked) prior to installation of the road subbase layer.

Rock fill material of nominal size of 400 mm could be used and should consist of pieces of hard and durable rock with no sign of decomposition. Concrete, masonry, brick and similar materials should not be used.

A geotextile separator is recommended between rockfill and any native soil surcharge in order to prevent infiltration of fine soils into the rockfill. Any silty soils left at the surface of the rockfill, after the surcharge is removed, could be potential cause of possible frost heaving of the pavement. The separator should comprise of a Class II non-woven geotextile as per OPSS 1860 with a Filtration Opening Size (FOS) of 50 to 100 μm.

As the removal of the rockfill after the surcharging period may not be practical, the use of granular subbase material may be preferable (e.g., Granular 'B') rather than rockfill for the surcharge materials. In this instance, in order to reduce the quantities of the granular subbase material to be used, a minimum settlement of 300 mm can be assumed between Station 10+840 and 10+940 along the WBL. Along the EBL, a minimum of 200 mm can be assumed from Station 10+850 to 10+950. These values could be reduced by half beyond these limits.

The groundwater table throughout much of the site was near the existing ground surface. This aspect should be taken into consideration when carrying out stripping and backfilling. If necessary, dewatering by gravity drainage and pumping from strategically positioned sumps may be required to facilitate these tasks.

Proper erosion control measures should be implemented both during construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

APPENDIX E3

Slope Stability Analysis Results

Appendix E3

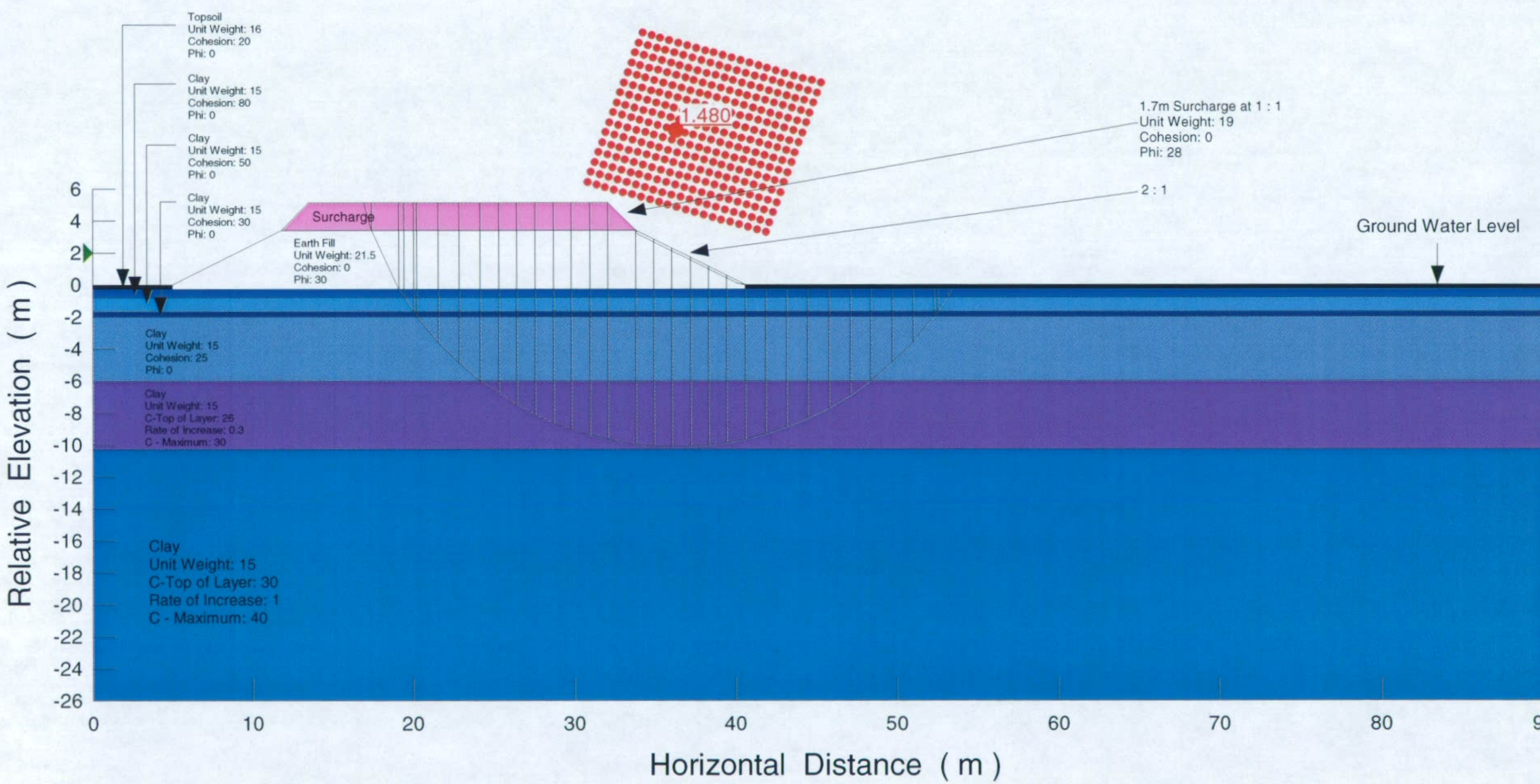
Calculated Minimum Safety Factors – Short-Term Analysis

Borehole Number	Proposed Embankment Height (m) **WBL**	Additional Surcharge Height (m)	Total Embankment Height with Surcharge (m)	Embankment Material	Side Slopes	Minimum Safety Factor
10+860	3.41	1.7	5.1	Rock Fill Rock Fill + Earth Fill Earth Fill Earth Fill	1¼ : 1 1¼ : 1 + 4 : 1* 2 : 1 4 : 1	1.52 1.65 1.45 1.54
10+885	3.42	1.7	5.1	Rock Fill Rock Fill + Earth Fill Earth Fill Earth Fill	1¼ : 1 1¼ : 1 + 4 : 1* 2 : 1 4 : 1	1.64 1.72 1.56 1.62
10+900	3.36	1.7	5.1	Rock Fill Rock Fill + Earth Fill Earth Fill Earth Fill	1¼ : 1 1¼ : 1 + 4 : 1* 2 : 1 4 : 1	1.55 1.66 1.48 1.55
10+923	3.06	1.7	4.8	Rock Fill Rock Fill + Earth Fill Earth Fill Earth Fill	1¼ : 1 1¼ : 1 + 4 : 1* 2 : 1 4 : 1	1.63 1.67 1.54 1.57
10+940	2.77	1.7	4.5	Rock Fill Rock Fill + Earth Fill Earth Fill Earth Fill	1¼ : 1 1¼ : 1 + 4 : 1* 2 : 1 4 : 1	1.78 1.80 1.68 1.70

* 1¼ : 1 side slopes built of rock fill and then flattened to 4 : 1 side slopes using earth fill prior to placing the surcharge or 4 : 1 rock fill.

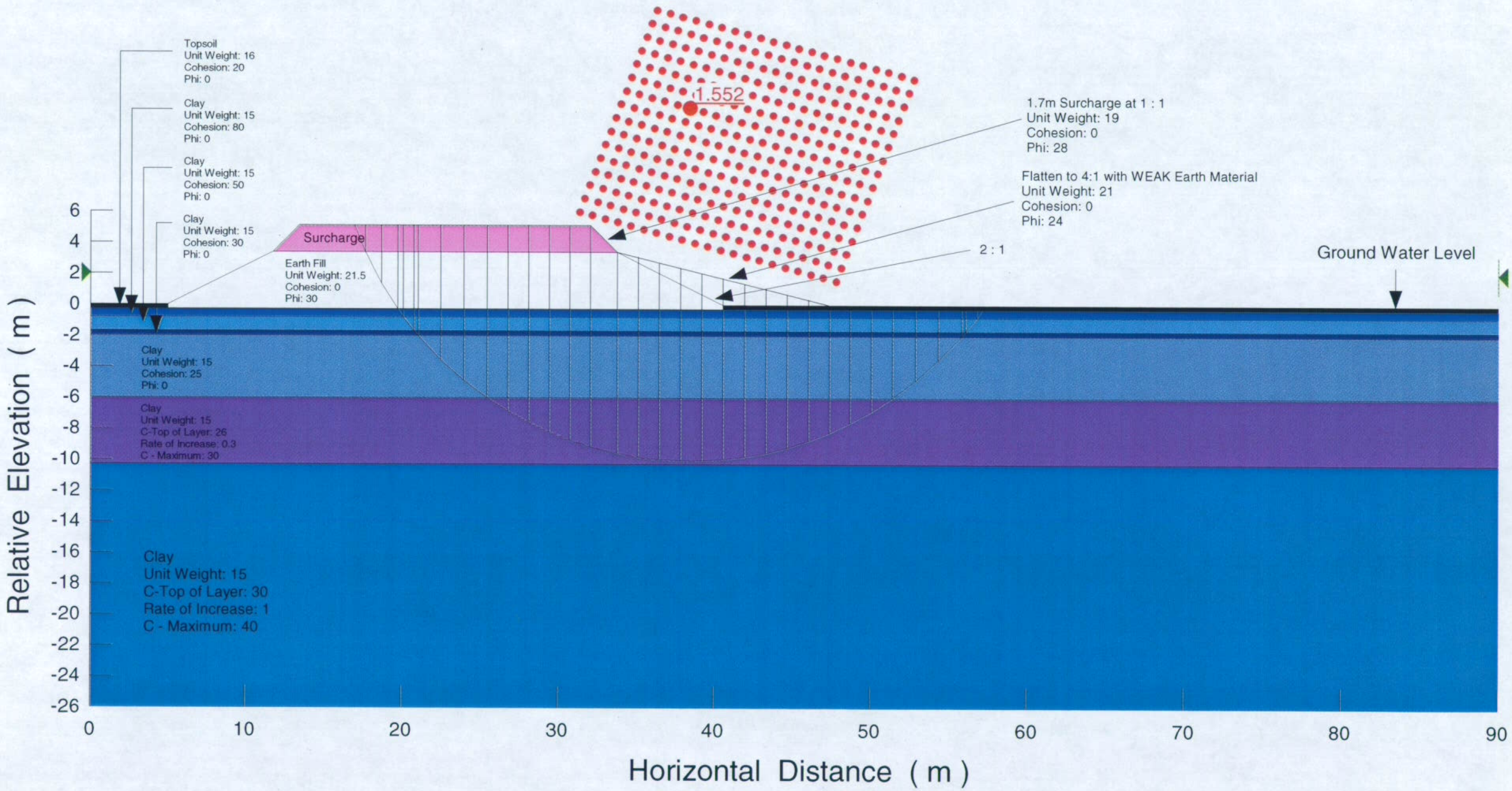
SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 10+900, 3.4m High, Earth Fill Embankment (Plus 1.7m Surcharge)
Undrained Case (Total Stress Analysis)

Platform Width = 22m



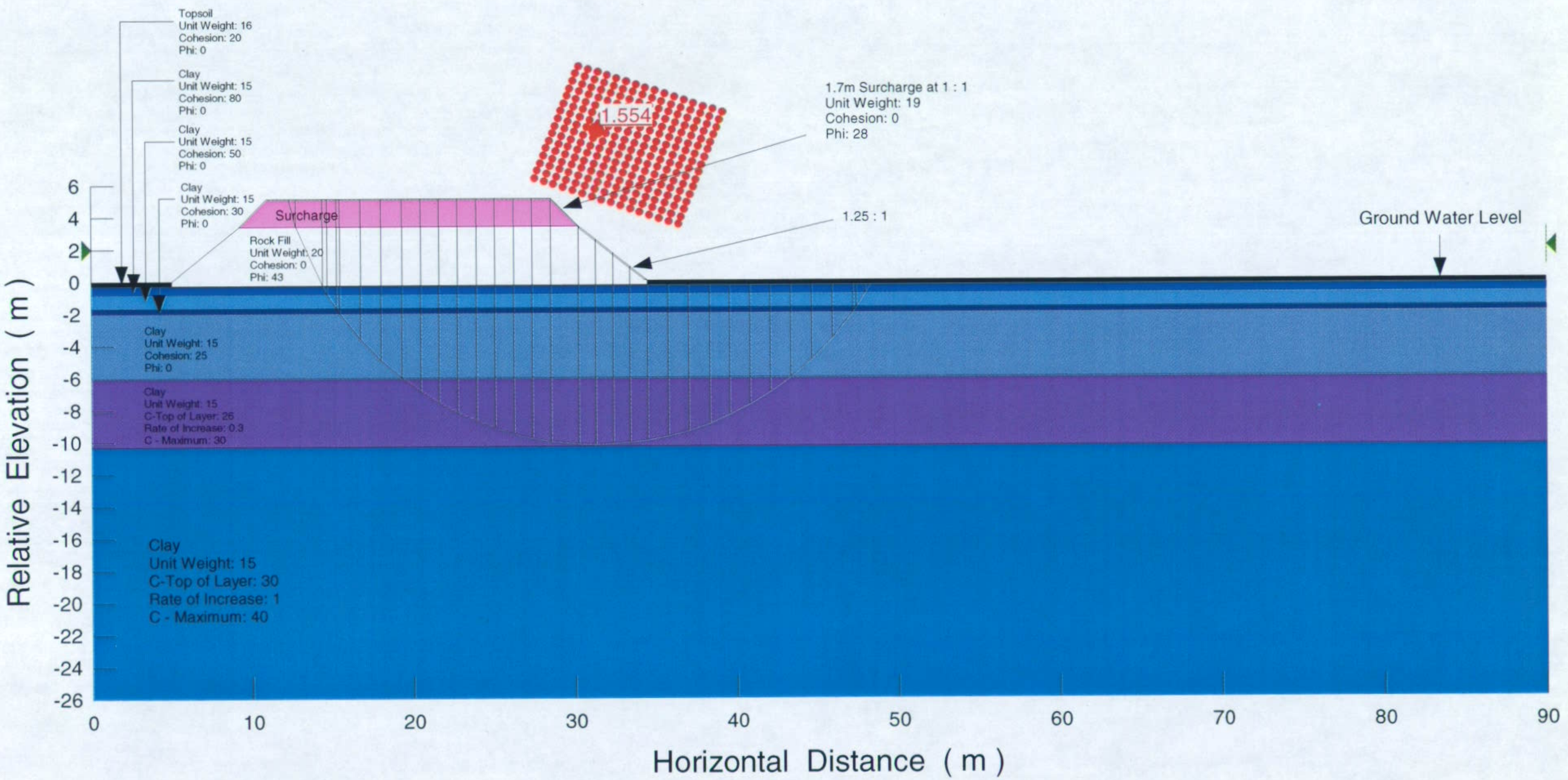
SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 10+900, 3.4m High, Earth Fill Embankment (Plus 1.7m Surcharge)
Undrained Case (Total Stress Analysis)

Platform Width = 22m



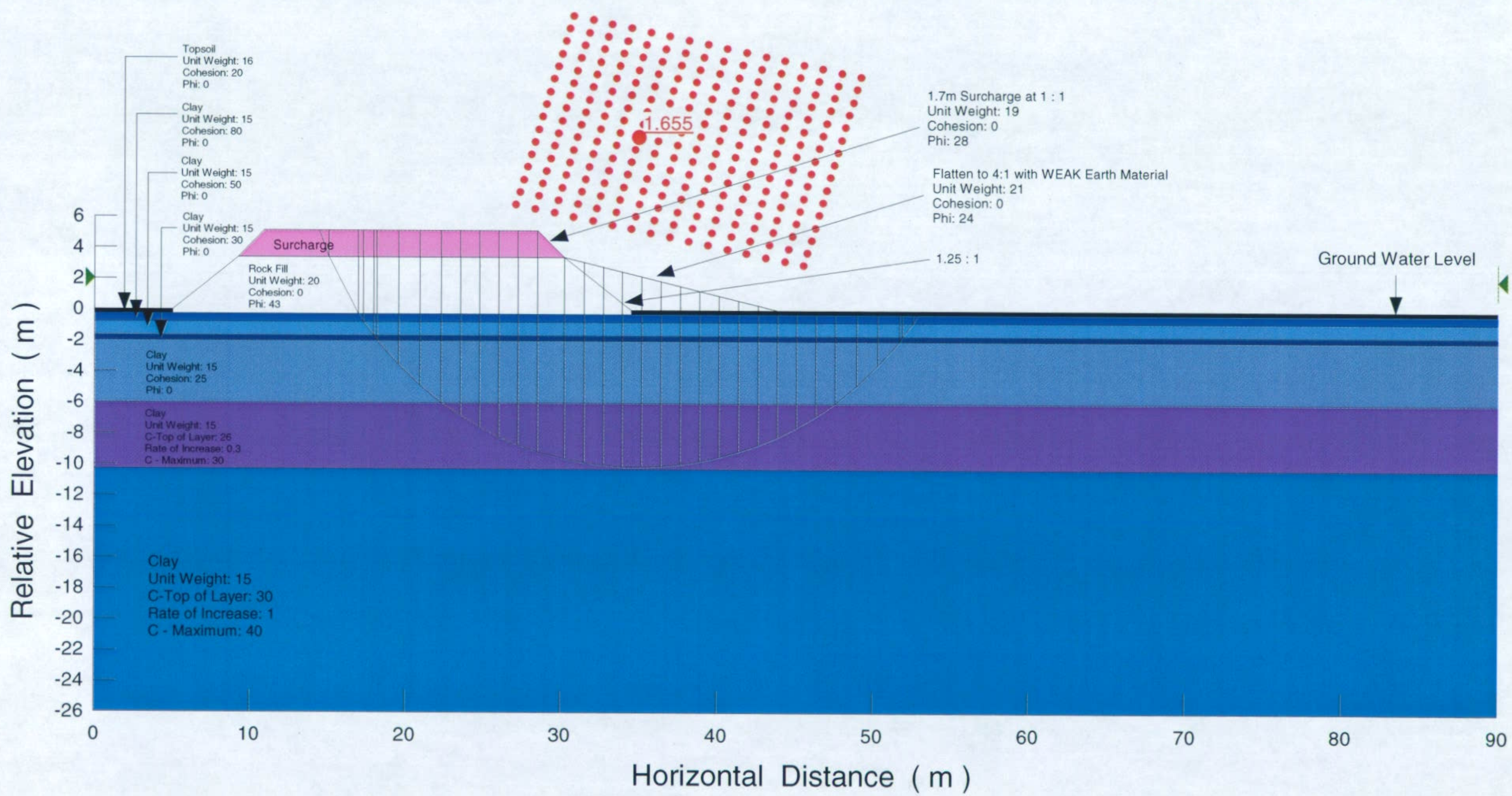
SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 10+900, 3.4m High, Rock Fill Embankment (Plus 1.7m Surcharge)
Undrained Case (Total Stress Analysis)

Platform Width = 21m

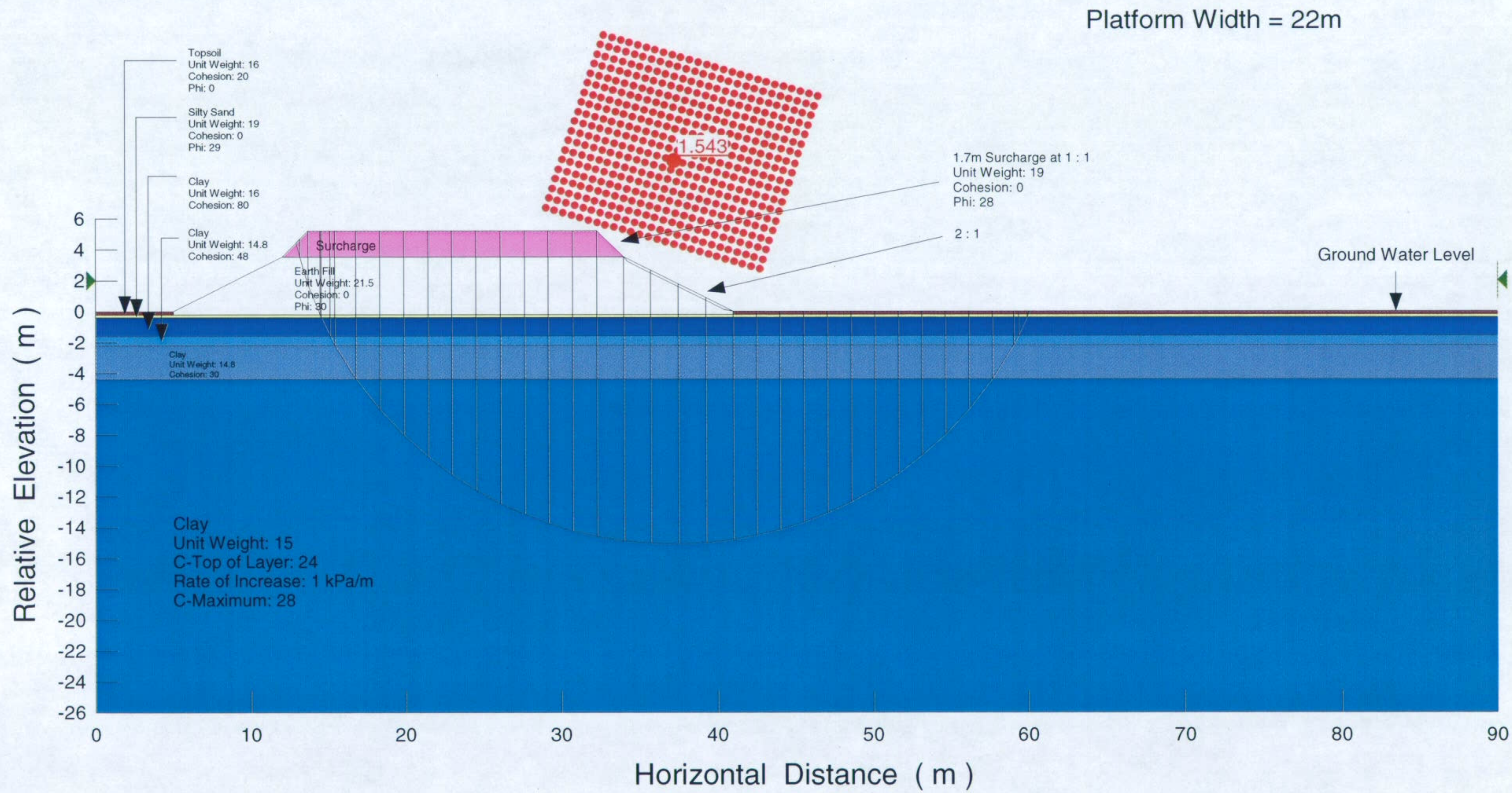


SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 10+900, 3.4m High, Rock Fill Embankment (Plus 1.7m Surcharge)
Undrained Case (Total Stress Analysis)

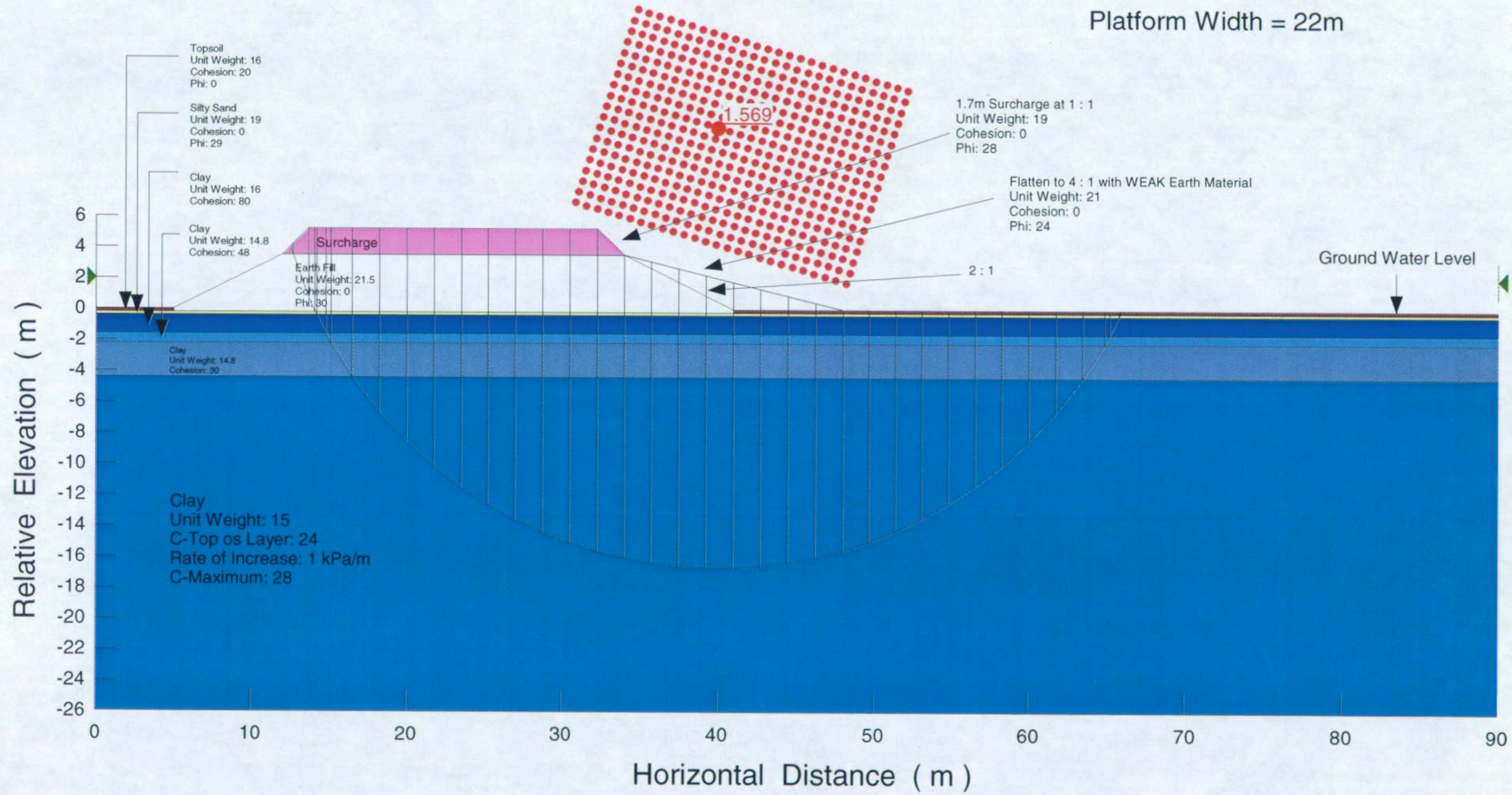
Platform Width = 21m



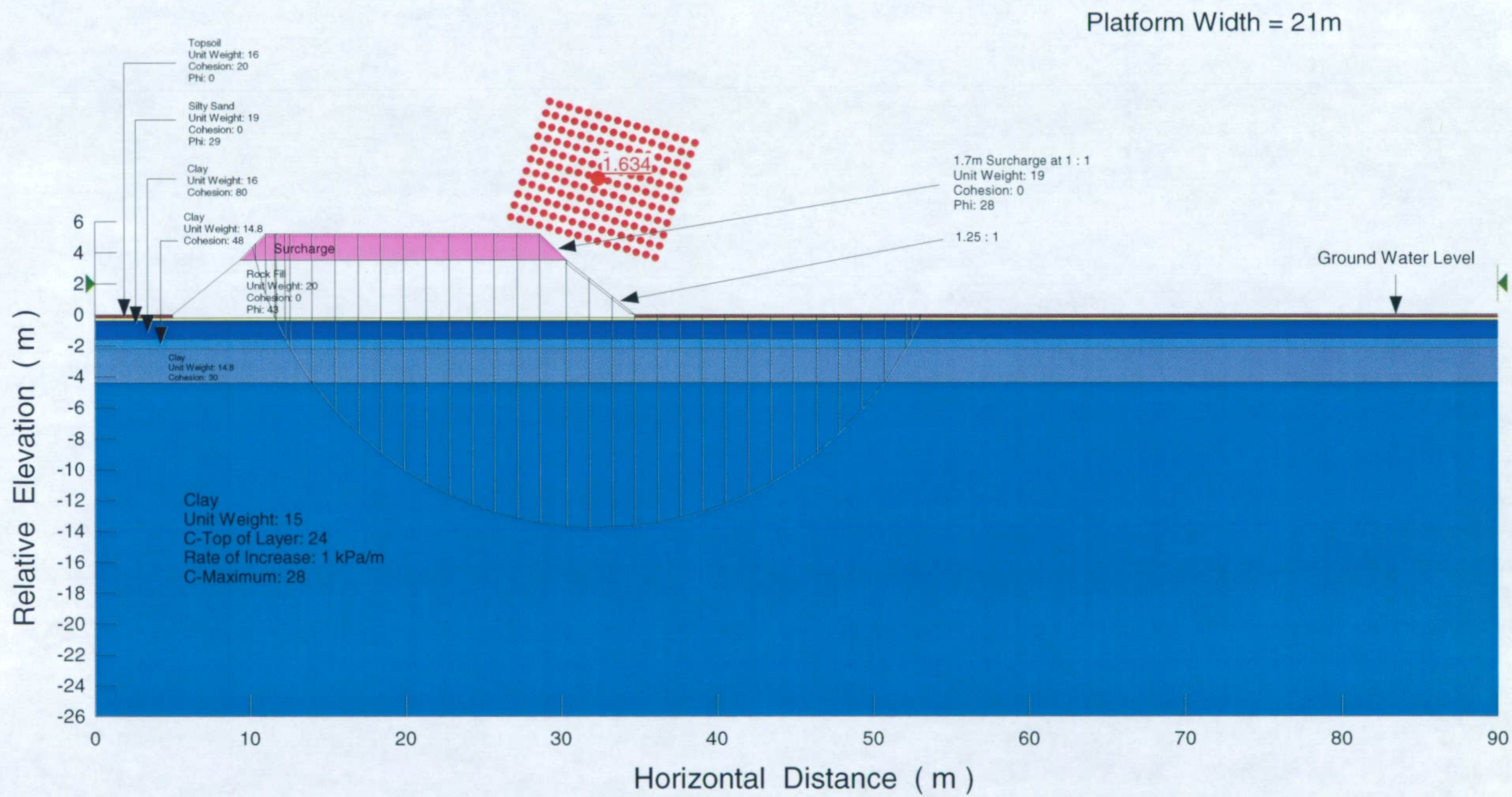
SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 10+923, 3.5m High, Earth Fill Embankment (Plus 1.7m Surcharge)
Undrained Case (Total Stress Analysis)



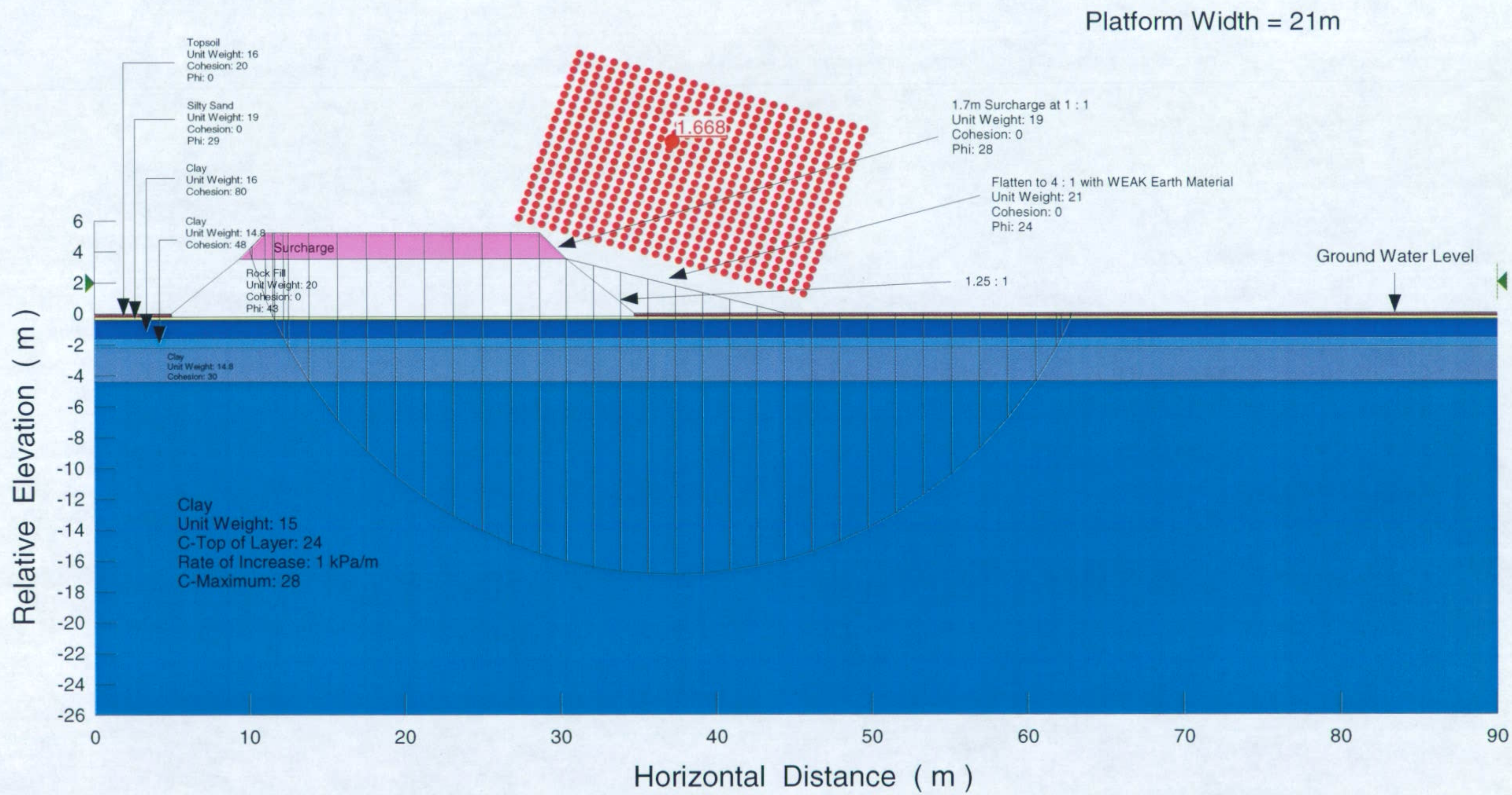
SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 10+923, 3.5m High, Earth Fill Embankment (Plus 1.7m Surcharge)
Undrained Case (Total Stress Analysis)



SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 10+923, 3.5m High, Rock Fill Embankment (Plus 1.7m Surcharge)
Undrained Case (Total Stress Analysis)

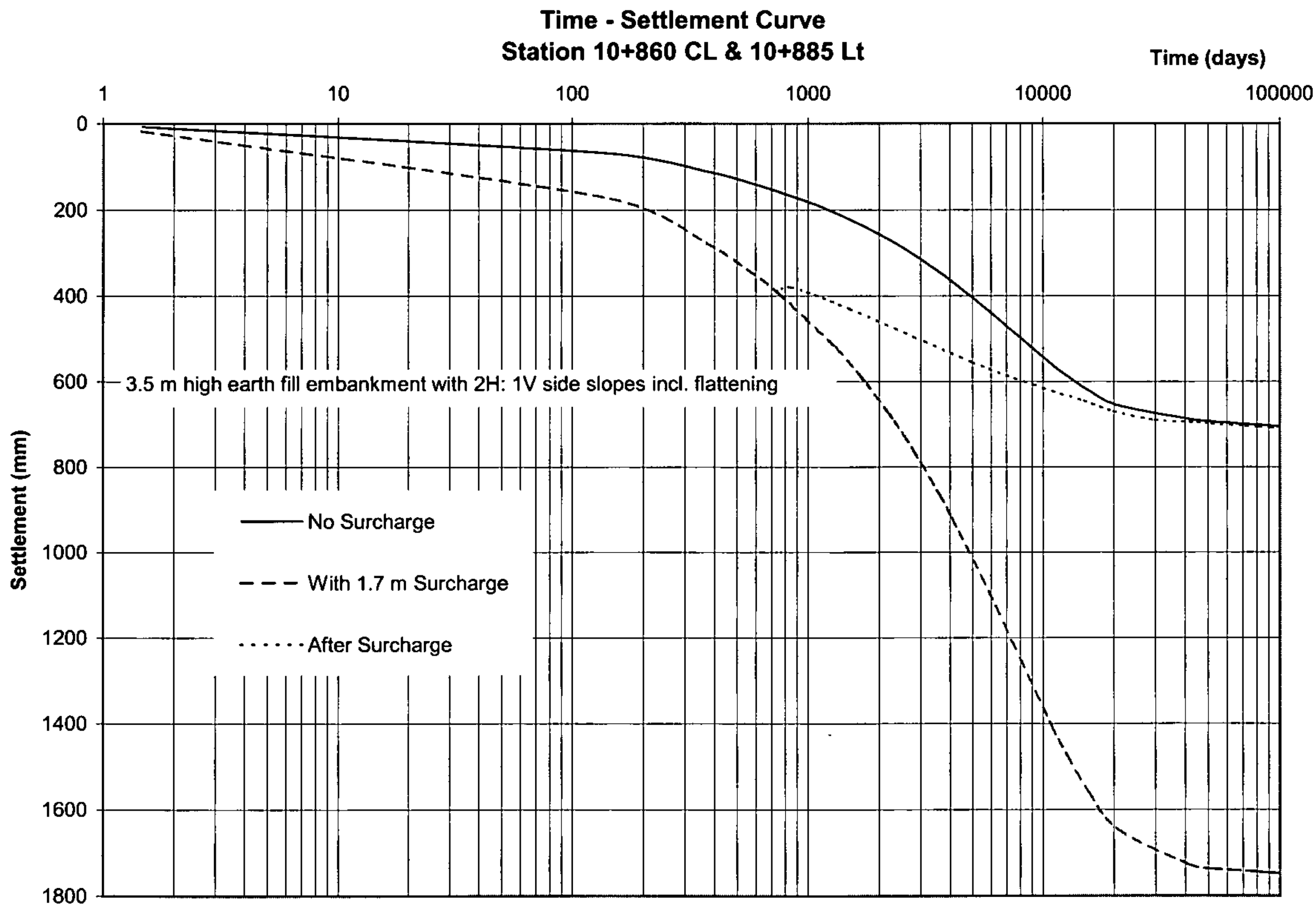


SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 10+923, 3.5m High, Rock Fill Embankment (Plus 1.7m Surcharge)
Undrained Case (Total Stress Analysis)



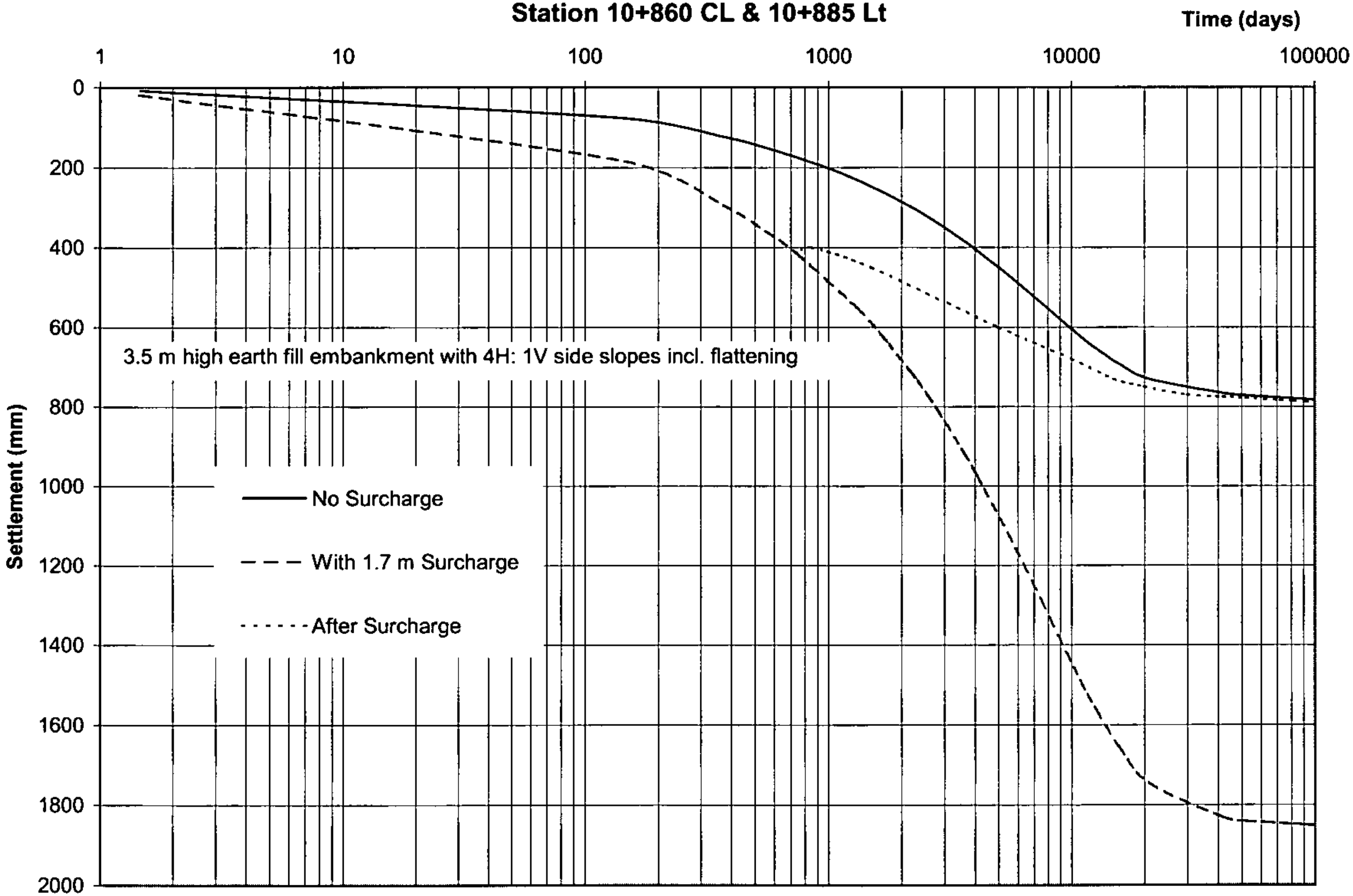
APPENDIX F3

Time-Settlement Curves



Time - Settlement Curve
Station 10+860 CL & 10+885 Lt
Time (days)
1
10
100
1000
10000
100000
Settlement (mm)
0
200
400
600
800
1000
1200
1400
1600
1800
3.5 m high earth fill embankment with 2H: 1V side slopes incl. flattening
No Surcharge
With 1.7 m Surcharge
After Surcharge

Time - Settlement Curve
Station 10+860 CL & 10+885 Lt



5.4 SITE NO. 4 : HIGHWAY 17 (NEW) FILL SECTION BETWEEN STATIONS 11+375 AND 11+690 EASTBOUND LANES AND BETWEEN STATIONS 11+380 AND 11+670 WESTBOUND LANES – BOREHOLES 11+441CL, 11+475Lt, 11+505Rt, 11+540Lt, 11+547CL, 11+572Rt, 11+597CL, 11+622Lt, 11+655Rt, 11+657Lt

A total of ten boreholes was put down for this 225 m stretch of alignment between Station 11+440 and 11+665. The boreholes showed the presence of 0.15 m to 0.25 m thick topsoil or 0.15 to 0.6 m of peat. The organic soils are underlain at most of the borehole locations by surficial sand to sandy silt. These surficial and generally fine grained granular soils were found to extend to a depth of 0.7 m below the ground surface, except in Boreholes 11+475Lt and 11+540Lt, where they extended to 1.4 m.

At depths ranging between 0.3 and 2.1 m below the ground surface, the site is underlain by an extensive clay deposit. The clay is irregularly layered with occasional thin silty clay to silt interlayers. Particle-size distribution analyses show that the material has a high clay-size particle content and is of high plasticity, with Liquid Limit values generally in excess of 50%. Frequently, the measured Liquidity Indices are greater than 1. The in-situ shear strengths, measured by means of Field Vane tests, range from generally 20 to 40 kPa, as shown in Figure C4-1, in Appendix C4.

In some of the boreholes, the clay deposit is underlain by competent granular soils, at depths ranging from 7.5 to 9.0 m below the ground surface, while in others a maximum depth of 19 m for the clay deposit was inferred from Dynamic Cone Penetration tests.

Upon their completion, water levels in the boreholes were measured at depths between 3 and 9 m below the ground surface. These water levels are believed to be unstabilized due to the practically impervious nature of the clay deposit. From the colour of the soil, site observations and the moisture contents of the soil samples, the groundwater table is believed to be at or very near the ground surface level. A piezometer installed in one of the boreholes indicated a mild artesian condition (i.e. water level measured at 0.5 m above the ground surface).

5.4.1 EMBANKMENT STABILITY

In this section, the height of the proposed embankments ranges from about 1.1 to 3.2 m for the eastbound lanes and 0.6 to 2.8 m for the westbound lanes. The difference in embankment heights for the east and westbound lanes is mainly because the existing grade at the site slopes from east to west, albeit mildly.

The foundation stability of the embankments was analyzed by the limit equilibrium method, utilizing Bishop's Simplified method of analyses. For this purpose, the computer program

Slope/W was utilized. Bishop's Simplified method is known to be slightly on the conservative side because of the fact that the side forces on the slices are ignored, as opposed to more rigorous methods.

For the undrained (short-term) stability analyses, undrained shear strengths (c-values) of the clay were based on the Field Vane Test results at each borehole location and assuming a ϕ-value of zero. The c-values used in our analyses ranged from 16 to 80 kPa. No correction factor to the Field Vane tests was applied (e.g. Bjerrum, Aas, etc. correction). Because of this and due to irregularly layered, fat (highly plastic) nature of the clay deposit, a minimum safety factor of 1.40 was deemed necessary. In addition to this, the conditions of the site are variable in relation to the thickness of the clay deposit and to a certain extent its shear strength. The latter can partially be attributed to the layered nature of the clay.

Long-term (drained) analyses were also performed at some selected locations and, as was expected, these were not found to be critical.

In our analyses, a maximum surcharge of 1.6 m was added to the height of the embankments, in order to deal with the settlement aspects, as will be discussed.

The soil parameters used in the slope stability analyses are presented in Table 5.4.1.1.

Table 5.4.1.1 Soil Parameters Used in Slope Stability Analyses

Material Type	Short-Term Analysis			Long-Term Analysis		
	ϕ (degrees)	c (kPa)	γ (kN/m^3)	ϕ' (degrees)	c' (kPa)	γ (kN/m^3)
Embankment Fill (select subgrade material)	30	0	21.5	30	0	21.5
Embankment Fill (Rock Fill)	43	0	18.0-20.0	43	0	18.0-20.0
Sand Backfill (used to replace existing peat and other surficial unsuitable soils)	30	0	20.0	30	0	20.0
Rock Backfill (used to replace existing peat and other surficial unsuitable soils)	43	0	18.0-20.0	43	0	18.0-20.0
Surficial Granular Soils	29	0	19.5	29	0	19.5
Clay	0	16-80	15.0-17.0	24	2	15.0-17.0
Lower sand	33	0	21.0	33	0	21.0
Lower Glacial Till	33	0	21.5	33	0	21.5

Typical embankment slope stability sections are presented in Appendix E4, while a summary of the calculations is also presented in Appendix E4.

The analyses results show that both earth and rock fills with 4H:1V side slopes provide a minimum safety factor of 1.40 for the proposed height of embankments. Rock fills can be built to 4H:1V slopes entirely of rock fill or alternatively the rock fill can be built to the usual 1.25H:1V side slopes which can then be flattened to 4H:1V side slopes, using excess inorganic compactable earth fill materials. In this case, however, the rock fill should be built to the design height and flattened to 4H:1V side slopes, prior to placing the surcharge material.

It should be pointed out that in our analyses, we have assumed that all the existing peat, topsoil and other unsuitable soils will be removed and replaced with suitable granular soils. We have also assumed that southerly from Station 11+550 along the westbound lanes (WBL) and from Station 11+450 along the eastbound lanes (EBL), the existing subgrade will be removed to a depth of not less than 1.0 m and replaced with rock fill (or granular earth fill, where earth fill is to be used to construct the embankment). If unsuitable soils are encountered below 1.0 m depth these too should be removed. This operation should be continued southerly to about Station 11+700.

After examining the options, our recommendation is to use rock fill for the construction of the embankments, using 4H:1V side slopes. As mentioned before, the flattening from 1.25H:1V to 4H:1V side slopes can be implemented using excess, compactable, inorganic earth fill materials. The rock fill is recommended not only because it provides a higher factor of safety against embankment failures but also it is believed that it will provide a better performance of the completed road, especially where the subgrade consists of very soft to soft clays immediately beneath the existing peat or topsoil.

5.4.2 SETTLEMENT OF EMBANKMENTS

Consolidation test results show that the weak clay deposit is a highly compressible material. The anticipated settlements under the embankment loadings along this stretch will depend on the

- Thickness of the clay deposit
- Stresses imposed by the embankments (i.e. height of the embankment, weight of the material used, side slopes utilized, etc.)
- Relative thickness of the relatively more compressible zones/layers in the clay deposit

Table 5.4.2.1 Fill Heights at Station Centreline

Station Number	Proposed Embankment Height (m) **WBL**	Proposed Embankment Height (m) **EBL**
11+441	1.57	2.94
11+475	2.77	3.14
11+505	2.20	3.19
11+540	1.35	2.52
11+547	1.18	2.41
11+572	0.84	2.05
11+597	0.68	1.93
11+622	0.77	1.55
11+655	0.65	1.15
11+657	0.62	1.12

Our analyses show that under the proposed height of the embankments (as detailed in Table 5.4.2.1, presented above) the anticipated settlements range from about 90 mm to 500 mm.

The highest settlement estimate values were obtained along the EBL Stations 11+441 where the height of the proposed embankment is 2.94 m and the thickness of the clay deposit is greater than 10 m and also at Station 11+547 where the thickness of the clay is about 11m and the embankment height is 2.41 m. At Stations 11+572 and 11+597 where the embankment height is only about 2.0 m, the settlements are also high (i.e. about 400 to 500 mm) because the inferred thickness of clay deposit is about 18 to 19 m. The quoted settlement values were obtained assuming that stripping requirements (i.e. generally at least 1.0 m) will be effected, as discussed earlier.

These settlements can be expected to take place over a period of about three to more than ten years, depending on the thickness of the clay layer, as well as the presence of more pervious silt to clayey silt interbeds, the presence of which is difficult to model into the calculations.

In order to alleviate the effects of these rather excessive settlements, we recommend surcharging. Our stability analyses show that the maximum height of surcharge that can be applied at most critical locations is 1.6 m. We also understand that the time restraints limit the maximum length of surcharging to between one and two years, more realistically about 1.5 to 2 years. Based on this, the following surcharge programme is recommended.

Surcharging should start along the WBL at Station 11+400 and gradually increase to 1.6 m height at Station 11+450. It should remain at this height to Station 11+650, gradually reducing to zero at Station 11+680.

Along the EBL, the surcharging should start at about Station 11+380 and gradually increase to 1.6 m height at Station 11+420 and remain at that level to Station 11+670. Southerly from Station 11+670, the surcharge should be reduced to zero at Station 11+700.

With this approach, the anticipated settlements after 1.5 years of surcharging are reduced to a maximum of about 150 mm (at several locations) along the EBL and to a maximum of about 50 mm along the WBL. These values would reduce to about 100 mm and 25 mm, respectively after two years of surcharging. These residual settlements are expected to take place gradually over a period of three to more than ten years and should therefore not cause major concern, if the surcharge can remain in place at least two years. Alternatively, wick drains may be used to speed up the consolidation settlements. This will, however, have to be done with due consideration of the artesian condition encountered at the site.

Another alternative would be to use light weight fill which will reduce the magnitude of the anticipated settlements. This alternative is, however, not believed to be viable based on cost and reliability for the performance of the highway.

In any event, it is recommended that surcharging be carried out with proper instrumentation for monitoring settlements. It is furthermore recommended that the surcharge be placed gradually (i.e., preferably at least 3 layers, starting from one end of the site and proceeding to the other end), to allow excess pore pressures to dissipate.

Embankments should be provided with a widened cross-section to allow for settlements of the underlying soils and any possible future minor grade raises. In this case, we recommend that the road platform should be widened by at least 2 m on each side of the centerline (total 4 m). This is also in accordance with the Northern Region Engineering Directive NRE-98-200. In addition, certain amount of regrading may be necessary due to settlements, including the side slopes after the surcharging period.

5.4.3 CONSTRUCTION

For embankment construction, all organic or otherwise unsuitable materials should be removed within an envelope given by an imaginary slope no steeper than 1:1 under the entire embankment width including flattening to 4:1. As mentioned before, a minimum of 1.0 m stripping is recommended starting from Station 11+550 along the WBL and 11+450 along the EBL and extending to Station 11+700. We recommend that where the height of embankment is less than 1.3 m, the thickness of the stripping and soil replacement be increased from 1.0 to 1.2 m, gradually increasing to 1.5 m where the height of the embankment is 1.0 m or less (i.e., provide a minimum of 2.0 m separation between the top of pavement and the clay subgrade). This is to improve the performance of the pavement over soft subgrade conditions.

After stripping, the exposed subgrade should be inspected and approved by the Quality Verification Engineer. The subgrade may need to be compacted from the surface using a suitable compactor. This should, however, only be done at the discretion of the Quality Verification Engineer, as it may not be feasible to effect surface compaction due to site conditions (i.e. high water table), weak nature of the clay, etc. It is believed that because of the presence of high water table at the site dewatering will be required to facilitate the stripping and backfilling operations. This is normally achieved by gravity drainage and pumping from open sumps. Construction procedures will have to be cognizant of the soft nature of the clay at most borehole locations.

The type of material to be used for soil replacement (i.e. to replace the stripped soils to the existing original ground surface) should be compatible with the proposed embankment fills. For example, where rock fill is to be used for embankment construction, the existing soils beneath the existing ground surface should be replaced with rock fill. In this case, it is important that the rock fill be penetrated the exposed subgrade where the subgrade consists of weak clay (i.e. very soft to firm consistency) and the size of the rock fill should be limited to 400 mm.

Similarly, where the embankment is to consist of earth fill, granular fill, which is compactable, should be used (e.g. Granular 'B' material). In this instance, the first one to two lifts of the fill may need to be thicker than normal (i.e. thicker than 300 mm lifts).

The materials used for the construction of the embankment should consist of approved acceptable earth or rock fill. The earth 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 (SPMDD) (OPSS 206 and OPSS 501).

As mentioned before, we recommend that the embankments be constructed using rock fill, if available. Rock fill should be placed in lifts with thickness not exceeding 1.5 m. The rock fill should be compacted by overlapping track prints of the construction equipment. Depending on the size and type of equipment used, six to eight passes along each path should be required. The surface voids of each layer of rock fill materials should be filled with fragments of rock before the next layer is deposited. The final surface of rock fill material should be compacted by at least two additional passes and should be blinded with compacted fine fill material (or chinked) prior to installation of the road subbase layer.

Rock fill material of maximum nominal size of 400 mm could be used and should consist of pieces of hard and durable rock with no sign of decomposition. Concrete, masonry, brick and similar materials should not be used.

A geotextile separator is recommended between rock fill and any native soil surcharge in order to prevent infiltration of fine soils into the rock fill. Any silty soils left at the surface of the rock fill, after the surcharge is removed, could be potential cause of possible frost heaving of the pavement. The separator should comprise of a Class II non-woven geotextile as per OPSS 1860 with a Filtration Opening Size (FOS) of 50 to 100 μm.

As the removal of rock fill after the surcharging period may not be practical, the use of granular subbase material may be preferable (e.g. Granular 'B') rather than rock fill for the surcharge materials. In this instance, in order to reduce the quantities of granular subbase material to be used, a minimum settlement of 100 mm can be assumed between Stations 11+460 and 11+600 along the WBL. Along the EBL, a minimum settlement of 150 mm can be assumed from Station 11+440 to 11+620. This value should be reduced to 100 mm between Station 11+620 and 11+660.

Proper erosion control measures should be implemented both during construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

APPENDIX E4

Slope Stability Analysis Results

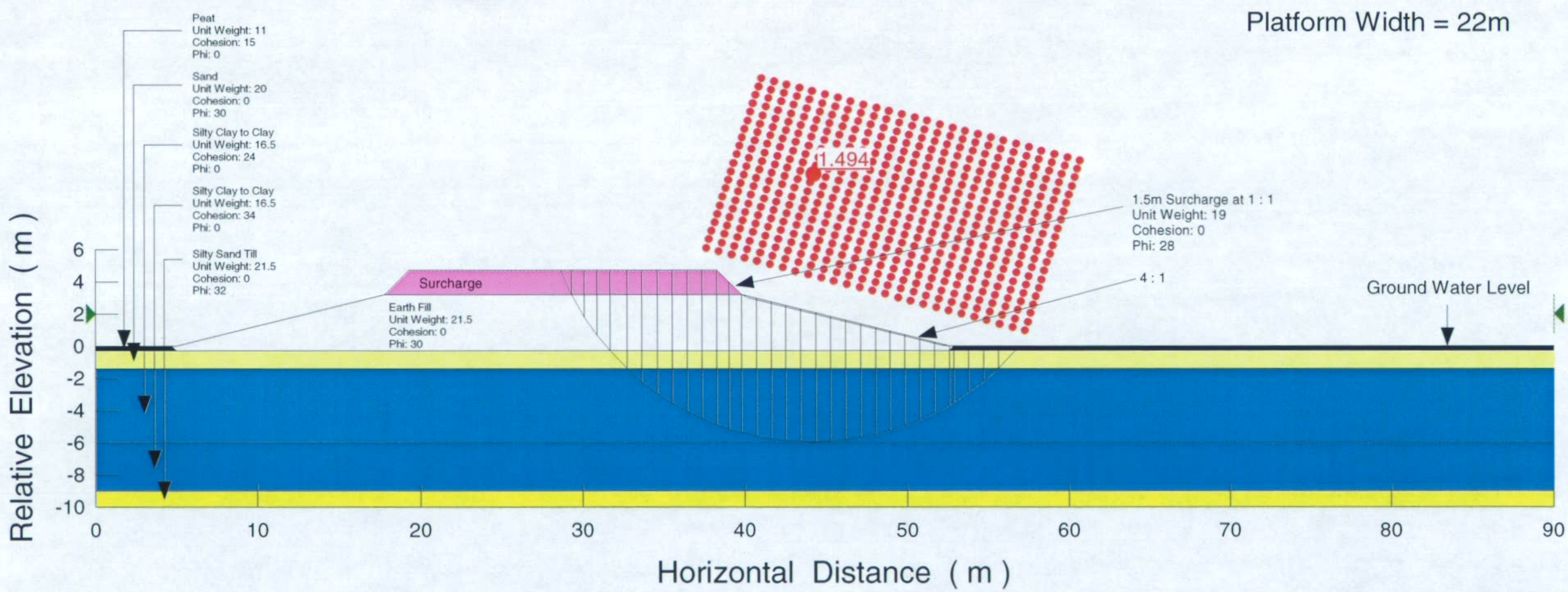
Appendix E4

Calculated Minimum Safety Factors – Short-Term Analysis

Borehole Number	Proposed Embankment Height (m) EBL	Additional Surcharge Height (m)	Total Embankment Height with Surcharge (m)	Embankment Material	Side Slopes (H:V)	Mini-mum Safety Factor
11+441	2.94	1.5	4.44	Rockfill	1¼ :1	1.63
				Rock Fill +Earth Fill	1¼:1+4:1*	1.66
				Earth Fill	2:1	1.55
				Earth Fill	4:1	1.57
11+475	3.14	1.5	4.29	Rockfill	1¼ :1	1.38
				Rock Fill +Earth Fill	1¼:1+4:1*	1.58
				Earth Fill	2:1	1.34
				Earth Fill	4:1	1.49
11+505	3.19	1.5	4.02	Rockfill	1¼ :1	1.22
				Rock Fill +Earth Fill	1¼:1+4:1*	1.52
				Earth Fill	2:1	1.19
				Earth Fill	4:1	1.46
11+540	2.52	1.5	4.02	Rockfill	1¼ :1	1.50
				Rock Fill +Earth Fill	1¼:1+4:1*	1.69
				Earth Fill	2:1	1.40
				Earth Fill	4:1	1.60
11+547	2.41	1.5	3.96	Rockfill	1¼ :1	1.75
				Rock Fill +Earth Fill	1¼:1+4:1*	2.44
				Earth Fill	2:1	1.75
				Earth Fill	4:1	2.20
11+572	2.05	1.5	3.55	Rockfill	1¼ :1	1.55
				Rock Fill +Earth Fill	1¼:1+4:1*	1.98
				Earth Fill	2:1	1.37
				Earth Fill	4:1	1.60
11+597	1.93	1.5	3.43	Rockfill	1¼ :1	1.39
				Rock Fill +Earth Fill	1¼:1+4:1*	2.00
				Earth Fill	2:1	1.43
				Earth Fill	4:1	1.81

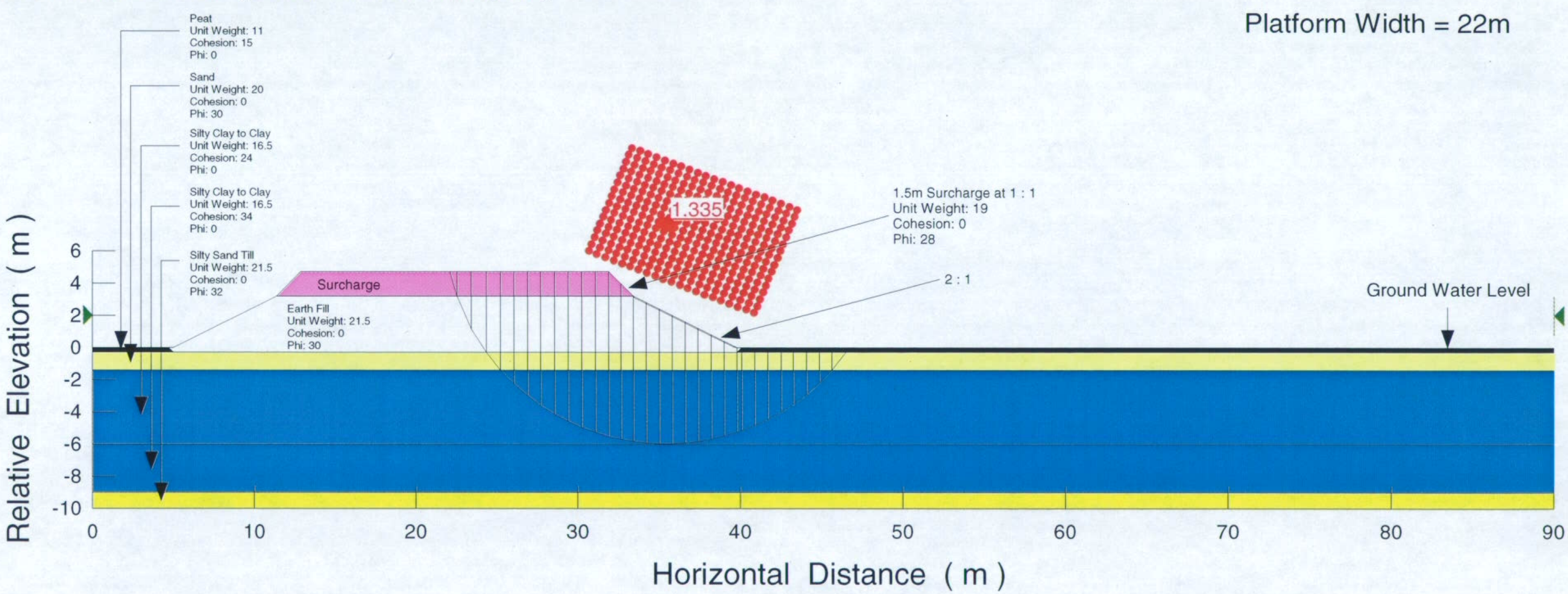
*1 ¼:1 side slopes built of rock fill and then flattened to 4:1 side slopes using earth fill prior to placing the surcharge or 4:1 rock fill.

SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 11+475, 3.2m High, Earth Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)



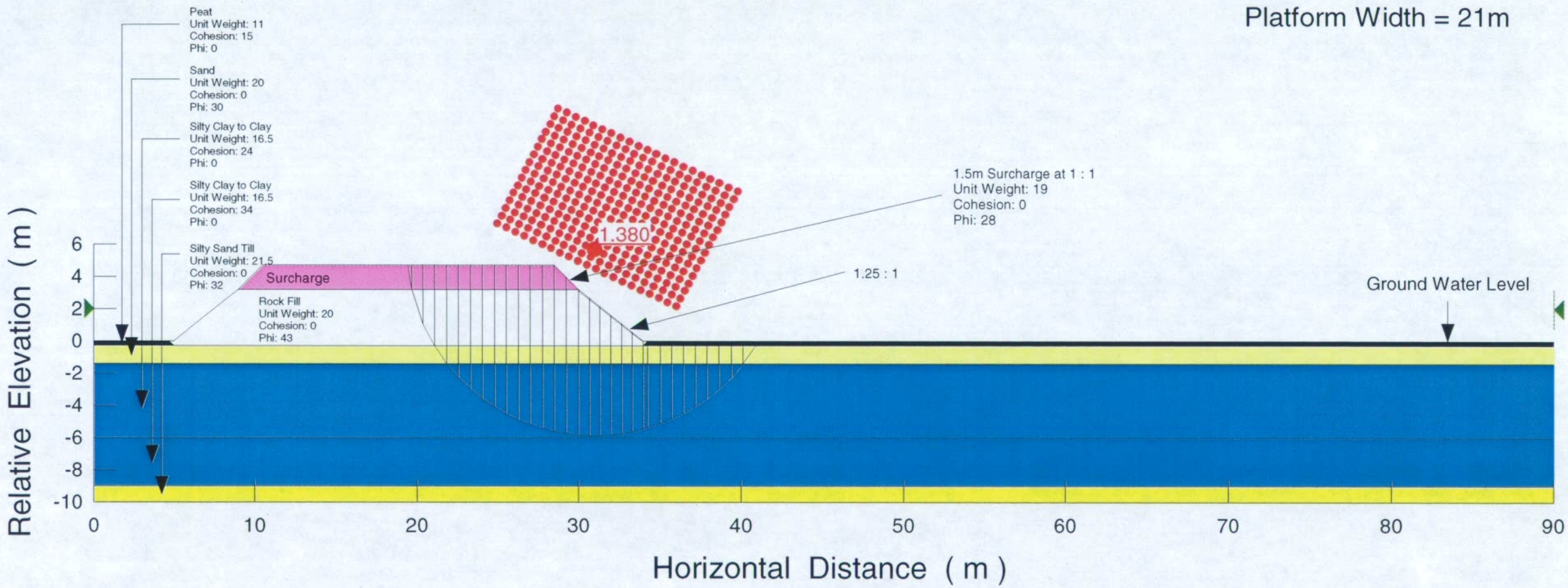
SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 11+475, 3.2m High, Earth Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)

Platform Width = 22m

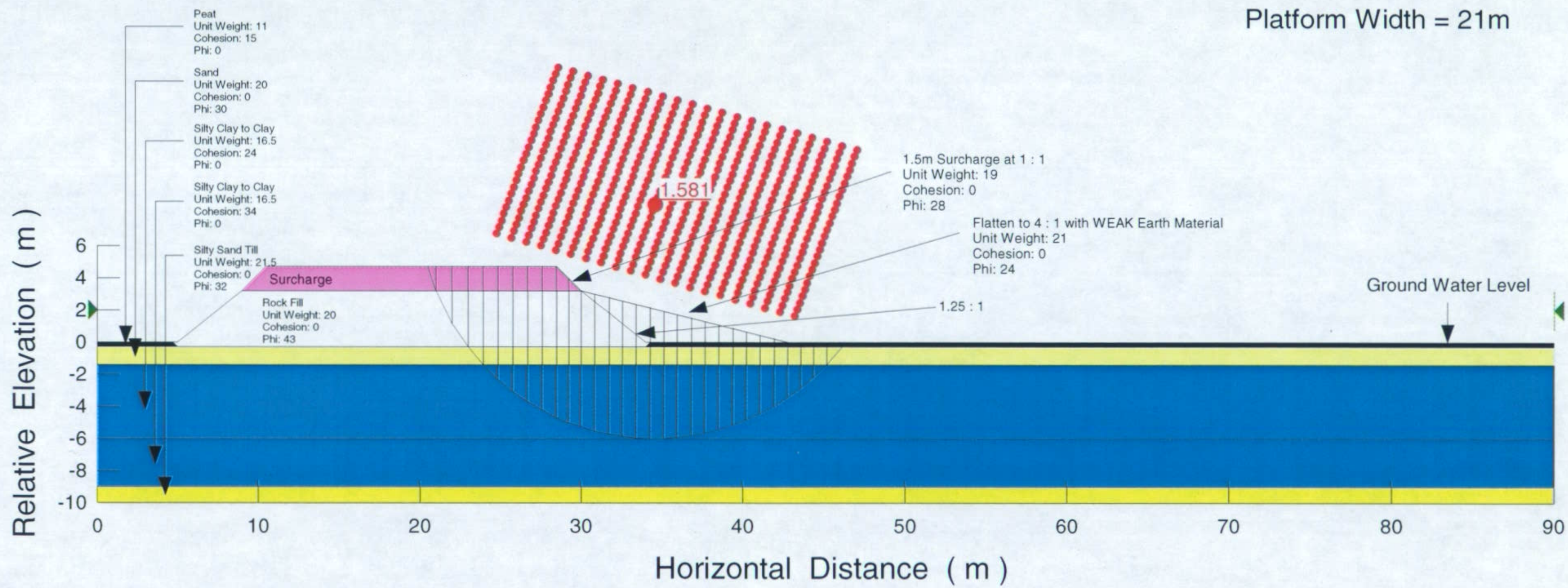


SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 11+475, 3.2m High, Rock Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)

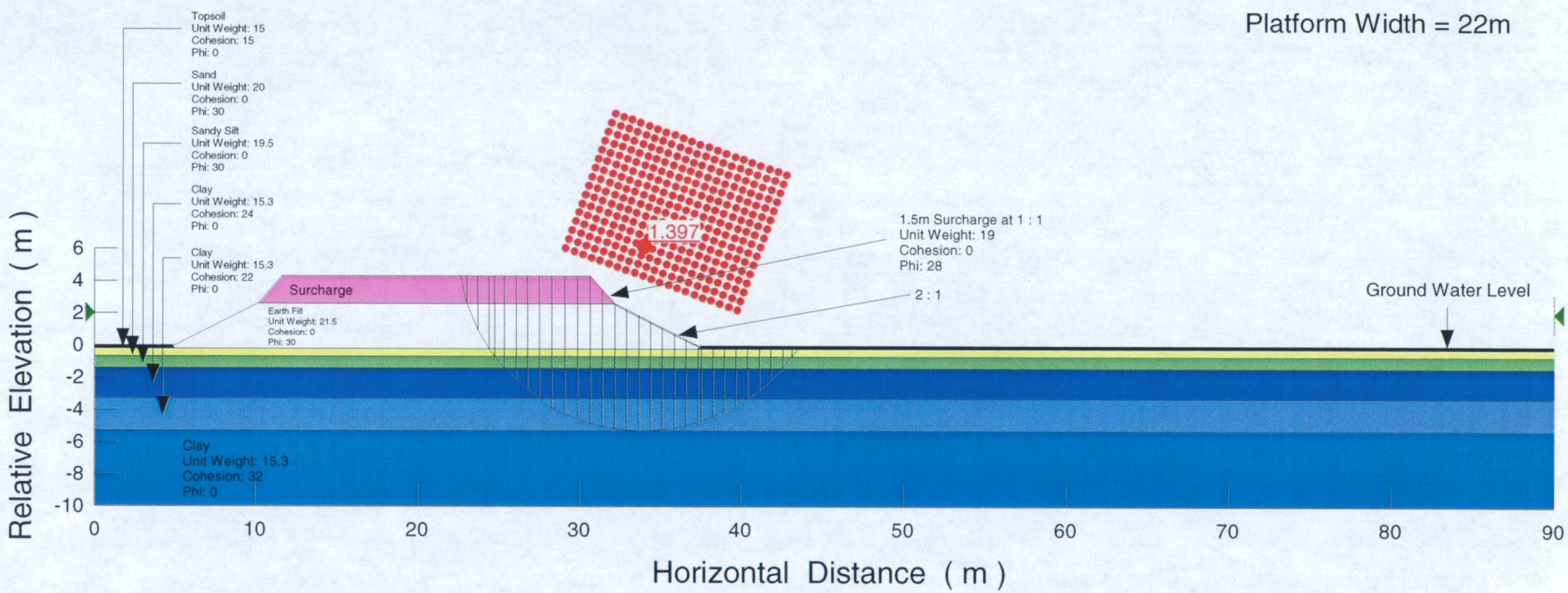
Platform Width = 21m



SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 11+475, 3.2m High, Rock Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)

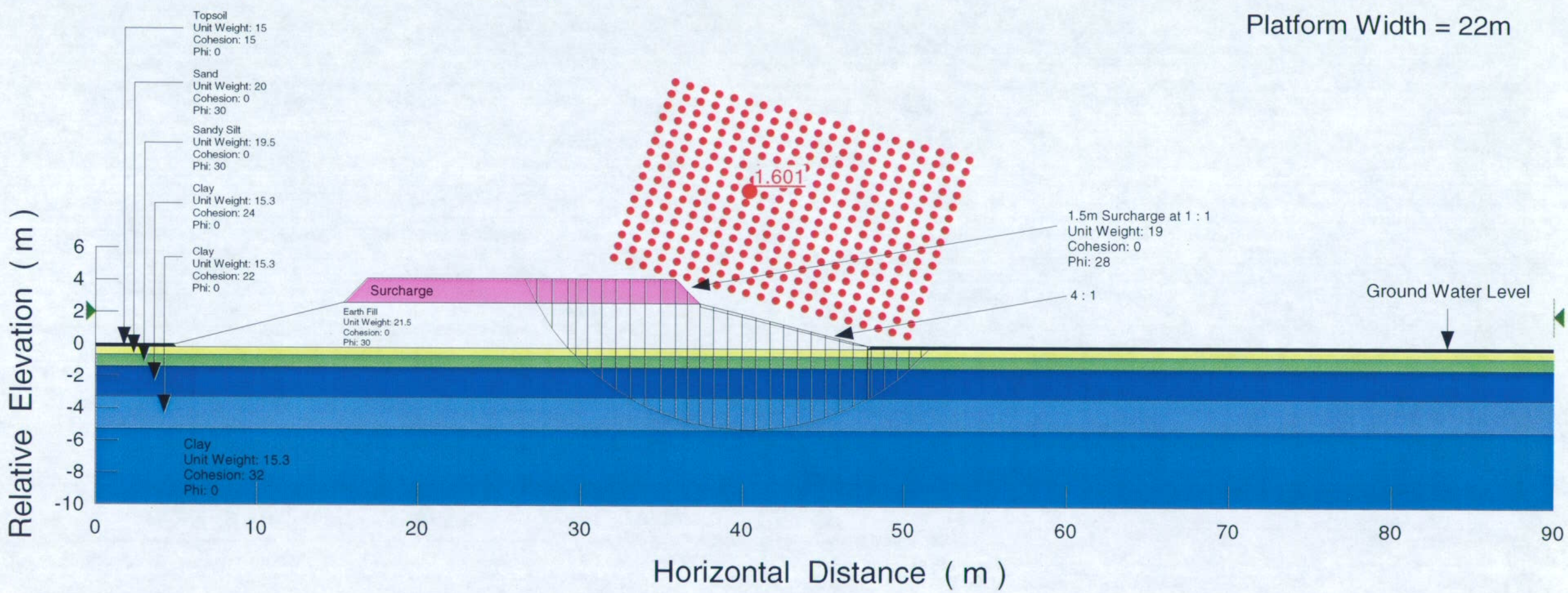


SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 11+540, 2.6m High, Earth Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)

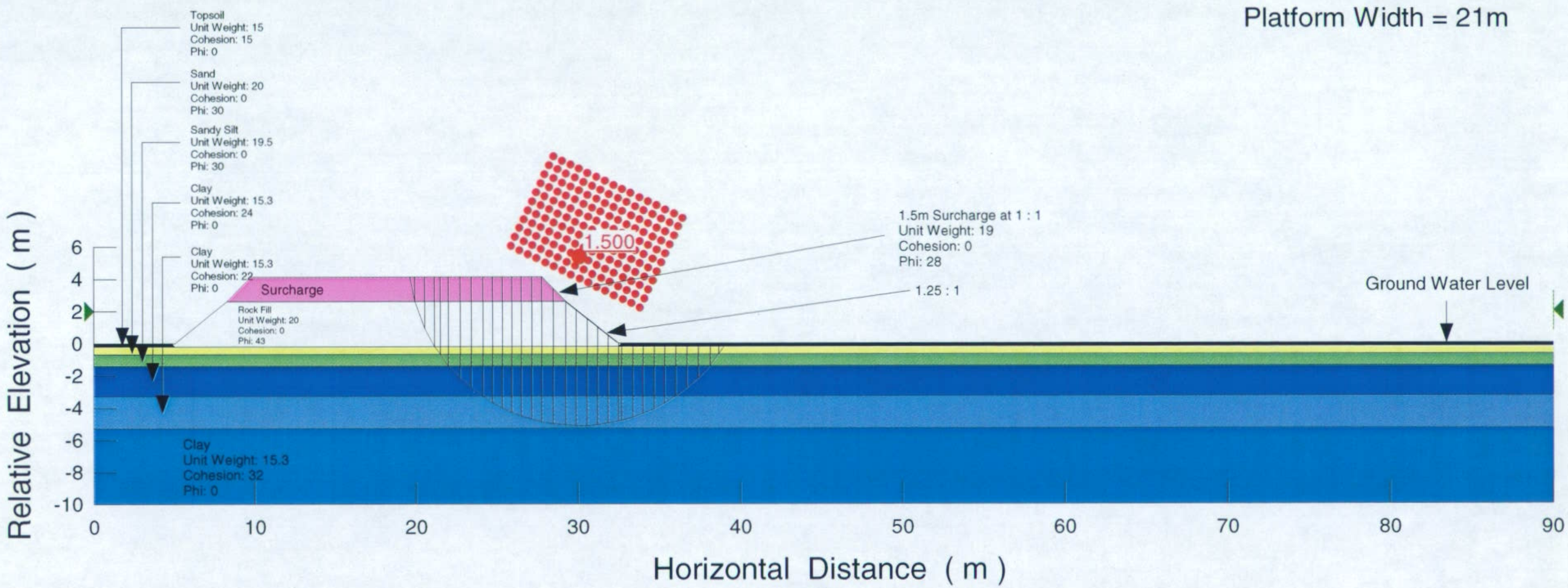


SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 11+540, 2.6m High, Earth Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)

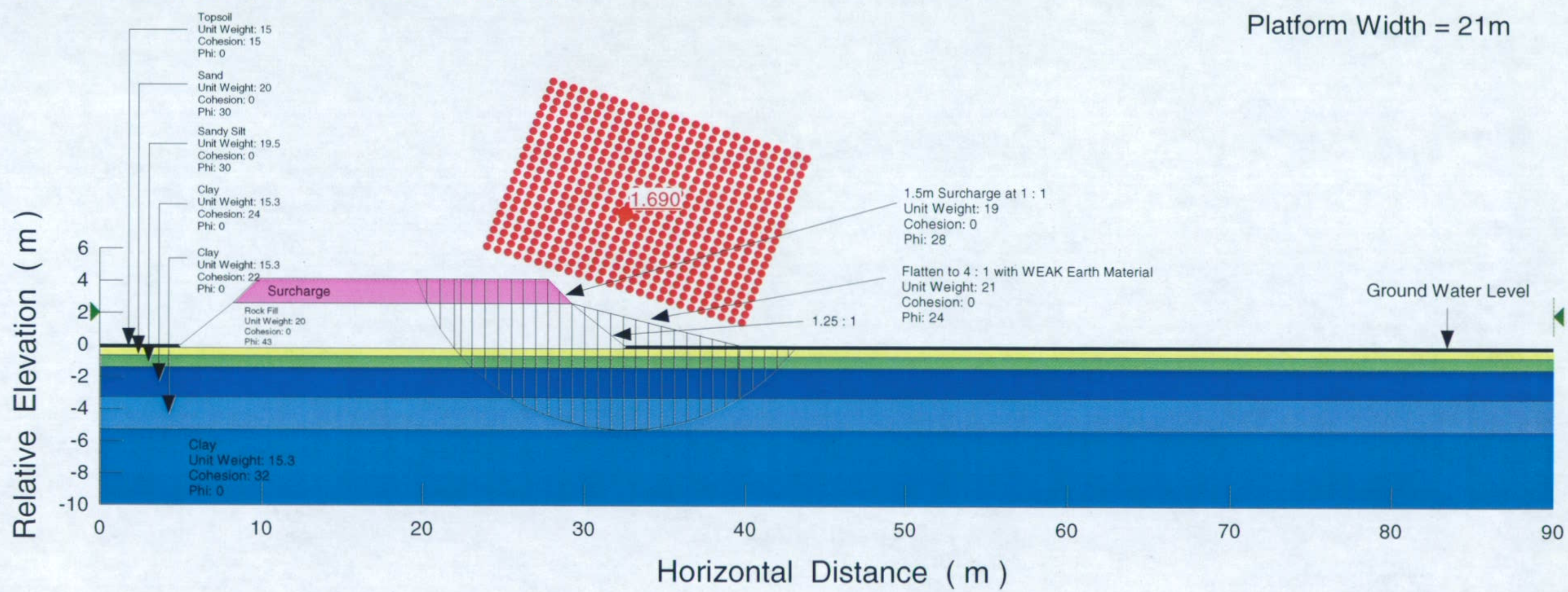
Platform Width = 22m



SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 11+540, 2.6m High, Rock Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)



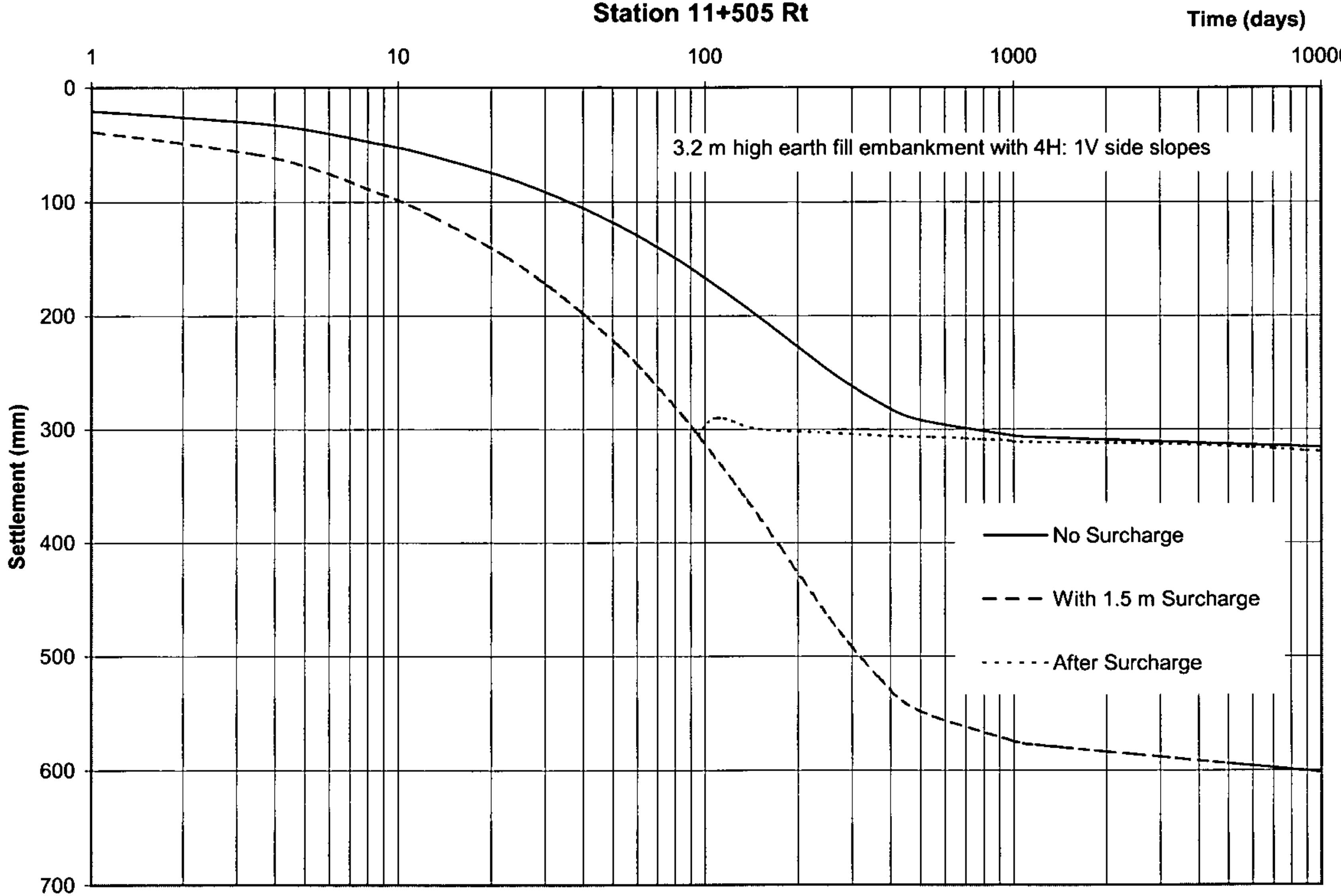
SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 11+540, 2.6m High, Rock Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)



APPENDIX F4

Time-Settlement Curve

Time - Settlement Curve
Station 11+505 Rt



5.5 **SITE NO. 5 :** HIGHWAY 17 (NEW) SWAMP SECTION BETWEEN STATIONS 11+800 AND 13+400 AND SWAMP SECTION ALONG HIGHWAY 638

5.5.1 SWAMP SECTION BETWEEN STATIONS 11+800 AND 13+400

5.5.1.1 EMBANKMENT STABILITY

Along this swamp section, as shown on Drawing No. 5A, the height of proposed embankment with vertical Profile 'A' along the EBL ranges from about 0.5 m at Station 11+800, gradually increasing to about 2.4 m at Station 12+550 and thereafter remaining at about 2.4 to 2.5 m to about Station 12+760. From thereon, the proposed embankment height gradually reduces to about 1.1 m at about Station 13+200, remaining between 0.9 m and 1.1 m thereafter.

Along the WBL (Drawing No. 5B), the fill starts at about Station 11+850. The height of the embankment increases from about 0.1 m gradually to about 2.4 m at about Station 12+550 (similar to the EBL profile). From there, it remains between about 2.4 and 2.0 m to about Station 13+000 and then gradually reduces to about 1.0 m at about Station 13+200 and remains between 0.9 and 1.1 m thereafter, similar to the EBL profile.

A second proposed vertical alignment reduces the height of embankments along both EBL and WBL by 0.6 m between Stations 11+830 and 12+890 (Profile 'B') in comparison with Profile 'A'.

The details of the proposed embankment heights are given in Appendix E5-1 and also on the profiles in Drawings 5A, 5B, 5C and 5D.

In this swamp section, the stability of the embankments was analyzed by the limit equilibrium method, utilizing the computer program Slope/W. In most cases, Bishop's Simplified method was used, which is known to be slightly conservative (in comparison with more rigorous methods) as in this method, the side forces on the individual slices are ignored. In certain sections of the swamp, where the measured undrained shear strengths are extremely low (i.e. less than 10 kPa) circular failure surfaces did not appear to represent a realistic picture. In such cases, the stability was also checked by forcing the failure surfaces along the horizontal weak zones, or by using Janbu method which is known to be more akin to a sliding block analysis. In many cases, these methods yielded safety factors which are as much as about 20% lower (e.g. 1.25 vs 1.50).

In order to reduce the magnitude of the post-construction settlements, as will be discussed in the next section, and to come up with an optimum design, various depths of sub-excavation of the very weak subgrade soils along with various combinations of surcharge loadings were examined at various select borehole locations.

For the undrained (short-term) stability analysis, undrained shear strengths (c-values) were utilized based on the Field Vane Test results at individual borehole locations. The angle of internal friction was assumed to be zero, as is normally done in undrained stability analysis. The c-values used in our analysis ranged from 4 kPa to 84 kPa. No correction factor (such as Bjerrum or Aas correction) was applied to the Field Vane Test results. A minimum safety factor of 1.40 was deemed necessary, where feasible, because of the extremely low shear strengths (c-values) that were measured in some of the boreholes. This was felt to be necessary because if an insignificant error such as 1 kPa is made in carrying out Field Vane tests, which is quite conceivable (in spite of taking all precautions such as using 'vane collars'), this would lead to an error of 25% over a shear strength value of 4 kPa. Whereas, under normal circumstances, with a typically low shear strength value of 25 kPa, such an error would represent a variation of only 4%.

Long-term or drained analysis was also carried out at selected borehole locations and, as can be expected, these were found to be less critical than the short-term (undrained) analysis.

In our analyses, surcharge loads of between 0.3 and 1.6 m of earth were also added to the embankment loadings, in order to deal with the settlement issues, as will be discussed in the next section of this report.

The soil parameters used in the slope stability analyses are presented in Table 5.5.1.1.

Table 5.5.1.1 Soil Parameters Used in Slope Stability Analyses – Highway 17 (New)

Soil Type	Short-Term Analysis			Long-Term Analysis		
	ϕ (degrees)	c (kPa)	γ (kN/m^3)	ϕ' (degrees)	c' (kPa)	γ (kN/m^3)
Embankment Fill (select subgrade material)	30	0	21.5	30	0	21.5
Embankment Fill (Rock Fill)	43	0	18.0-20.0	43	0	18.0-20.0
Sand Backfill (used to replace existing peat and other surficial unsuitable soils)	30	0	20.0	30	0	20.0
Rock Backfill (used to replace existing peat and other surficial unsuitable soils)	36	0	18.0-20.0	36	0	18.0-20.0
Surficial Silty Sand	30	0	20.0	30	0	20.0
Sand and Gravel	35	0	21.5	35	0	21.5
Fine Sand	30	0	20.0	30	0	20.0
Clay	0	4-84	15.0	20-24	0-2	15.0
Silty Sand (Probable Till)	33	0	21.5	33	0	21.5

Typical embankment slope stability sections are presented in Appendices E5-2 and E5-3.

We recommend the use of rock fill throughout this section, except to the south of Station 13+040. This is because rock fill will penetrate the upper zone of the very soft clay, thus strengthening it. In addition, in our opinion, rock fill will provide a better performance of the highway under low embankment situations where the subgrade is of very low shear strength. To the south of this station the use of earth fill is preferable to rock fill because along this section the subgrade is generally fine sand. In all cases 4H:1V side slopes were assumed as this is desirable for low embankments.

In our analyses, we assumed that all the existing peat, topsoil and any other unsuitable materials will be removed and will be replaced with suitable materials as per MTO procedures for embankments over swamp (OSPD 203.010).

The analyses results show rock fills with 4H:1V side slopes generally provide a minimum factor of safety 1.40 for the proposed height of embankments along Profile 'B' and with a few exceptions along Profile 'A' (where in several cases the factor of safety is about 1.3), provided that the following procedures are followed.

<u>Profile 'A'</u>

From the north end of the site (Station 11+800) to Station 11+880, stripping in accordance with OPSD 203.010 should be applied, prior to placing the rock fill. South of Station 11+880, the removal of existing soil should be increased to reach at least 1.0 m below the existing ground surface at Station 11+900. Starting at this station (Station 11+900), rock penetration into the clay subgrade should not be less than 0.5 m, after stripping the upper 1.0 m. The purpose of this exercise is to strengthen the clay within the upper 0.5 m (or to a depth of not less than 1.5 m below the existing ground surface). This process should be followed from Station 11+900 to 12+530 (i.e. removing at least 1.0 m and rock penetration not less than another 0.5 m). At Station 12+530 the subexcavation should be increased from 1.0 m to reach 2.0 m below the existing grade at Station 12+545. Below this, rock penetration into the subgrade should not be less than 0.5 m, as before, to strengthen the clay. This should be continued between Stations 12+545 and 13+040 beyond which the subexcavation should be decreased to normal stripping south of Station 13+050. These procedures are summarized below.

- From north end of the site to Station 11+880, carry out normal swamp soil removal.
- At Station 11+880, start increasing depth of stripping to reach 1.0 m at Station 11+900.

- From Station 11+900 to 12+530, remove minimum 1.0 m of soil + 0.5 m rock penetration into clay subgrade.

- At Station 12+530, start increasing the depth of subexcavation from 1.0 m to reach 2.0 m at Station 12+545 + at least 0.5 m rock penetration into clay subgrade.

- From Station 12+545 to 13+040, sub-excavate 2.0 m + minimum 0.5 m rock penetration into clay subgrade. The stripping should extend a minimum of 3.0 m beyond the toe of the embankment.

- At Station 13+040, discontinue rock penetration into subgrade and start reducing sub- excavation to normal swamp stripping at Station 13+050.

- South of Station 13+050, carry out normal swamp stripping (generally 0.2 to 0.5 m) procedures and use earth fill.

<u>Profile 'B'</u>

Stripping or sub-excavation for Profile 'B' is similar to Profile 'A' with some minor variations, as shown in Table 5.5.1.2.

Table 5.5.1.2 Amount of Required Stripping and/or Subexcavation – Highway 17 (New)

Section (Station to Station)	Stripping / Subexcavation for Profile 'A'	Stripping / Subexcavation for Profile 'B'
North End to 11+880	Normal swamp soil removal	Normal swamp soil removal
11+880 to 11+900	At Station 11+880, start increasing depth of stripping to reach 1.0 m at Station 11+900	
11+880 to 12+000		Normal swamp soil removal + 0.5 m rock penetration into clay subgrade
11+900 to 12+530	Remove minimum 1.0 m of soil + 0.5 m rock penetration into clay subgrade	
12+000 to 12+530		Remove minimum of 1.0 m of soil + 0.5 m rock penetration into clay subgrade
12+530 to 12+545	At Station 12+530, start increasing the depth of subexcavation from 1.0 m to 2.0 m at Station 12+545 + at least 0.5 m rock penetration into clay subgrade	At Station 12+530, start increasing the depth of subexcavation from 1.0 m to 2.0 m at Station 12+545 + at least 0.5 m rock penetration into clay subgrade
12+545 to 12+860		Sub-excavate 2.0 m + minimum 0.5 m rock penetration into clay subgrade. The subexcavation should extend a minimum of 3.0 m beyond the toe of the embankment
12+545 to 13+040	Sub-excavate 2.0 m + minimum 0.5 m rock penetration into clay subgrade. The stripping should extend a minimum 3.0 m beyond the toe of the embankment	

12+860 to 12+880		At Station 12+860, start reducing the depth of subexcavation from 2.0 m to 1.5 m at Station 12+880 + minimum 0.5 m rock penetration into clay subgrade. The subexcavation should extend a minimum of 3.0 m beyond the toe of the embankment
12+880 to 13+040		Sub-excavate 1.5 m + minimum 0.5 m rock penetration into clay subgrade. The subexcavation should extend a minimum of 3.0 m beyond the toe of the embankment
13+040 to 13+050	At Station 13+040, discontinue rock penetration into subgrade and start reducing subexcavation to normal swamp soil removal at Station 13+050	At Station 13+040, discontinue rock penetration into subgrade and start reducing subexcavation to normal swamp soil removal at Station 13+050
13+050 to South End	Normal swamp soil removal (0.2 to 0.5 m) and use earthfill	Normal swamp soil removal (0.2 to 0.5 m) and use earthfill

From stability point of view, Profile 'B' is much superior in comparison with Profile 'A' as it reduces the risk of occasional failures of the embankments due to the presence of very soft clay subgrade during construction.

Removal of soil beyond normal swamp stripping and rock penetration into subgrade can be avoided by using geogrid reinforcement. This, however, is believed to be more costly based on our preliminary discussion with a supplier. If you wish to pursue this avenue further, we will be pleased to provide more details.

5.5.1.2 SETTLEMENT OF EMBANKMENTS

Settlement of the embankments will be largely governed by the consolidation characteristics of the weak and compressible clay deposit underlying the site. Consolidation of the clay and time rate of consolidation also depends on the following factors, in addition to consolidation characteristics.

- Variation of consolidation characteristics of the clay deposit throughout the site.
- Thickness of the deposit.
- Relative thicknesses of the relatively more compressible zones/layers in the clay deposit.
- Stresses imposed by the embankments (i.e. height of the embankment, weight of the materials used to build the embankment, etc.)
- Absence or presence of sand and silt deposits which will help to more favourably distribute the embankment stresses on the weak clay, as well as influencing time rate of consolidation.

As was mentioned before, least favourable conditions were encountered near the centre of this swampy area, between about Stations 12+500 and 13+040, where the clay is weaker and more compressible, generally deeper and in most cases, lacks surficial or intermediate sand and/or silt layers. As well in this area, as shown on the vertical profiles presented on Drawings 5A, 5B, 5C and 5D and in Appendix E5-1, the embankment heights are highest (i.e. between 2.0 and 2.5 m for Profile 'A' and 1.4 to 1.9 m for Profile 'B'). Because of these reasons, our analysis, using consolidation test results presented in Figures B5-8 to B5-15, shows that total settlements of the order of 0.8 m (Profile 'A') and 0.6 m (Profile 'B') can be expected over a 10-year period of time, under the stresses imposed by the embankments, while near the north and south ends of the site, the anticipated settlements are less than

0.2 m, and will take place within relatively shorter periods of time. One way of alleviating the settlement problem is to surcharge the embankments. We understand that there is a time frame of 1 ½ to 2 years available to surcharge the embankments (and possibly up to 3 years for the WBL embankments). The heights of surcharge that can be applied is, however, limited by stability considerations (particularly with Profile 'A') and because of this, at some sections of the site little or no surcharge can be applied, all at once, especially in areas where the soil is weakest and show a tendency to undergo large settlements. In other words, where relatively weaker soils prevail, they are also expected to undergo greatest settlements and consequently high surcharges are required. But weak soils are unable to accommodate high surcharges, since in these areas the embankments are also relatively high, and thus combination of relatively high embankments and surcharge loads can cause failures.

Our calculations show that in these much weaker areas, some limited surcharge can be applied but in stages under engineering supervision by providing monitoring of pore-pressure and settlements through instrumentation. With these considerations, the following procedure is proposed, assuming that the embankments will be built at least two years ahead of asphalt paving of the highway.

<u>Profile 'A'</u> (also see Drawings 5AA and 5BB):

- From the north end of the site to Station 11+850, no surcharge is required provided that the embankment is built to its final elevation at least one year ahead of paving.
- At Station 11+850, start applying surcharge to reach 1.5 m (above the final pavement elevation) at Station 11+880.
- From Station 11+880 to Station 12+200, maintain 1.5 m of surcharge.
- At Station 12+200, start reducing the surcharge to reach 1.0 m at Station 12+400. Increase surcharge from 1.0 m to 1.6 m after 6 months, subject to monitoring.
- From Station12+400, reduce the surcharge further to reach 0.5 m* at Station 12+470, with a view to place another 0.5 m after six months and another 0.6 m after a further 6-month period (i.e. total surcharge of 1.6 m), subject to monitoring.
- From Station 12+470 to Station12+530, maintain 0.5 m* of surcharge. Increase surcharge to 1.0 m after 6 months, and to a total of 1.6 m after another 6 months, subject to monitoring.

- At Station 12+530, start reducing the surcharge to reach 0.3 m* (above the final pavement elevation) at Station 12+550, with a view of placing an additional 0.5 m of surcharge after six months and another 0.6 m after a further 6 months, subject to monitoring. Allowance should be made to place slightly higher surcharge (i.e. up to 0.3 m more) after the first 6 to 12 months should this be possible through monitoring.

- At Station 12+550, starts reducing the initial surcharge to zero at Station 12+600. From Station 12+600 to Station 12+860, maintain zero surcharge, with a view of placing 0.3 m after 3 to 6 months, and another 0.3 m after another 3 to 6 months, and another 0.7 m after a further six months period, subject to monitoring. Allowance should be made to place higher surcharge (i.e. up to 0.3 m more), should this be possible through monitoring during this period.

- At Station 12+860, gradually increase surcharge to reach 0.5 m* at Station 12+900 and maintain this 0.5 m* surcharge to Station 13+050, with a view to place an additional 0.5 m after six months and another 0.5 m after a further six months period (i.e. total surcharge of 1.5 m) subject to monitoring.

- From Station 13+050 to Station13+200, maintain 0.5 m* of surcharge with a view of placing another 0.5 m after six months and a further 0.5 m after another six months, gradually reducing the total surcharge to about 1.0 m at Station 13+200.

- At Station 13+200, gradually increase the initial surcharge from 0.5 to 1.0 m at Station 13+230, with a view to place an additional 0.5 m after six months, gradually reaching the total surcharge height of 1.0 m at Station13+230.

- From Station 13+230 to south end, maintain 1.0 m of surcharge.

<u>Profile 'B'</u> (also see Drawings 5CC and 5DD):

Surcharging programme for Profile 'B'** (which is 0.6 m lower than Profile 'A') is similar but less stringent, as presented in Table 5.5.1.3.

* A period of at least four weeks of embankment load should be allowed prior to placing the first surcharge of 0.3 to 0.5 m.

**A period of at least four weeks of embankment load should be allowed prior to placing the first surcharge of 0.3 to 0.6 m.

Table 5.5.1.3 Recommended Surcharging Programme on Highway 17 (New)

Section (Station to Station)	Profile 'A'		Profile 'B' (0.6 m lower than Profile 'A')	
	Surcharge Programme (also see Drawings 5AA and 5BB)	Maximum Height of Surcharge (m)	Surcharge Programme (also see Drawings 5CC and 5DD)	Maximum Height of Surcharge (m)
North End to 11+850	No surcharge required, provided that embankment is built to its final elevation at least one year ahead of paving.	0	No surcharge required, provided that embankment is built to its final elevation at least one year ahead of paving.	0
11+850 to 11+880	At Station 11+850, start applying surcharge to reach 1.5 m (above the final pavement elevation) at Station 11+880	1.5	At Station 11+850, start applying surcharge to reach 1.0 m (above the final pavement elevation) at Station 11+880	1.0
11+880 to 12+200	Maintain 1.5 m of surcharge	1.5		
11+880 to 11+910			At Station 11+880, gradually increase surcharge to reach 1.5 m at Station 11+910	1.5
11+910 to 12+200			Maintain 1.5 m of surcharge	1.5

12+200 to 12+400	At Station 12+200, start reducing the surcharge to reach 1.0 m at Station 12+400. Increase surcharge from 1.0 m to 1.6 m after 6 months, subject to monitoring	1.6	At Station 12+200, start reducing the surcharge to reach 1.0 m at Station 12+400. Increase surcharge from 1.0 m to 1.6 m after 6 months, subject to monitoring	1.6
12+400 to 12+470	From Station 12+400, reduce the surcharge further to reach 0.5 m* at Station 14+470, with a view to place another 0.5 m after six months and another 0.6 m after a further 6-month period (total surcharge of 1.6 m) subject to monitoring	1.6		
12+470 to 12+530	Maintain 0.5 m* of initial surcharge. Increase surcharge to 1.0 m after 6 months, and to a total of 1.6 m after another 6 months, subject to monitoring	1.6		
12+400 to 12+530			Maintain 1.0 m of surcharge. Increase surcharge to 1.6 m after 6 months, subject to monitoring	1.6
12+530 to 12+550	At Station 12+530, start reducing the initial surcharge to reach 0.3 m* (above the final pavement elevation) at Station 12+550, with a view of placing an additional 0.5 m of surcharge after 6 months and another 0.6 m after a further 6 months, subject to monitoring. Allowance should be made to place slightly higher surcharge (i.e. up to 0.3 m more) should this be possible through monitoring	1.4	At Station 12+530, start reducing the initial surcharge to reach 0.3 m* (above the final pavement elevation) at Station 12+550, with a view of placing an additional 0.5 m of surcharge after 6 months and another 0.6 m after a further 6 months, subject to monitoring. Allowance should be made to place slightly higher surcharge (i.e. up to 0.3 m more) should this be possible through monitoring	1.4

* A period of at least four weeks of embankment load should be allowed prior to placing the first surcharge of 0.3 to 0.6 m.

12+550 to 12+860	At Station 12+550, start reducing the initial surcharge to zero at Station 12+600. From Station 12+600 to 12+860, maintain zero surcharge, with a view of placing 0.3 m after 3 to 6 months, and another 0.3 m after another 3 to 6 months, and another 0.7 m after a further six months period, subject to monitoring. Allowance should be made to place higher surcharge (i.e. up to 0.3 m more), should this be possible through monitoring during this period	1.3	Maintain 0.3 m* surcharge, with a view of placing 0.5 m after 6 months, and another 0.6 m after another 6 months period. Allowance should be made to place higher surcharge (i.e. up to 0.3 m more), should this be possible through monitoring during this period	1.4
12+860 to 13+050	At Station 12+860, gradually increase surcharge to reach 0.5 m* at Station 12+900 and maintain this 0.5 m* surcharge to Station 13+050, with a view to place an additional 0.5 m after 6 months and another 0.5 m after a further 6 months period, subject to monitoring	1.5	At Station 12+860, gradually increase surcharge to reach 0.6 m* at Station 12+900 and maintain this 0.6 m* surcharge to Station 13+050, with a view to place an additional 0.5 m after 6 months and another 0.5 m after a further 6 months period, subject to monitoring	1.6
13+050 to 13+200	Maintain 0.5 m* of surcharge with a view of placing another 0.5 m after 6 months and a further 0.5 m after another 6 months period at Station 13+050. However, gradually reduce the total surcharge to about 1.0 m at Station 13+200 during the placement of the final surcharge.	1.5	Maintain 0.6 m* of surcharge with a view of placing another 0.5 m after 6 months and a further 0.5 m after another 6 months period at Station 13+050. However, gradually reduce the total surcharge to about 1.0 m at Station 13+200 during the placement of the final surcharge.	1.6

* A period of at least four weeks of embankment load should be allowed prior to placing the first surcharge of 0.3 to 0.6 m.

13+200 to 13+230	At Station 13+200, gradually increase the initial surcharge from 0.5 to 1.0 m at Station 13+230, with a view to place an additional 0.5 m after 6 months, gradually reaching the total surcharge height of 1.0 m at Station13+230	1.0	At Station 13+200, gradually increase the initial surcharge from 0.6 to 1.0 m at Station 13+230, with a view to place an additional 0.4 m after 6 months, gradually reaching the total surcharge height of 1.0 m at Station13+230	1.0
13+230 to South End	Maintain 1.0 m of surcharge	1.0	Maintain 1.0 m of surcharge	1.0

The staging of the surcharging is recommended because in our opinion the clay will experience a reduction in pore pressures and an increase in shear strength and that this will be sufficient to accommodate additional loadings without a failure. As mentioned in Section 4.5.1.6 of this report, a comparison of measured undrained shear strengths by Field Vane tests shows relatively higher shear strengths under the existing embankment of Highway 638, as depicted in Figure C5-3 of Appendix C5. This also leads us to believe that the shear strength of the clay will increase in time enabling the application of additional surcharge loads. It is, however, highly recommended that instrumentation be provided to further monitor and verify (and if necessary modify) this aspect.

The estimated additional residual settlements after a two-year surcharging period are presented in Table 5.5.1.4

Table 5.5.1.4 Estimated Maximum Post construction Settlements on Highway 17(New)

Section (Station to Station)	Profile 'A'		Profile 'B'	
	WBL Embankment (mm)	EBL Embankment (mm)	WBL Embankment (mm)	EBL Embankment (mm)
11+800 to 11+850	0	35	0	20
11+850 to 12+200	0	185	0	50
12+200 to 12+530	280	335	40	100
12+530 to 12+730	550	600	280	320
12+730 to 12+920	450	320	230	140
12+920 to 13+050	90	45	40	25
13+050 to 13+200	30	30	30	30
13+200 to 13+400	20	20	20	20

In areas where the residual settlements are expected to be in excess of 0.2 m, consideration can be given to maintaining slightly higher profiles (i.e. equal to about one-half of the anticipated residual settlements) so that when the settlement takes place over the following ten years or so its effects are reduced and that the design grades are more closely maintained. The actual figures should, however, be reviewed during the surcharging period, based on settlement observations. We believe, however, that post construction settlements of the order of 0.3 m over about 10 year period should not adversely affect the performance of the pavement of the highway in view of the flexible pavement design.

From a settlement point of view (as well as stability), Profile 'B' presents a much superior alternative in comparison with Profile 'A', based on reliability and economics.

Another alternative would be the use of wick drains to accelerate the consolidation settlements in the central sections of the site (between about Stations 12+550 and 12+900), where high residual settlements can be expected. In choosing the depth of wick drains, the probability of an artesian condition needs to be considered. Based on the available data, we recommend that the depth of the wick drains be limited to 14 m below the existing grades. The cost of the wick drains at 1.8 m spacing and extending to about 14 m depth of embedment per 100 m of 4-lane highway is expected to be of the order of $200,000. With this approach, the residual settlements are expected to be decreased by 30% after a surcharging period of two years.

Another alternative would be to use light weight fill which will reduce the magnitude of anticipated settlements.

All the alternatives presented above are comparable in performance. However, we recommend that surcharging be adopted, based on cost and to a certain extent on reliability for the performance of the highway.

In any event, it is recommended that surcharging be carried out with proper instrumentation for monitoring settlements. We also recommend that the surcharge application be monitored by means of piezometers to measure excess pore water pressures. It is furthermore recommended that surcharge be placed gradually (i.e. in 0.3 to 0.5 m thick lifts, starting from one end of the site and proceeding to the other end), to allow excess pore pressures to dissipate.

Embankments should be provided with a widened cross-section to allow for settlements of the underlying soils and any possible future minor grade raises. In this case, we recommend that the road platform should be widened by at least 2 m on each side of the centerline (total 4 m). Between Stations 12+530 and 12+920, platform widening by 3.5 m for Profile 'A' and by 3.0 m for Profile 'B' on each side of the roadway is recommended due to the anticipated relatively greater settlements in this section. This is also in accordance with the Northern Region Engineering Directive NRE-98-200. In addition, certain amount of regrading may be necessary due to settlements, including the side slopes after the surcharging period.

The use of rock fill is recommended to build the embankments from the north end to Station 13+050 and normal earth fill beyond this station, except at the existing Highway 638 (Station 12+740 to 12+780 WBL, and Station 12+790 to 12+835 EBL) where earthfill is recommended. Proper transitions should be provided at the interface between the earthfill and rockfill, as per standard MTO procedures. This can be provided by proper chinking of the rockfill or provision of a suitable geotextile separator (Class II, non-woven with FOS of 50 to 100 μm).

5.5.1.3 SELECTION OF VERTICAL ALIGNMENT

Our analyses indicate that the presence of very deep weak and compressible clay deposit preclude the application of sufficient surcharge to effect a major portion of consolidation settlements within the period available for surcharging prior to paving. This is particularly true between about Stations 12+500 and 13+040 Highway 17 (New) where the surficial clay is relatively weaker and the proposed embankments are relatively higher. As mentioned before, the heights of the surcharge that can be applied is limited by stability considerations and because of this, little or no surcharge can be applied all at once in this weak section. In view of this, relatively smaller settlements will be pre-induced and, therefore, larger post construction settlements can be expected, as presented in Table 5.5.1.4. The table shows that with Profile 'A', post-construction settlements of up to 600 mm is expected to occur within the following 30 years, with about 60% of this value (approximately 360 mm) is anticipated to occur within about 10 years. With Profile 'B', which is 0.6 m lower than Profile 'A', the anticipated settlements are much lower (i.e., up to 320 mm after 30 years and about 200 mm after 10 years following the surcharge period). For this reason, and the reasons stated above, Profile 'B' is the preferred choice.

- Existing Highway 638 traffic can be maintained with minimal disruption from the construction of Highway 17 (New) and it can be utilized as detour, since there will be no grade raise in this section of the road.
- Embankment heights are reduced and therefore rockfill requirements in the swamp section are also reduced.
- Settlements are reduced due to lower embankment load.
- It is possible that, depending on monitoring results, higher surcharges can be applied to effect and expedite consolidation settlements resulting in reduced post construction settlements.

- Relatively higher factor of safety is obtained which gives more confidence on the stability of the embankments.

Some minor concerns due to lowering of the grades from Profile 'A' to 'B' are as follows:

- The revised finished grades of the highway are closer to the prevailing water level of the swamp. We understand, however, that the revised proposed grades (approximate Elevation 179.0 m) are about 1.0 m above the 100-year storm level (Elevation 177.9 m) and that this is sufficient from hydrological point of view. With the aid of instrumentation, allowance can be made for future settlements to ensure that the future grades can be maintained reasonably close to the final grades.

- Between Stations 11+680 and 11+840, the area is in cut by up to about 3 m (from the original proposed centerline grade), and lowering will increase the height of cut even further. The groundwater level in this area, at the time of the investigation, was at the original proposed grade and lowering the grades will bring the excavation even lower below the water table. This can, however, be alleviated by providing gravel sheeting.

Based on the arguments presented above, we recommend that vertical Profile 'B' be adopted.

5.5.1.4 CONSTRUCTION

For embankment construction, all organic and other unsuitable soils should be removed and replaced with suitable materials as per OPSD 203.010 for embankments over swamp. Within the median of the highway, any remaining organics beyond the toes of the embankment need not be removed.

The materials used for the construction of the embankment should consist of approved, acceptable earth or rock fill. The earth fills above the water table should be placed in lifts not exceeding 300 mm (except for the first lift which may have to be increased to about 600 mm) before compaction and each lift of earth fill should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density (SPMDD) (OPSS 206 and OPSS 501). Below the water table, earth fill should consist of granular material (e.g., Granular 'B').

Proper erosion control measures for earthfill embankments should be implemented both during construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

The type of material to be used for soil replacement (i.e. to replace the stripped soils to the existing original ground surface) should be compatible with the proposed embankment fills. For example, where rock fill is to be used for embankment construction, the existing soils beneath the existing ground surface should be replaced with rock fill. In this case, it is important that the rock fill be penetrated the exposed subgrade where the subgrade consists of weak clay (i.e. very soft to firm consistency) and the size of the rock fill should be limited to 400 mm.

In case of rockfill, the rock fill construction should be as per OPSS 206, except it should be placed in lifts with thickness not exceeding 1.5 m. The rock fill should be compacted by overlapping track prints of the construction equipment. Depending on the size and type of equipment used, six to eight passes along each path should be required. The final surface of rock fill material should be compacted by at least two additional passes and should be blinded with compacted fine fill material (or chinked) prior to installation of the road subbase layer.

Placement in layers and compaction is not required for rock fill to be placed under water. Heavy equipment, such as fully-loaded haul truck, should not be allowed within about 10 m of the crest of the rockfill slope to avoid instability of the excavation. At any time, the access ramp and side slopes should not be steeper than 4H : 1V slope. This slope could be modified to 3H : 1V if this is found to be stable in the field. To minimize the mixing of clay subgrade and rock fill along the edges of the subexcavation, a separation of about 1 m between the advancing rock fill and edge of the excavation could be provided.

In any case, the rock fill should be compacted by overlapping track prints of the construction equipment after backfilling to about 0.3 m above the original ground level. Depending on the size and type of equipment used, at least ten passes along each path should be required at this level.

Rock fill material of maximum nominal size of 400 mm could be used and should consist of pieces of hard and durable rock with no sign of decomposition. Concrete, masonry, brick and similar materials should not be used.

A geotextile separator is recommended between rock fill and any native soil surcharge in order to prevent infiltration of fine soils into the rock fill. Any silty soils left at the surface of the rock fill, after the surcharge is removed, could be potential cause of possible frost heaving of the pavement. The separator should comprise of a Class II non-woven geotextile as per OPSS 1860 with a Filtration Opening Size (FOS) of 50 to 100 μm or 150 mm of Granular 'B' layer.

As the removal of the rockfill after the surcharging period may not be practical, the use of granular subbase material may be preferable (e.g., Granular 'B') rather than rockfill for the surcharge materials.

The groundwater table throughout much of the site was at or near the existing ground surface. This aspect should be taken into consideration when carrying out stripping and backfilling. Dewatering may be required to facilitate these tasks.

Between Stations 12+545 and 13+040, where 2.0 m deep sub-excavation is recommended, the measured undrained shear strengths of the soil indicate that basal heave could occur. Based on this, it is recommended that the excavation be carried out in short sections with side slopes no steeper than 3H:1V (e.g., in 4 to 5 m strips) and should be backfilled expeditiously with rockfill.

For the construction of the Highway 17 (New) embankments, we understand that traffic on the existing Highway 638 has to be maintained. From the findings of two boreholes (Boreholes 12+747 Lt and 12+772 CL), the existing Highway 638 embankment consists of compacted sand and gravel fill, underlain by clay deposit interbedded with silt to sandy silt soils. No peat or significant organic soil was encountered at these locations. Based on these, the existing granular embankment can remain and additional granular fill can be placed on top of the embankment (after removing the asphalt). However, the removal of organic soils adjacent to this embankment should be carried out such that the stability of the existing embankment is ensured during construction. This could be achieved by stripping the organic materials in short section (e.g., about 4 m wide strips) perpendicular to the embankment and immediately backfilling with suitable materials such as rock fill. The rockfill to be placed adjacent to the granular embankment should be fine graded (maximum 100 mm in size) and becoming coarse within about 5 m from the existing embankment to act as filter and minimize migration of fines into the voids of the rockfill, especially below the groundwater table. Alternatively, a geotextile separator, as described above, could be used. Above the water table, proper chinking of the rockfill could be implemented prior to placement of the granular fill over the rockfill.

Based on the thickness of the clay deposit and the presence of silt layers at shallow depths, majority of the settlements of these upper deposits (e.g., top 4 to 6 m) are expected to be complete once the full height of the embankment is reached.

If vertical alignment 'A' will be adopted for Highway 17 (New), this will require that additional 0.6 m of fill will be added over the existing Highway 638. In this instance, it is suggested that a temporary detour be constructed to the south of the existing Highway 638 during fill placement and should be surfaced with granular materials for ease of maintaining the road when excessive settlements occur during construction. However, if alignment 'B' is to be used (which is recommended) then the existing grades at Highway 638 will be maintained and therefore, Highway 638 could be utilized as detour during the construction of the Highway 17 (New) embankments.

We understand that spill containment berm will be constructed at the toe of the embankment within the swamp area. We also understand that positive drainage under the embankment will be provided when the spill containment berm is constructed. Based on our slope stability analysis, there is no instability of the highway embankment caused by the choice of material for constructing the spill containment berm.

5.5.2 SWAMP SECTION ALONG HIGHWAY 638

5.5.2.1 EMBANKMENT STABILITY

Along the proposed Highway 638 in this swamp section, as shown on Drawing No. 5E, the height of proposed embankment with vertical Profile 'C' (original proposed grades) ranges from about 0.2 m at Station 11+500, gradually increasing to about 2.3 m at Station 11+744. From thereon, the proposed embankment height gradually reduces to about 0.2 m at about Station 11+865, and eventually matching existing grade at Station 11+930 at Pioneer Road.

A second proposed vertical alignment (Profile 'D') reduces the height of embankments along this alignment. The reduction in the height of the embankment ranges from less than 0.1 m at Station 11+530 to about 0.5 m at Station 11+750 and then reducing to match Profile 'C'.

Details of the proposed embankment heights are given in Appendix E5-1 and also on the profiles in Drawings 5E and 5F.

Similar to Highway 17 (New) in this swamp section, the stability of the embankment on Highway 638 was analyzed by the limit equilibrium method, utilizing the computer program Slope/W.

In order to reduce the magnitude of the post construction settlements, as will be discussed in the next section, and to come up with an optimum design, various depths of sub-excavation of the very weak subgrade soils along with various combinations of surcharge loadings were examined at various select borehole locations.

In our analyses, surcharge loads of between 0.2 and 1.6 m of earth were also added to the embankment loadings, in order to deal with the settlement issues, as will be discussed in the next section of this report.

The soil parameters used in the slope stability analyses are presented in Table 5.5.2.1.

Table 5.5.2.1 Soil Parameters Used in Slope Stability Analyses – Highway 638

Soil Type	Short-Term Analysis			Long-Term Analysis		
	ϕ (degrees)	c (kPa)	γ (kN/m^3)	ϕ' (degrees)	c' (kPa)	γ (kN/m^3)
Embankment Fill (select subgrade material)	30	0	21.5	30	0	21.5
Embankment Fill (Rock Fill)	43	0	18.0-20.0	43	0	18.0-20.0
Rock Backfill (used to replace existing peat and other surficial unsuitable soils)	36	0	18.0-20.0	36	0	18.0-20.0
Surficial Silty Sand	30	0	20.0	30	0	20.0
Sand and Gravel	35	0	21.5	35	0	21.5
Clay	0	4-86	15.0	20-24	0-2	15.0
Silt to Sandy Silt	26	0	20.5	26	0	20.5

Typical embankment slope stability sections are presented in Appendices E5-4 and E5-5.

Since the proposed Highway 17 (New) will be intersecting the proposed Highway 638, we recommend the use of rock fill throughout this section. As mentioned before, this is because rock fill will penetrate the upper zone of the very soft clay, thus strengthening it. In addition, in our opinion, rock fill will provide a better performance of the highway under low embankment situations where the subgrade is of very low shear strength. In all cases 3H:1V and/or 4H:1V side slopes were assumed as this is desirable for low embankments.

In our analyses, we assumed that all the existing peat, topsoil and any other unsuitable materials will be removed and will be replaced with suitable materials as per MTO procedures for embankments over swamp (OSPD 203.010).

The analyses results show rock fills with 3H:1V side slopes generally provide a minimum factor of safety 1.40 for the proposed height of embankments along Profile 'D'. For rock fills with 4H:1V side slopes along Profile 'C', with a few exceptions where in several cases the factor of safety is about 1.3, they generally provide a minimum factor of safety 1.40 for the proposed height of embankments. In both cases, the following stripping/subexcavation procedures (Table 5.5.2.2) were considered.

From the stability point of view, Profile 'D' is preferable in comparison with Profile 'C' as it reduces the possibility of occasional minor failures of the embankments during construction due to the presence of very soft clay subgrade.

Table 5.5.2.2 Amount of Required Stripping and/or Subexcavation on Highway 638

Section (Station to Station)	Stripping / Subexcavation for Profile 'C'	Stripping / Subexcavation for Profile 'D'
11+570 to 11+730	At Station 11+570, start increasing depth of stripping to reach 1.0 m at Station 11+600 + 0.5 m rock penetration	At Station 11+570, start increasing depth of stripping to reach 1.0 m at Station 11+600 + 0.5 m rock penetration
11+730 to 11+740	At Station 11+730, start increasing the sub-excavation to reach 2.0 m at Station 11+740 + 0.5 m rock penetration	At Station 11+730, start increasing the sub-excavation to reach 2.0 m at Station 11+740 + 0.5 m rock penetration
11+740 to 11+830	Remove minimum 2.0 m of soil + 0.5 m rock penetration into clay subgrade	Remove minimum 2.0 m of soil + 0.5 m rock penetration into clay subgrade
11+830 to 11+835	At Station 11+830, start decreasing sub-excavation to reach normal organic soil removal	At Station 11+830, start decreasing sub-excavation to reach normal organic soil removal
11+835 to end (PR17)	Normal organic soil removal	Normal organic soil removal

5.5.2.2 SETTLEMENT OF EMBANKMENTS

Our analysis, using consolidation test results presented in Figures B5-12 through B5-15, B5-22 and B5-23, shows that maximum settlements of the order of 0.5 m (Profile 'C') and 0.3 m (Profile 'D') can be expected over a 10-year period of time, under the stresses imposed by the embankments, while near the west and east ends of the site, the anticipated settlements are less than 0.15 m, and will take place within relatively shorter periods of time. One way of alleviating the settlement problem is to surcharge the embankments. We understand that there is a time frame of 1½ to 2 years available to surcharge the embankments. The height of surcharge that can be applied is, however, limited by stability considerations (particularly with Profile 'C') and because of this, at some sections of the site little or no surcharge can be applied, all at once, especially in areas where the soil is weakest and show a tendency to undergo large settlements. In other words, where relatively weaker soils prevail, they are also expected to undergo greatest settlements and consequently high surcharges are required. But weak soils are unable to accommodate high surcharges, since in these areas the embankments are also relatively high, and thus combination of relatively high embankments and surcharge loads can cause failures.

Our slope stability analyses show that in these much weaker areas, some limited surcharge can be applied but in stages under engineering supervision. With these considerations, the surcharge programme for Profile 'C' presented in Table 5.5.2.3 is proposed, assuming that the embankments will be built at least two years ahead of asphalt paving of the highway.

Surcharging programme for Profile 'D' (which is up to 0.5 m lower than Profile 'C') is similar but less stringent, as presented in Table 5.5.2.3.

The surcharge programmes for Profiles 'C' and 'D' are also presented graphically in Drawings 5EE and 5FF, respectively.

Table 5.5.2.3 Recommended Surcharging Programme for Highway 638

Section (Station to Station)	Profile 'C'		Profile 'D' (up to 0.5 m lower than Profile 'A')	
	Surcharge Programme (also see Drawing 5EE)	Maximum Height of Surcharge (m)	Surcharge Programme (also see Drawing 5FF)	Maximum Height of Surcharge (m)
11+480 to 11+500	At Station 11+480, start applying surcharge to reach 1.0 m (above the final pavement elevation, right of C/L) at Station 11+500	1.0	At Station 11+480, start applying surcharge to reach 1.0 m (above the final pavement elevation, right of C/L) at Station 11+500	1.0
11+500 to 11+560	Maintain 1.0 m of surcharge (right side of C/L without impacting traffic)	1.0	Maintain 1.0 m of surcharge (right side of C/L without impacting traffic)	1.0
11+560 to 11+580	At Station 11+560, start reducing the surcharge to reach 0.3 m at Station 11+580. Increase surcharge from 0.3 m to 0.6 m after 6 months, and from 0.6 m to 1.0 m after another 6 months	1.0	At Station 11+560, start reducing the surcharge to reach 0.3 m at Station 11+580. Increase surcharge from 0.3 m to 0.6 m after 6 months, and from 0.6 m to 1.0 m after another 6 months	1.0
11+580 to 11+620	Maintain 0.3 m* of initial surcharge. Increase surcharge to 0.6 m after 6 months, and to a total of 1.0 m after another 6 months	1.0		

11+580 to 11+700			Maintain 0.3 m* of initial surcharge. Increase surcharge to 0.6 m after 6 months, and to a total of 1.0 m after another 6 months	1.0
11+620 to 11+700	At Station 11+620, start reducing the initial surcharge to zero at Station 11+640. From Station 11+640 to 11+700, maintain zero surcharge, with a view of placing 0.3 m after 6 months, and another 0.7 m (for a total of 1.0 m) after another 6 months	1.0		
11+700 to 11+720	At Station 11+700, gradually increase surcharge from zero to reach 0.5 m* at Station 11+720, with a view to place an additional 0.5 m after 6 months for a total of 1.0 m at Station 11+720	1.0	At Station 11+700, gradually increase surcharge to reach 0.5 m* at Station 11+720, with a view to place an additional 0.5 m (for a total of 1.0 m) after 6 months	1.0
11+720 to 11+740	Maintain 0.5 m* of initial surcharge, with a view to place an additional 0.5 m of after 6 months. At Station 11+720, gradually increase final surcharge to reach a total of 1.5 m at Station 11+740 after another 6 months	1.5	Gradually increase initial surcharge to 0.6 m* to Station 11+740, with a view to place an additional 0.5 m after 6 months. At Station 11+720, gradually increase final surcharge to reach a total of 1.6 m at Station 11+740 after another 6 months	1.6

11+740 to 11+830	Maintain 0.5 m* of initial surcharge. Increase surcharge to 1.0 m after 6 months, and to a total of 1.5 m after another 6 months, subject to monitoring (as recommended under Highway 17 New)	1.5	Maintain 0.6 m* of initial surcharge. Increase surcharge to 1.1 m after 6 months, and to a total of 1.6 m after another 6 months, subject to monitoring (as recommended under Highway 17 New)	1.6
11+830 to 11+850	Maintain 0.5 m* of surcharge with a view of placing another 0.5 m after 6 months and a further 0.5 m after another 6 months period at Station 11+830. However, gradually reduce the total surcharge to about 1.0 m at Station 11+850 during the placement of the final surcharge.	1.5	Gradually reduce initial surcharge to 0.5 m* to Station 11+850, with a view of placing another 0.5 m after 6 months and a further 0.5 m after another 6 months period at Station 11+830. However, gradually reduce the total surcharge to about 1.0 m at Station 11+850 during the placement of the final surcharge.	1.6
11+850 to 11+900	At Station 11+850, gradually decrease the initial surcharge from 0.5 to zero at Station 11+900, with a view to place an additional 0.5 m after 6 months, gradually reaching the total surcharge height of zero at Station11+900	1.0	At Station 11+850, gradually increase the initial surcharge from 0.5 to zero at Station 11+900, with a view to place an additional 0.5 m after 6 months, gradually reaching the total surcharge height of zero at Station11+900	1.0

* A period of at least four weeks of embankment load should be allowed prior to placing the first surcharge of 0.3 to 0.6 m.

Similar to Highway 17 (New), the staging of the surcharging is also recommended on Highway 638 because in our opinion the clay will experience a reduction in pore pressures and an increase in shear strength and that this will be sufficient to accommodate additional loadings without a failure.

The estimated additional residual settlements along Highway 638 after a two-year surcharging period are presented in Table 5.5.2.4

Table 5.5.2.4 Estimated Maximum Post construction Settlements – Highway 638

Station to Station	Profile 'C' (mm)	Profile 'D' (mm)
11+480 to 11+560	5	5
11+560 to 11+580	20	11
11+580 to 11+720	50	45
11+720 to 11+830	40	35
11+830 to 11+900	20	5

From a settlement point of view, Profile 'D' is relatively comparable with Profile 'C' but presents a better alternative, based on reliability and economics.

For Profile 'C', it is recommended that surcharging be carried out with proper instrumentation for monitoring settlements. We also recommend that the surcharge application be monitored by means of piezometers to measure excess pore water pressures.

In any event, it is recommended that the surcharge be placed gradually (i.e. in 0.3 to 0.5 m thick lifts, starting from one end of the site and proceeding to the other end), to allow excess porewater pressure to dissipate.

Embankments should be provided with a widened cross-section to allow for settlements of the underlying soils and any possible future minor grade raises. In this case, we recommend that the road platform should be widened by at least 2 m on each side of the centerline (total 4 m). In addition, certain amount of regrading may be necessary due to settlements, including the side slopes after the surcharging period.

5.5.2.3 SELECTION OF VERTICAL ALIGNMENT

Our analyses indicate that the presence of very deep weak and compressible clay deposit preclude the application of sufficient surcharge to effect a major portion of consolidation settlements within the period available for surcharging prior to paving. As mentioned before, the height of the surcharge that can be applied is limited by stability considerations and because of this, little or no surcharge can be applied all at once in this weak section. Table 5.5.2.4 shows that post construction settlements of up to 50 mm is expected to occur within the following 10 years, which in our opinion is acceptable for this road.

Based on the arguments presented above, both profiles are comparable but we recommend that vertical Profile 'D' be adopted. In any event, it is anticipated that Profile 'D' will likely be used since the profile for Highway 638 will be governed by the recommended profile for Highway 17 (New) (i.e., Profile 'B') and therefore Profile 'C' becomes redundant.

5.5.2.4 CONSTRUCTION

For embankment construction, all organic and other unsuitable soils should be removed and replaced with suitable materials as per OPSD 203.010 for embankments over swamp.

The materials used for the construction of the embankment should consist of approved, acceptable earth or rock fill. The earth fills above the water table should be placed in lifts not exceeding 300 mm (except for the first lift which may have to be increased to about 600 mm) before compaction and each lift of earth fill should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density (SPMDD) (OPSS 206 and OPSS 501). Below the water table, earth fill should consist of granular material (e.g., Granular 'B').

Proper erosion control measures for earthfill embankments should be implemented both during construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

The type of material to be used for soil replacement (i.e. to replace the stripped soils to the existing original ground surface) should be compatible with the proposed embankment fills. For example, where rock fill is to be used for embankment construction, the existing soils beneath the existing ground surface should be replaced with rock fill. In this case, it is important that the rock fill be penetrated the exposed subgrade where the subgrade consists of weak clay (i.e. very soft to firm consistency) and the size of the rock fill should be limited to 400 mm.

In case of rockfill, the rock fill construction should be as per OPSS 206, except it should be placed in lifts with thickness not exceeding 1.5 m. The rock fill should be compacted by overlapping track prints of the construction equipment. Depending on the size and type of equipment used, six to eight passes along each path should be required. The final surface of rock fill material should be compacted by at least two additional passes and should be blinded with compacted fine fill material (or chinked) prior to installation of the road subbase layer.

Placement in layers and compaction is not required for rock fill to be placed under water. Heavy equipment, such as fully-loaded haul truck, should not be allowed within about 10 m of the crest of the rockfill slope to avoid instability of the excavation. At any time, the access ramp and side slopes should not be steeper than 4H : 1V slope. This slope could be modified to 3H : 1V if this is found to be stable in the field. To minimize the mixing of clay subgrade and rock fill along the edges of the subexcavation, a separation of about 1 m between the advancing rock fill and edge of the excavation could be provided.

In any case, the rock fill should be compacted by overlapping track prints of the construction equipment after backfilling to about 0.3 m above the original ground level. Depending on the size and type of equipment used, at least ten passes along each path should be required at this level.

Rock fill material of maximum nominal size of 400 mm could be used and should consist of pieces of hard and durable rock with no sign of decomposition. Concrete, masonry, brick and similar materials should not be used.

A geotextile separator is recommended between rock fill and any native soil surcharge in order to prevent infiltration of fine soils into the rock fill. Any silty soils left at the surface of the rock fill, after the surcharge is removed, could be potential cause of possible frost heaving of the pavement. The separator should comprise of a Class II non-woven geotextile as per OPSS 1860 with a Filtration Opening Size (FOS) of 50 to 100 μm or 150 mm of Granular 'B' layer.

As the removal of the rockfill after the surcharging period may not be practical, the use of granular subbase material may be preferable (e.g., Granular 'B') rather than rockfill for the surcharge materials.

The groundwater table throughout much of the site was at or near the existing ground surface. This aspect should be taken into consideration when carrying out stripping and backfilling. Dewatering may therefore be required to facilitate these tasks.

Drawings

℄ PROFILE 'A' EASTBOUND LANES

SCALES
10m 0 10 20m HORIZ
1m 0 1 2m VERT

NOTE:
BOREHOLES 95-PH1,95-PH2 DONE BY GOLDER ASSOCIATES LTD.
LOCATIONS ARE VERY APPROXIMATE.

NOTE:
FOR DETAILED SUBSURFACE CONDITIONS OF ALL BOREHOLES REFER TO RECORD OF BOREHOLE SHEETS.
FOR DETAILED SUBSURFACE CONDITIONS OF BOREHOLE C2-A,C2-B (STATION 13+072), 13+068 Rt REFER TO RECORD OF BOREHOLE SHEETS.

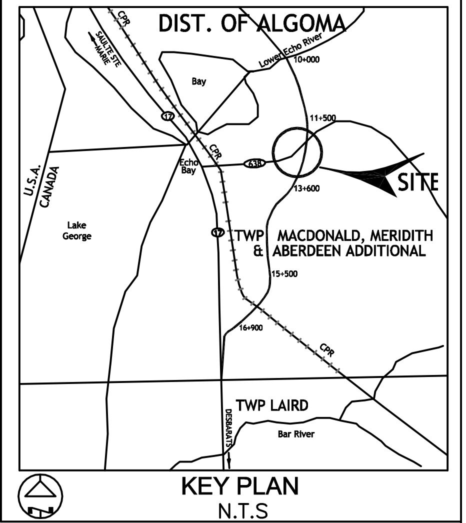
METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

CONT No.
GWP: 354-94-00

HIGHWAY 17 (NEW) EBL
ECHO RIVER TO BAR RIVER ROAD
SITE No. 5
BORE HOLE LOCATIONS & SOIL STRATA

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

- Bore Hole
- Dyanmic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Cu Undrained Shear Strength measured by Field Vane Test
- Water Level at Time of Investigation Feb. and Mar. ,2002
- Water Level in Piezometer
- Piezometer
- Artesian
- Bore Hole Done By Golder Associates Ltd.
- Stage 1 Surcharge-immediately after completion of embankment
- Stage 2 Surcharge-3 months after completion of embankment
- Stage 3 Surcharge-6 months after completion of embankment
- Stage 4 Surcharge-1 year after completion of embankment

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office,Downsview. Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

REV.	DATE	BY	DESCRIPTION

Geocres No. 41K00-059

HWY No. 17 (New)			DIST 62
SUBM'D ZO	CHECKED RM	DATE Apr, 2003	SITE
DRAWN JZ	CHECKED	APPROVED	DWG 5AA

℄ PROFILE 'A' WESTBOUND LANES

10m 0 10 20m HORIZ
1m 0 1 2m VERT
SCALES

NOTE:
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FOR DETAILED SUBSURFACE CONDITIONS OF BOREHOLE C2-A,C2-B (STATION 13+072), 13+068 Rt REFER TO RECORD OF BOREHOLE SHEETS.

METRIC

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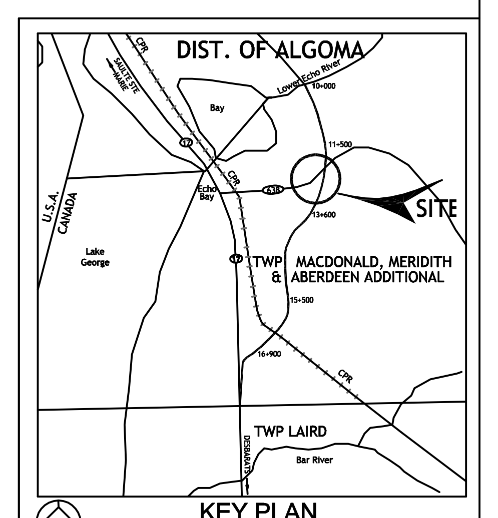
CONT No.
GWP: 354-94-00

HIGHWAY 17 (NEW) WBL
ECHO RIVER TO BAR RIVER ROAD
SITE No. 5
BORE HOLE LOCATIONS & SOIL STRATA

SHAHEEN & PEAKER LIMITEI



KEY PLAN
N.T.S

LEGEND

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- Water Level at Time of Investigation Feb. and Mar. ,2002
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- Piezometer
- Artesian
- Stage 1 Surcharge-immediately after completion of embankment
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- Stage 3 Surcharge-6 months after completion of embankment
- Stage 4 Surcharge-1 year after completion of embankment

NOTE
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REV.	DATE	BY	DESCRIPTION

Geocres No. 41K00-059

HWY No. 17 (New)			DIST 62
SUBM'D ZO	CHECKED RM	DATE Apr, 2003	SITE
DRAWN JZ	CHECKED	APPROVED	DWG 5BB

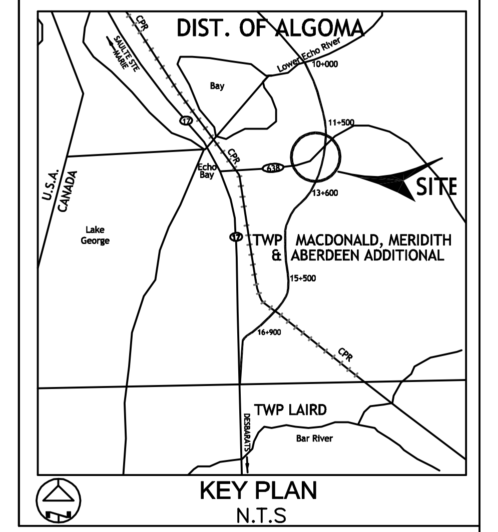
METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

CONT No.
GWP: 354-94-00

HIGHWAY 17 (NEW) EBL
ECHO RIVER TO BAR RIVER ROAD
SITE No. 5
BORE HOLE LOCATIONS & SOIL STRATA

SHAHEEN & PEAKER LIMITE



KEY PLAN
N.T.S

LEGEND

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- Dyanmic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N — Blows/0.3m (Std. Pen. Test, 475 J/blow)
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NOTE

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REV.	DATE	BY	DESCRIPTION

Geocres No. 41K00-059

HWY No. 17 (New)			DIST 62
SUBM'D ZO	CHECKED RM	DATE Apr, 2003	SITE
DRAWN JZ	CHECKED	APPROVED	DWG 5CC

℄ PROFILE 'B' WESTBOUND LANES

10m 0 10 20m HORIZ
1m 0 1 2m VERT
SCALES

NOTE:
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FOR DETAILED SUBSURFACE CONDITIONS OF BOREHOLE C2-A,C2-B (STATION 13+072), 13+068 Rt REFER TO RECORD OF BOREHOLE SHEETS.

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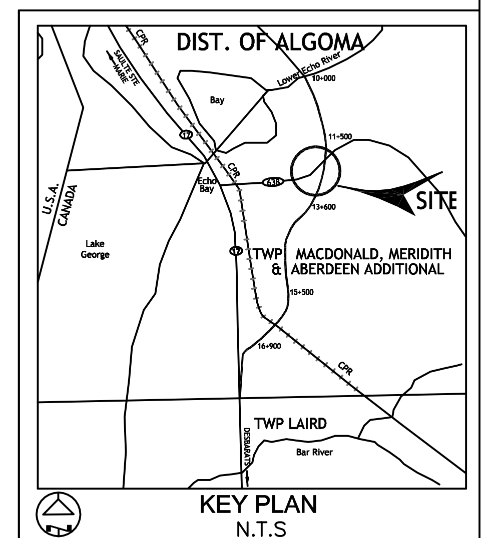
CONT No.
GWP: 354-94-00

HIGHWAY 17 (NEW) WBL
ECHO RIVER TO BAR RIVER ROAD
SITE No. 5
BORE HOLE LOCATIONS & SOIL STRATA

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

- Bore Hole
- Dyanmic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
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NOTE
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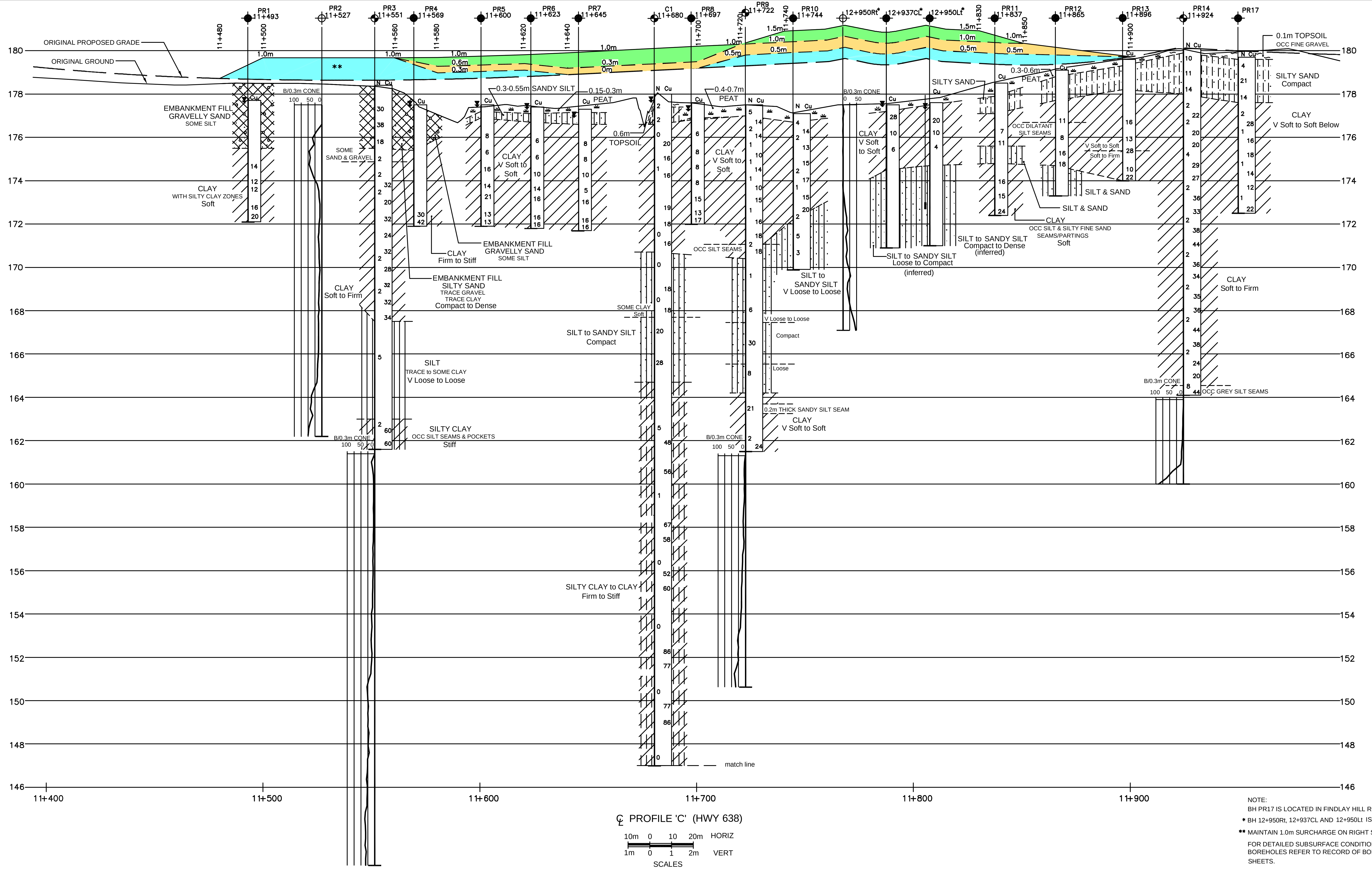
REV.	DATE	BY	DESCRIPTION

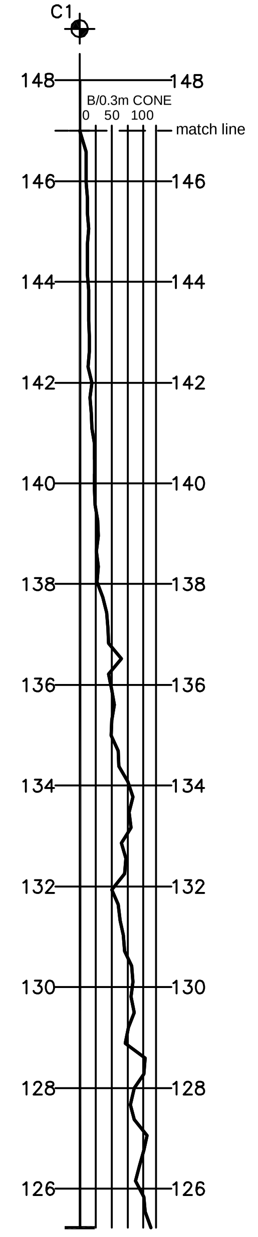
Geocres No. 41K00-059

HWY No. 17 (New)			DIST 62
SUBM'D ZO	CHECKED RM	DATE Apr, 2003	SITE
DRAWN JZ	CHECKED	APPROVED	DWG 5DD



METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.



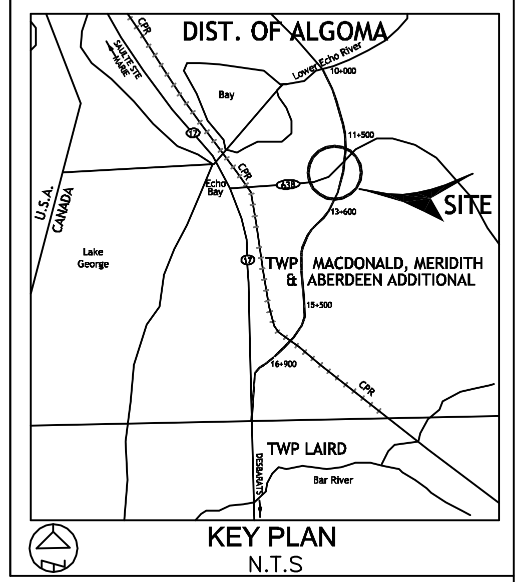
NOTE:
BH PR17 IS LOCATED IN FINDLAY HILL ROAD.
* BH 12+950Rt, 12+937CL AND 12+950Lt IS LOCATED IN HWY 17.
** MAINTAIN 1.0m SURCHARGE ON RIGHT SIDE OF ROAD ONLY.
FOR DETAILED SUBSURFACE CONDITIONS OF ALL BOREHOLES REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.

GWP: 354-94-00

HIGHWAY 638
ECHO RIVER TO BAR RIVER ROAD
BORE HOLE LOCATIONS & SOIL STRATA

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Cu Undrained Shear Strength measured by Field Vane Test
- Water Level at Time of Investigation Mar. , Apr. and Jul ,2002
- Water Level in Piezometer
- Piezometer
- Stage 1 Surcharge-immediately after completion of embankment
- Stage 2 Surcharge-6 months after completion of embankment
- Stage 3 Surcharge-1 year after completion of embankment

NOTE

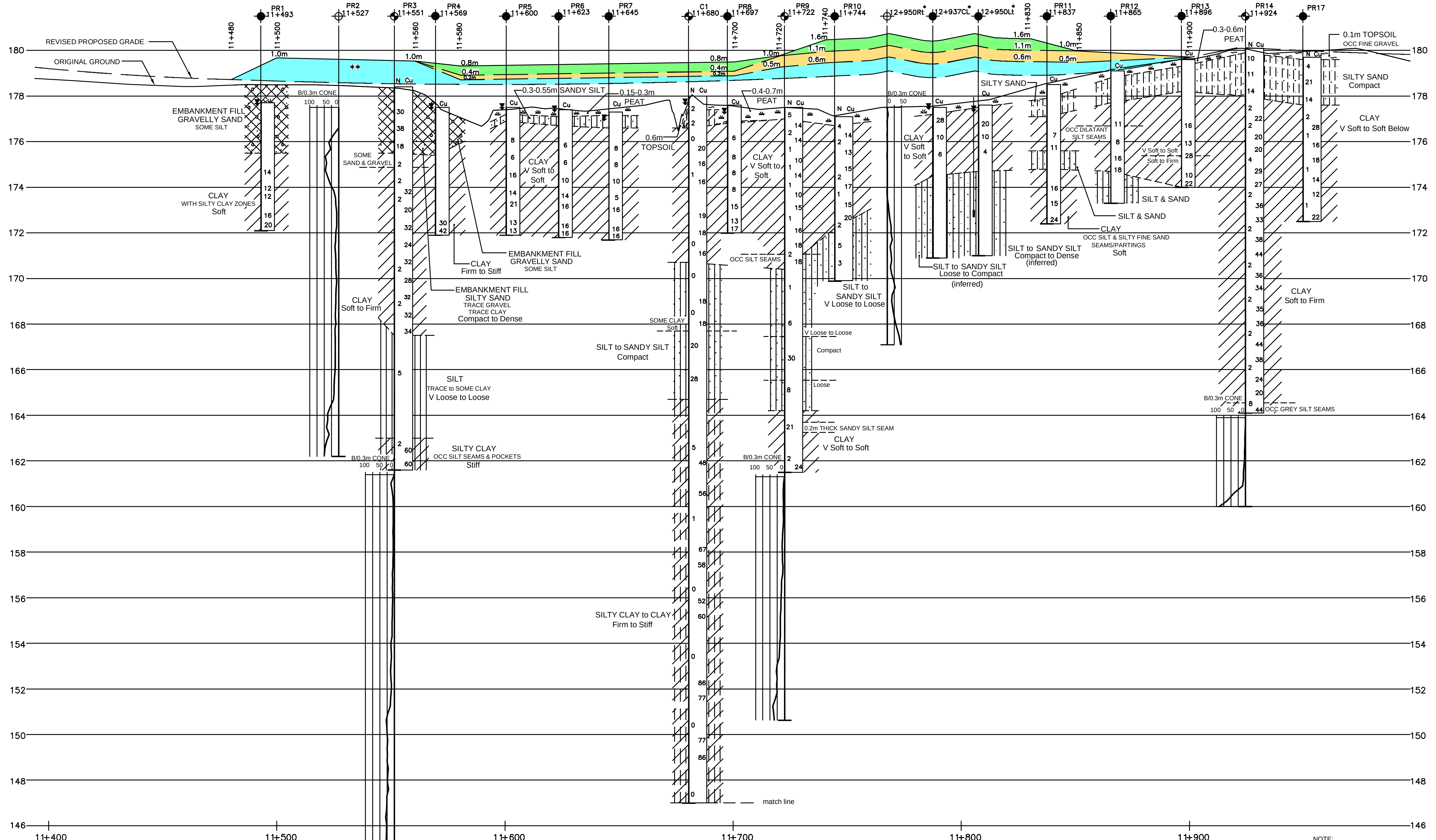
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office,Downsview. Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

REV.	DATE	BY	DESCRIPTION

Geocres No.

HWY No. 638			DIST 62
SUBM'D ZO	CHECKED RM	DATE Jun, 2003	SITE
DRAWN JZ	CHECKED	APPROVED	DWG 5EE

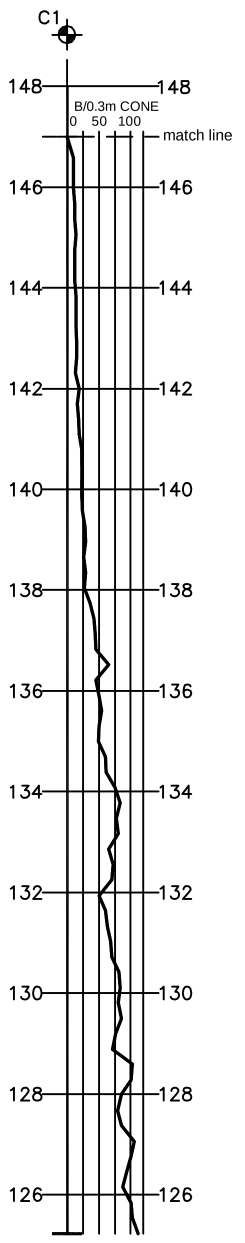


℄ PROFILE 'D' (HWY 638)

10m 0 10 20m HORIZ

1m 0 1 2m VERT

SCALES



NOTE:
BH PR17 IS LOCATED IN FINDLAY HILL ROAD.
* BH 12+950Rt, 12+937CL AND 12+950Lt IS LOCATED IN HWY 17.
** MAINTAIN 1.0m SURCHARGE ON RIGHT SIDE OF ROAD ONLY.
FOR DETAILED SUBSURFACE CONDITIONS OF ALL BOREHOLES REFER TO RECORD OF BOREHOLE SHEETS.

METRIC

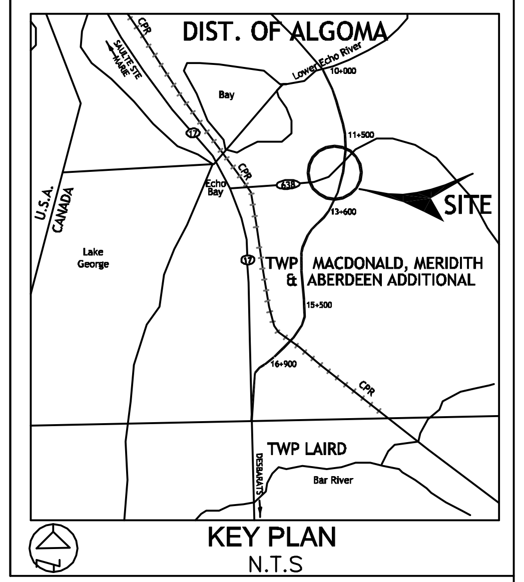
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

CONT No.

GWP: 354-94-00

HIGHWAY 638
ECHO RIVER TO BAR RIVER ROAD
BORE HOLE LOCATIONS & SOIL STRATA

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

- Bore Hole
- Dyanmic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N — Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Cu — Undrained Shear Strength measured by Field Vane Test
- Water Level at Time of Investigation Mar. , Apr. and Jul ,2002
- Water Level in Piezometer
- Piezometer
- Stage 1 Surcharge-immediately after completion of embankment
- Stage 2 Surcharge-6 months after completion of embankment
- Stage 3 Surcharge-1 year after completion of embankment

=NOTE=
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office,Downsview. Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

REV.	DATE	BY	DESCRIPTION

Geocres No.

HWY No. 638			DIST 62
SUBM'D ZO	CHECKED RM	DATE Aug, 2003	SITE
DRAWN JZ	CHECKED	APPROVED	DWG 5FF

APPENDIX E5-1

Proposed Embankment Heights At Station Centreline

Original Proposed Embankment Heights at Station Centreline

Station Number	Proposed Embankment Height (m)	
	Westbound Lane	Eastbound Lane
11+793	2.03 (Cut)	0.48
11+817	0.99 (Cut)	0.56
11+844	0.10	0.83
11+861	0.66	1.36
11+863	0.72	1.41
11+890	1.09	1.48
11+917	1.16	1.63
11+920	1.21	1.64
11+940	1.24	1.68
11+965	1.22	1.62
11+967	1.21	1.63
12+015	1.07	1.81
12+016	0.99	1.81
12+040	1.24	1.89
12+064	1.35	1.72
12+119	1.68	2.07
12+145	1.78	2.13
12+146	1.78	2.14
12+172	1.85	2.13
12+195	1.69	2.07
12+220	1.52	2.15
12+240	1.46	2.15
12+244	1.45	2.14
12+265	1.32	2.20
12+285	1.40	2.05
12+287	1.41	2.05

12+334	1.64	2.19
12+340	1.68	2.16
12+365	1.73	2.12
12+391	1.90	2.29
12+393	1.88	2.30
12+419	2.01	2.31
12+440	2.15	2.33
12+468	2.30	2.43
12+485	2.23	2.41
12+530	2.26	2.42
12+543	2.39	2.23
12+560	2.40	2.46
12+583	2.39	2.47
12+585	2.40	2.48
12+630	2.38	2.50
12+637	2.32	2.41
12+639	2.34	2.35
12+697	2.38	2.45
12+699	2.37	2.45
12+730	1.87	2.48
12+747	0.93	2.50
12+759	0.63	2.45
12+772	0.57	2.27
12+794	2.04	1.63
12+842	2.13	2.10
12+855	2.24	2.11
12+887	2.16	2.18
12+900	2.22	2.16
12+937	1.98	2.17
12+950	2.01	2.10

12+985	2.00	1.90
13+000	1.71	1.43
13+018	1.28	1.40
13+024	1.29	1.46
13+050	1.11	1.39
13+069	1.46	1.45
13+072	1.56	1.44
13+085	1.27	1.52
13+095	1.34	1.60
13+114	1.34	1.77
13+140	1.20	1.54
13+165	1.30	1.43
13+187	1.01	1.34
13+191	0.97	1.29
13+206	0.94	1.09
13+227	0.93	0.87
13+230	0.93	0.87
13+255	0.91	0.95
13+279	1.01	0.92
13+282	1.03	0.89
13+307	1.09	1.09

Revised Proposed Embankment Heights at Station Centreline

Station Number	Proposed Embankment Height (m)	
	Westbound Lane	Eastbound Lane
11+793	2.53 (Cut)	0.00
11+817	1.54 (Cut)	0.00
11+844	0.50 (Cut)	0.23
11+861	0.00	0.76
11+863	0.12	0.81
11+890	0.49	0.88
11+917	0.56	1.03
11+920	0.61	1.04
11+940	0.64	1.08
11+965	0.62	1.02
11+967	0.61	1.03
12+015	0.47	1.21
12+016	0.39	1.21
12+040	0.64	1.29
12+064	0.75	1.12
12+119	1.08	1.47
12+145	1.18	1.53
12+146	1.18	1.54
12+172	1.25	1.53
12+195	1.09	1.47
12+220	0.92	1.55
12+240	0.86	1.55
12+244	0.85	1.54
12+265	0.72	1.60
12+285	0.80	1.45
12+287	0.81	1.45

12+334	1.04	1.59
12+340	1.08	1.56
12+365	1.13	1.52
12+391	1.30	1.69
12+393	1.28	1.70
12+419	1.41	1.71
12+440	1.55	1.73
12+468	1.70	1.83
12+485	1.63	1.81
12+530	1.66	1.82
12+543	1.79	1.63
12+560	1.80	1.86
12+583	1.79	1.87
12+585	1.80	1.88
12+630	1.78	1.90
12+637	1.72	1.81
12+639	1.74	1.75
12+697	1.78	1.85
12+699	1.77	1.85
12+730	1.27	1.88
12+747	0.33	1.90
12+759	0.00	1.85
12+772	0.00	1.67
12+794	1.44	1.03
12+842	1.53	1.50
12+855	1.64	1.51
12+887	1.56	1.58
12+900	1.62	1.57
12+937	1.43	1.62
12+950	1.49	1.58

12+985	1.60	1.50
13+000	1.05	1.17
13+018	1.39	1.11
13+024	1.08	1.25
13+050	1.01	1.29
13+069	1.37	1.40
13+072	1.48	1.39
13+085	1.24	1.50
13+095	1.33	1.59
13+114	1.34	1.76
13+140	1.20	1.54
13+165	1.30	1.43
13+187	1.01	1.34
13+191	0.97	1.29
13+206	0.94	1.09
13+227	0.93	0.87
13+230	0.93	0.87
13+255	0.91	0.95
13+279	1.01	0.92
13+282	1.03	0.89
13+307	1.09	1.09

HWY 638

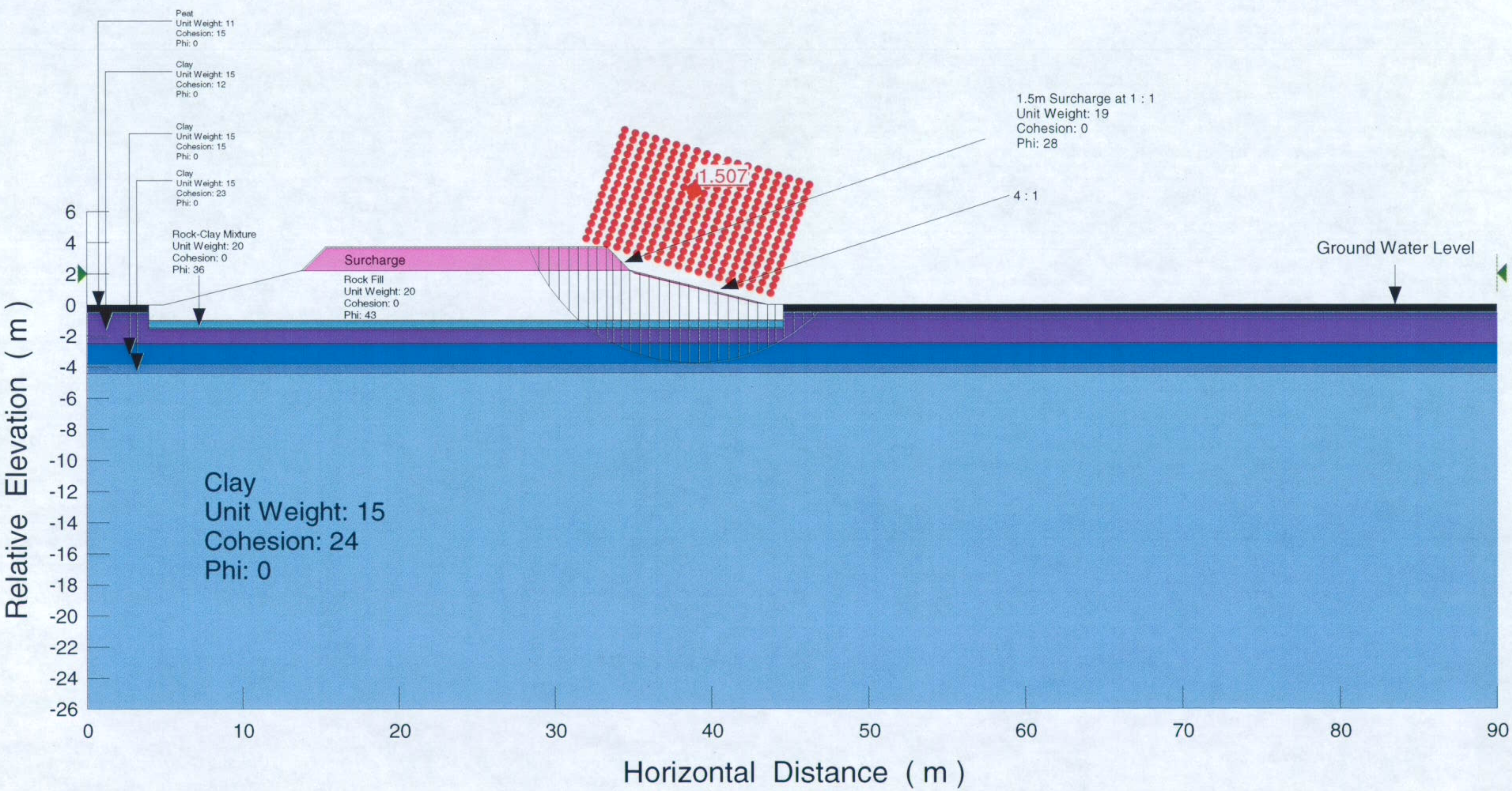
BOREHOLE NO.	STATION NO.	ORIGINAL PROPOSED HEIGHTS (m)	REVISED PROPOSED HEIGHTS (m)	CHANGES (m)
PR1	11+493	0.20	0.19	0.01
PR2	11+527	0.18	0.13	0.05
PR3	11+551	0.29	0.19	0.10
PR4	11+569	0.97	0.83	0.14
PR5	11+600	1.33	1.11	0.22
PR6	11+623	1.38	1.08	0.30
PR7	11+645	1.49	1.12	0.38
C1	11+680	1.25	0.78	0.47
PR8	11+697	1.60	1.10	0.50
PR9	11+722	1.81	1.28	0.53
PR10	11+744	2.28	1.74	0.54
PR11	11+837	0.83	0.37	0.46
PR12	11+865	0.23	0.00	0.23
PR13	11+896	0.07	0.03	0.04
PR14	11+924	0.06	0.06	0.00

APPENDIX E5-2

Highway 17 Slope Stability Analysis Results
(Based on Original Proposed Grade)

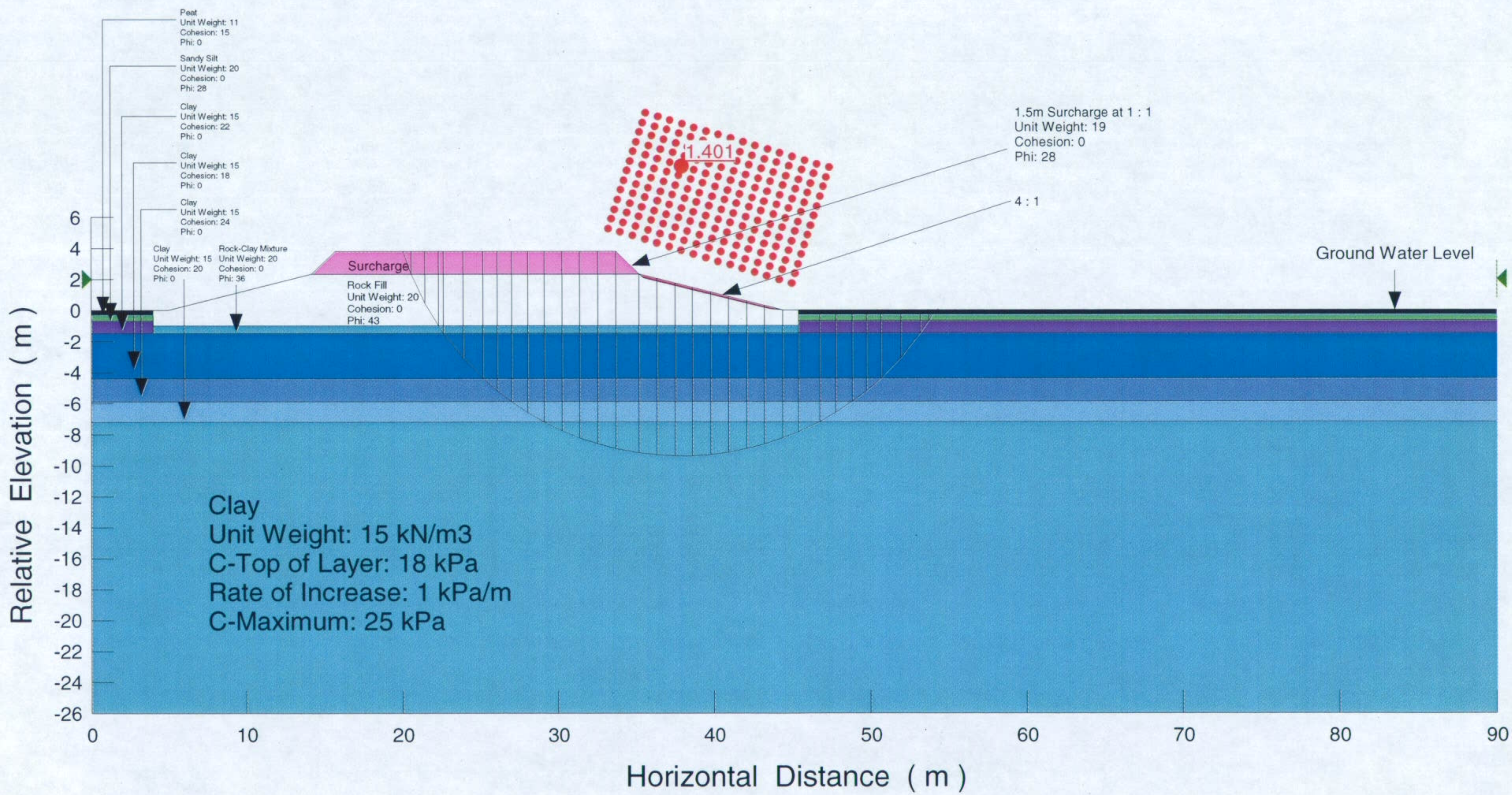
SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, Station 12+240
2.2m High, Rock Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)

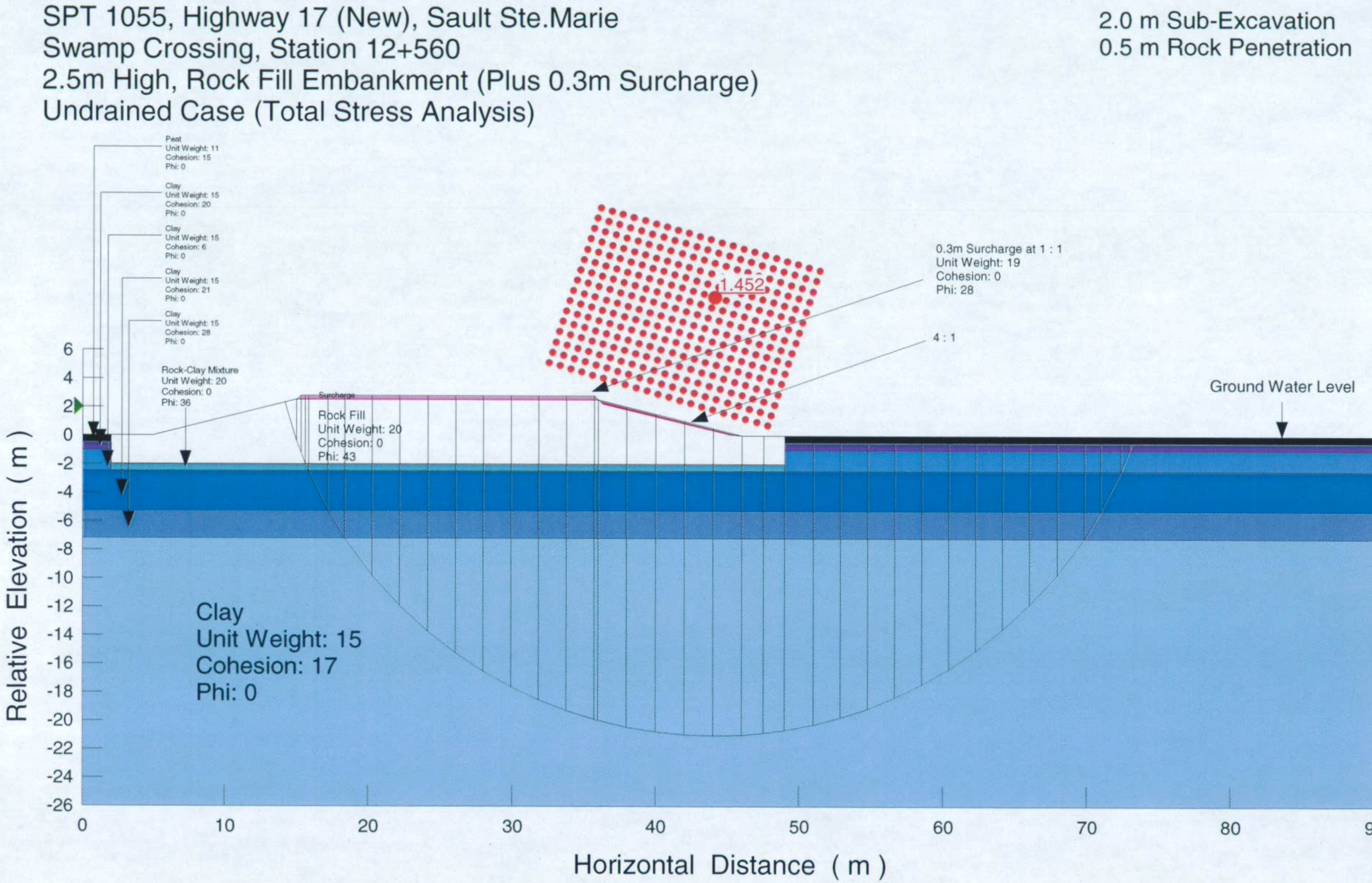
1.0 m Sub-Excavation
0.5 m Rock Penetration



SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, Station 12+391
2.3m High, Rock Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)

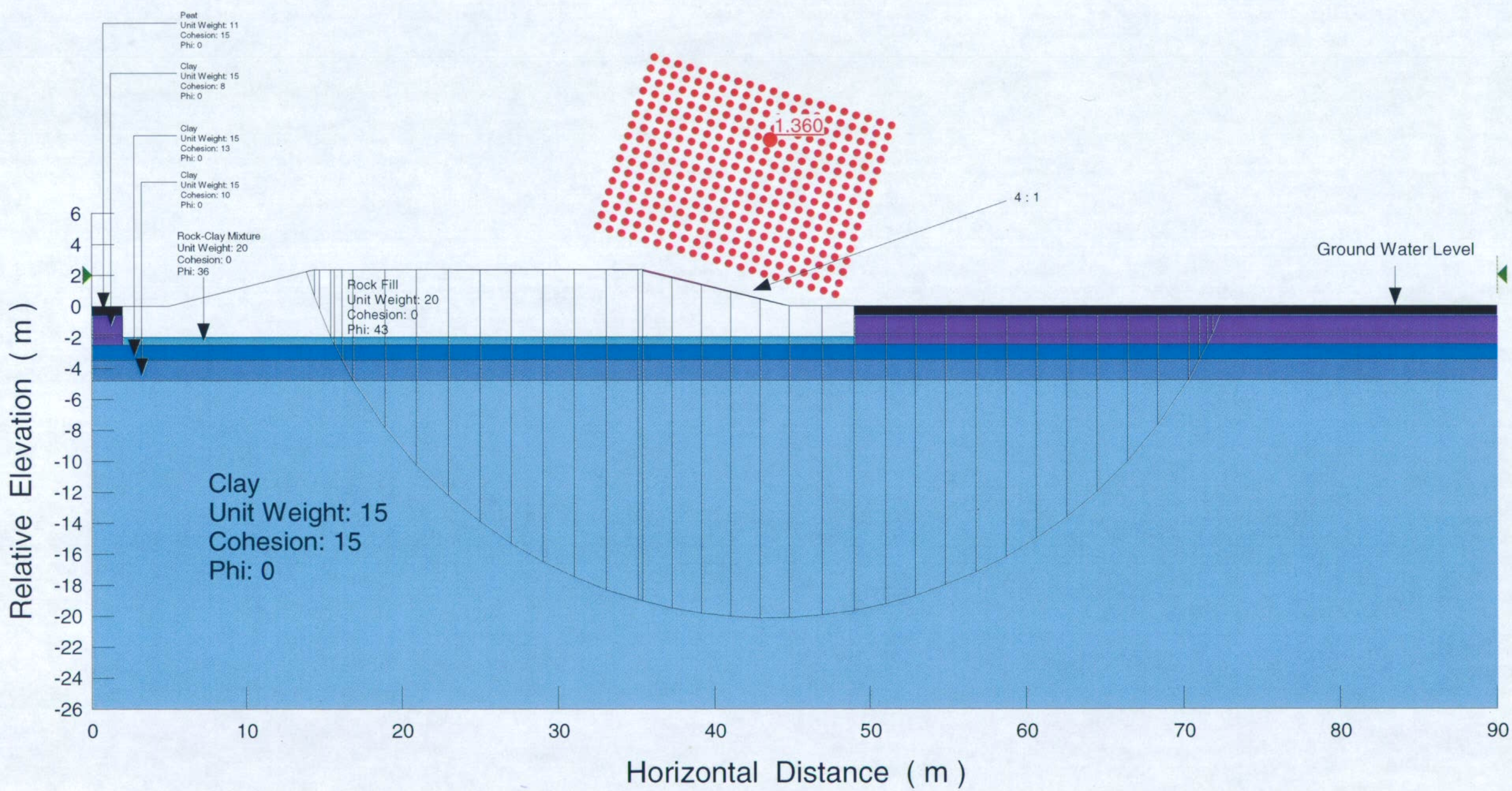
1.0 m Sub-Excavation
0.5 m Rock Penetration




SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, Station 12+560
2.5m High, Rock Fill Embankment (Plus 0.3m Surcharge)
Undrained Case (Total Stress Analysis)
2.0 m Sub-Excavation
0.5 m Rock Penetration
Peat
Unit Weight: 11
Cohesion: 15
Phi: 0
Clay
Unit Weight: 15
Cohesion: 20
Phi: 0
Clay
Unit Weight: 15
Cohesion: 6
Phi: 0
Clay
Unit Weight: 15
Cohesion: 21
Phi: 0
Clay
Unit Weight: 15
Cohesion: 28
Phi: 0
Rock-Clay Mixture
Unit Weight: 20
Cohesion: 0
Phi: 36
Surcharge
Rock Fill
Unit Weight: 20
Cohesion: 0
Phi: 43
1.452
0.3m Surcharge at 1 : 1
Unit Weight: 19
Cohesion: 0
Phi: 28
4 : 1
Ground Water Level
Clay
Unit Weight: 15
Cohesion: 17
Phi: 0
Relative Elevation (m)
Horizontal Distance (m)
6
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
0
10
20
30
40
50
60
70
80
90

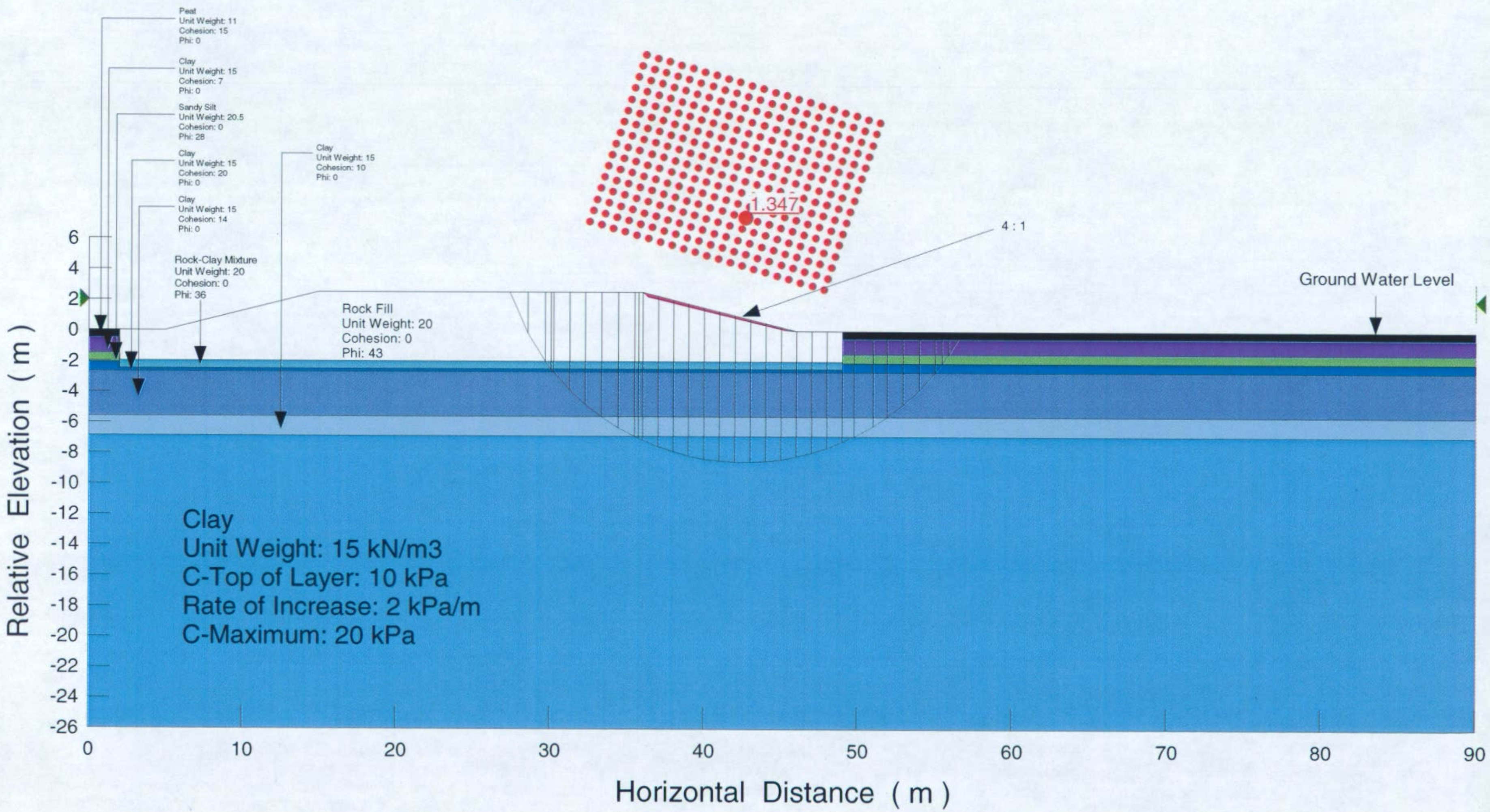
SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, Station 12+637
2.35m High, Rock Fill Embankment (NO Surcharge)
Undrained Case (Total Stress Analysis)

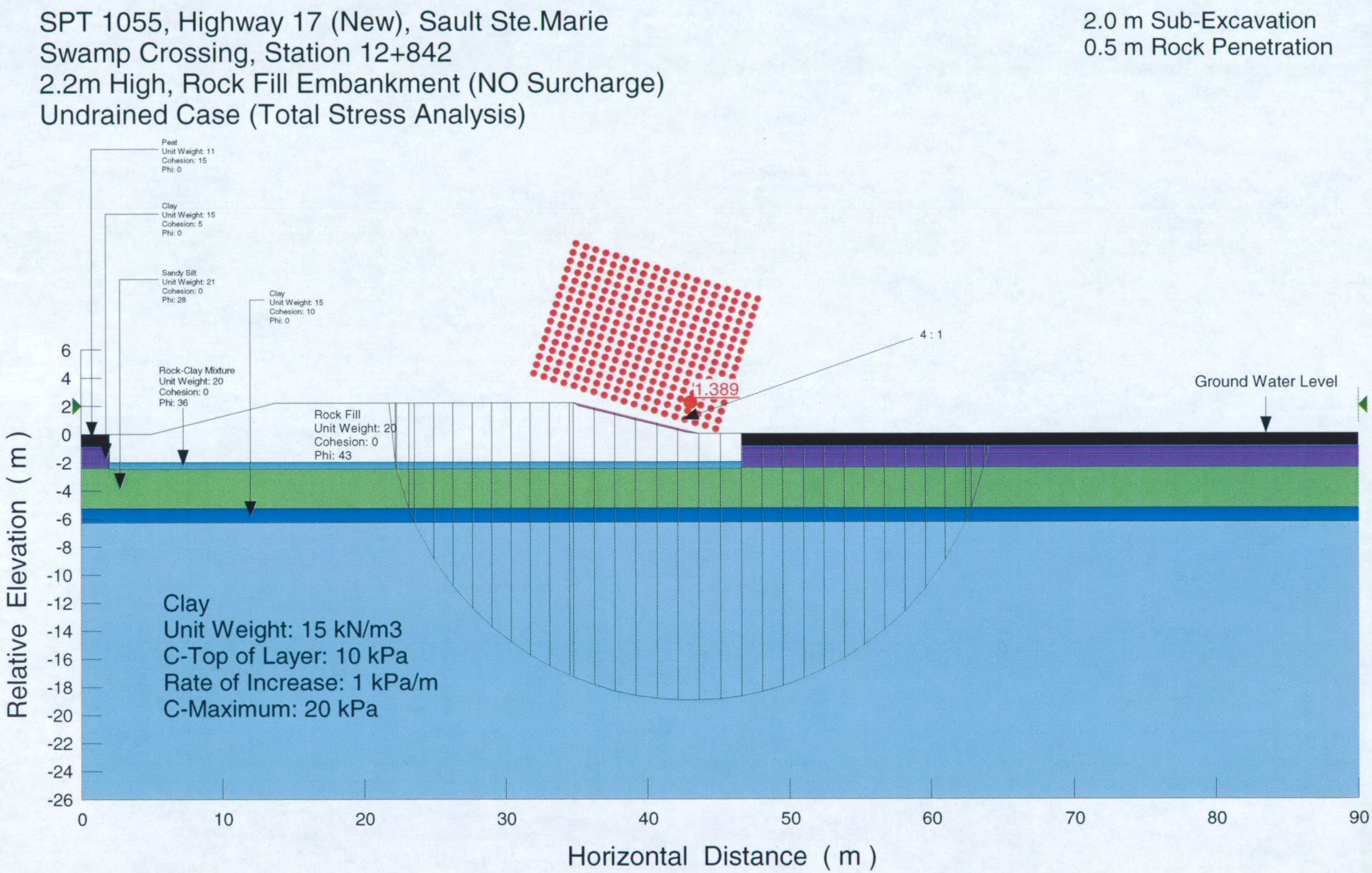
2.0 m Sub-Excavation
0.5 m Rock Penetration



SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, Station 12+730
2.5m High, Rock Fill Embankment (NO Surcharge)
Undrained Case (Total Stress Analysis)

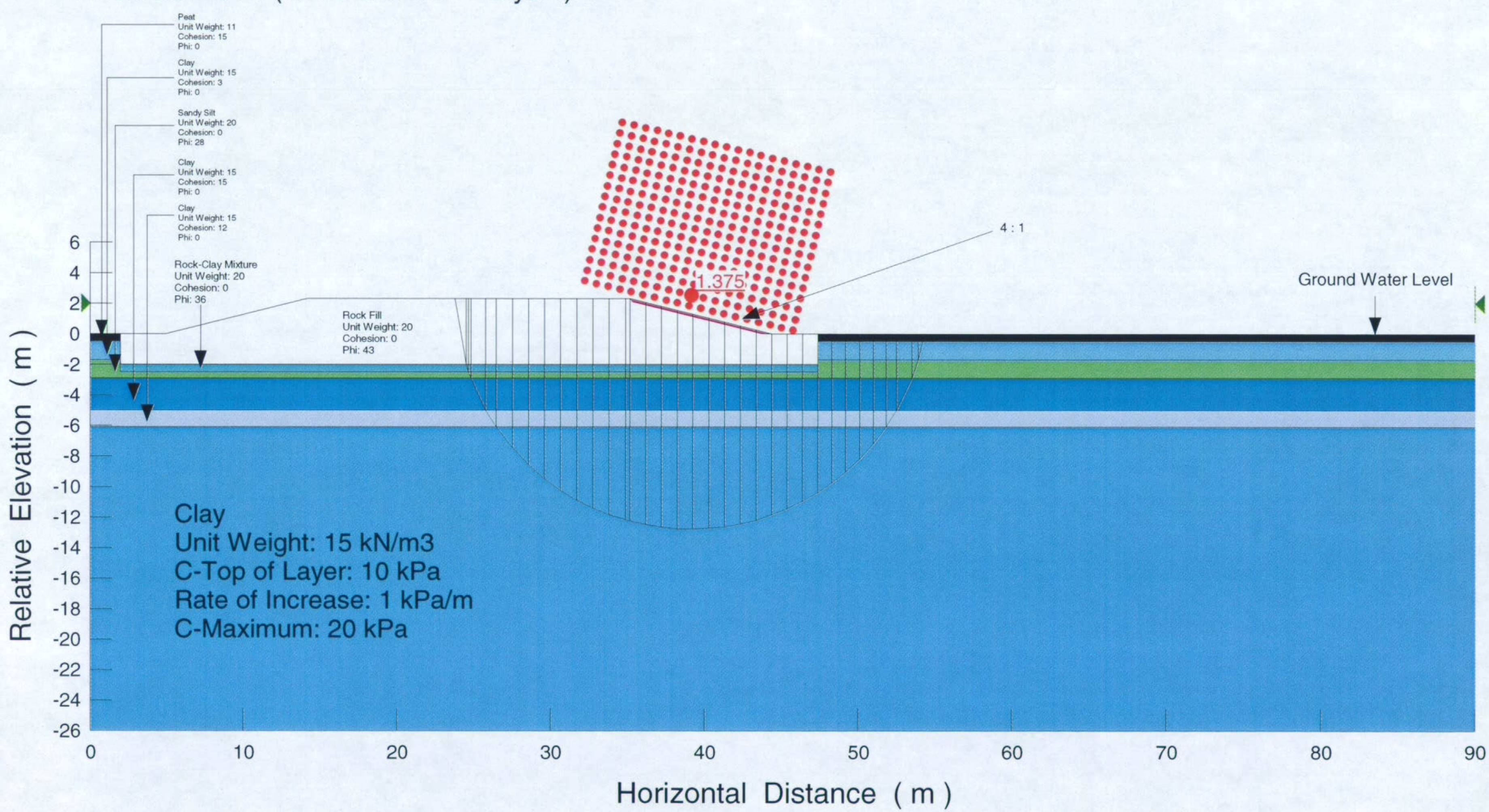
2.0 m Sub-Excavation
0.5 m Rock Penetration




SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, Station 12+842
2.2m High, Rock Fill Embankment (NO Surcharge)
Undrained Case (Total Stress Analysis)
2.0 m Sub-Excavation
0.5 m Rock Penetration
Peat
Unit Weight: 11
Cohesion: 15
Phi: 0
Clay
Unit Weight: 15
Cohesion: 5
Phi: 0
Sandy Silt
Unit Weight: 21
Cohesion: 0
Phi: 28
Clay
Unit Weight: 15
Cohesion: 10
Phi: 0
Rock-Clay Mixture
Unit Weight: 20
Cohesion: 0
Phi: 36
Rock Fill
Unit Weight: 20
Cohesion: 0
Phi: 43
4 : 1
1.389
Ground Water Level
Clay
Unit Weight: 15 kN/m3
C-Top of Layer: 10 kPa
Rate of Increase: 1 kPa/m
C-Maximum: 20 kPa
Relative Elevation (m)
6
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
0
10
20
30
40
50
60
70
80
90
Horizontal Distance (m)

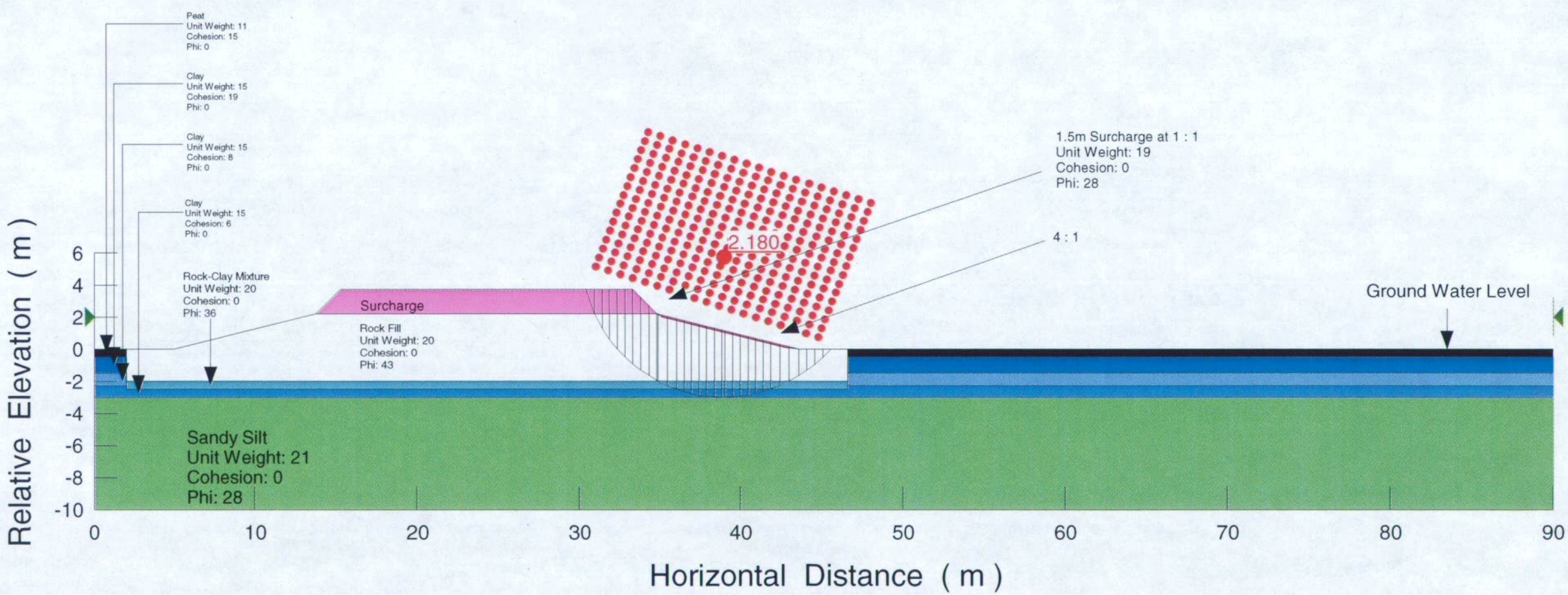
SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, Station 12+855
2.3m High, Rock Fill Embankment (NO Surcharge)
Undrained Case (Total Stress Analysis)

2.0 m Sub-Excavation
0.5 m Rock Penetration



SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, Station 12+937
2.2m High, Rock Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)

2.0 m Sub-Excavation
0.5 m Rock Penetration

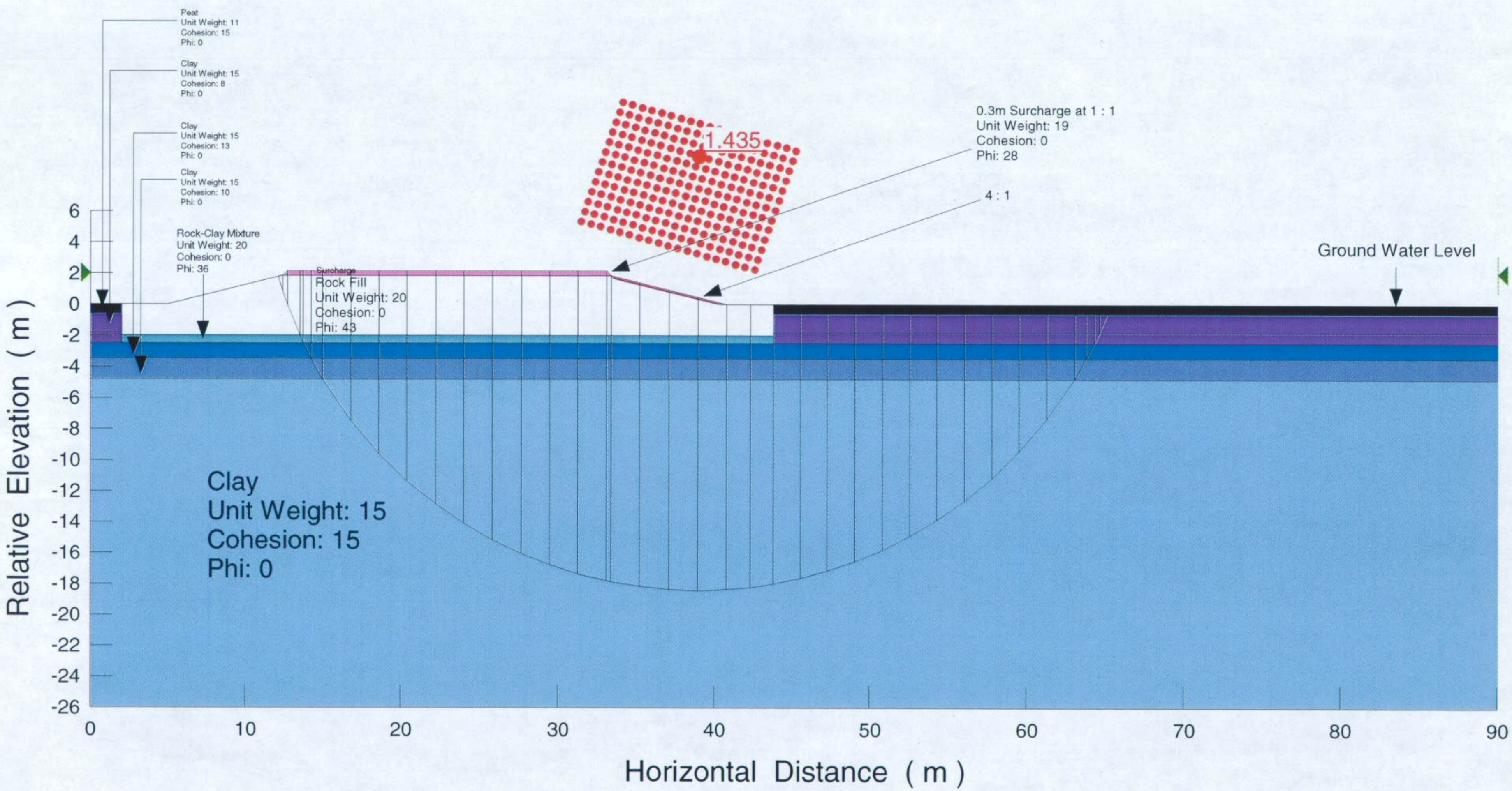


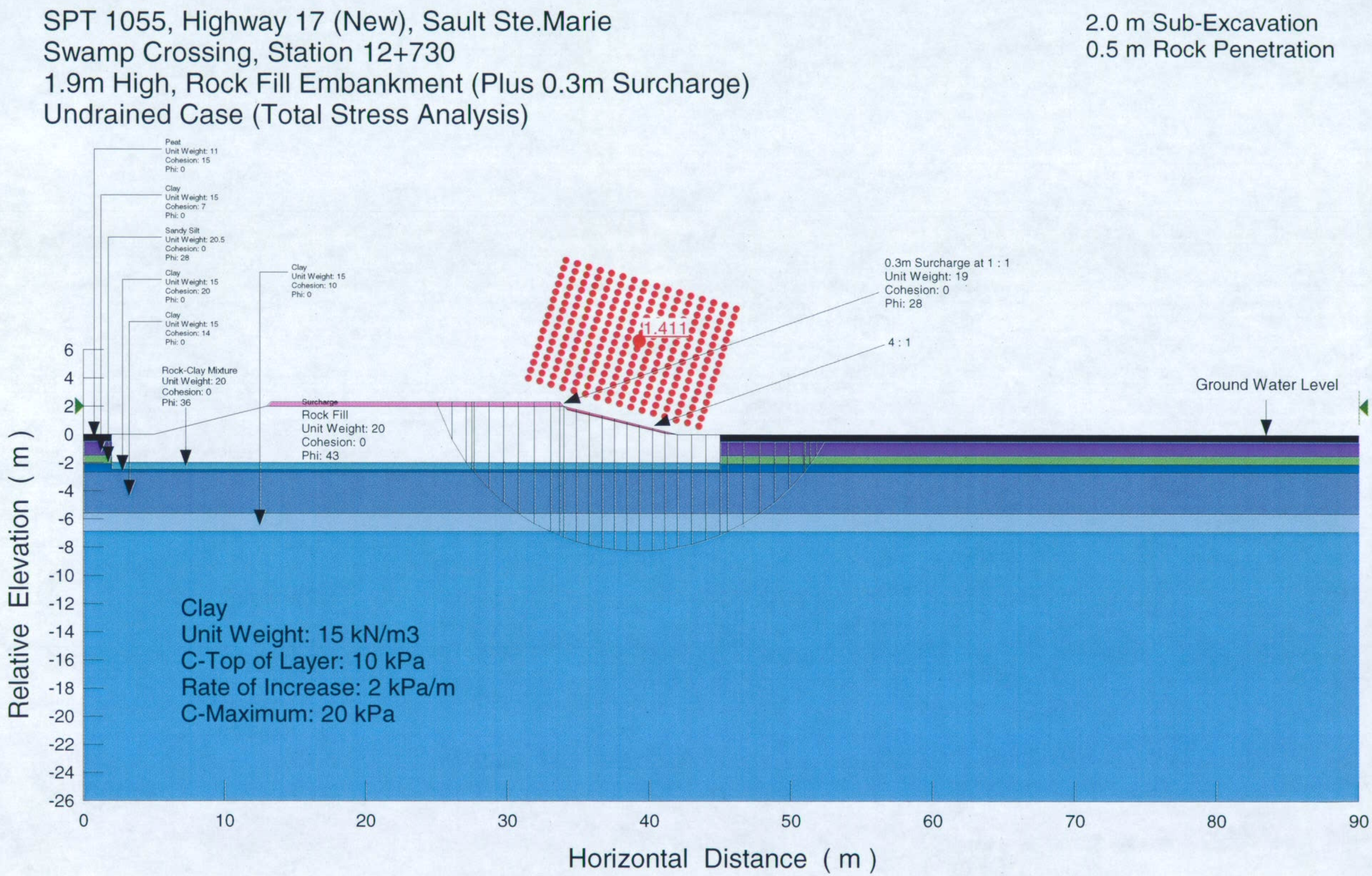
APPENDIX E5-3

Highway 17 Slope Stability Analysis Results (Based on Revised Proposed Grade)

SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, Station 12+637
1.75m High, Rock Fill Embankment (Plus 0.3m Surcharge)
Undrained Case (Total Stress Analysis)

2.0 m Sub-Excavation
0.5 m Rock Penetration

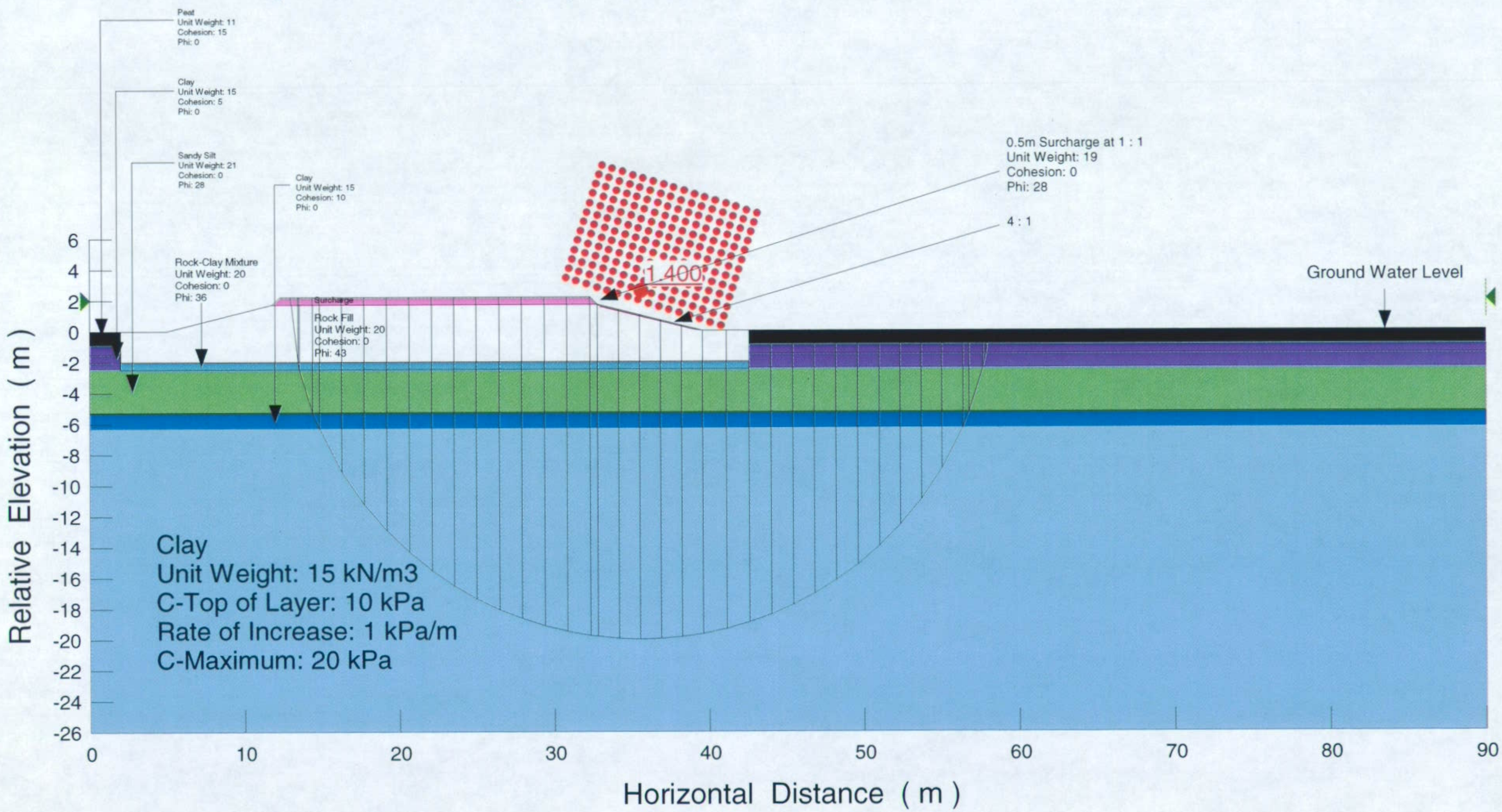




SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, Station 12+730
1.9m High, Rock Fill Embankment (Plus 0.3m Surcharge)
Undrained Case (Total Stress Analysis)
2.0 m Sub-Excavation
0.5 m Rock Penetration
Peat
Unit Weight: 11
Cohesion: 15
Phi: 0
Clay
Unit Weight: 15
Cohesion: 7
Phi: 0
Sandy Silt
Unit Weight: 20.5
Cohesion: 0
Phi: 28
Clay
Unit Weight: 15
Cohesion: 20
Phi: 0
Clay
Unit Weight: 15
Cohesion: 14
Phi: 0
Rock-Clay Mixture
Unit Weight: 20
Cohesion: 0
Phi: 36
Clay
Unit Weight: 15
Cohesion: 10
Phi: 0
Surcharge
Rock Fill
Unit Weight: 20
Cohesion: 0
Phi: 43
1.411
0.3m Surcharge at 1 : 1
Unit Weight: 19
Cohesion: 0
Phi: 28
4 : 1
Ground Water Level
Clay
Unit Weight: 15 kN/m3
C-Top of Layer: 10 kPa
Rate of Increase: 2 kPa/m
C-Maximum: 20 kPa
Relative Elevation (m)
Horizontal Distance (m)
6
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
0
10
20
30
40
50
60
70
80
90

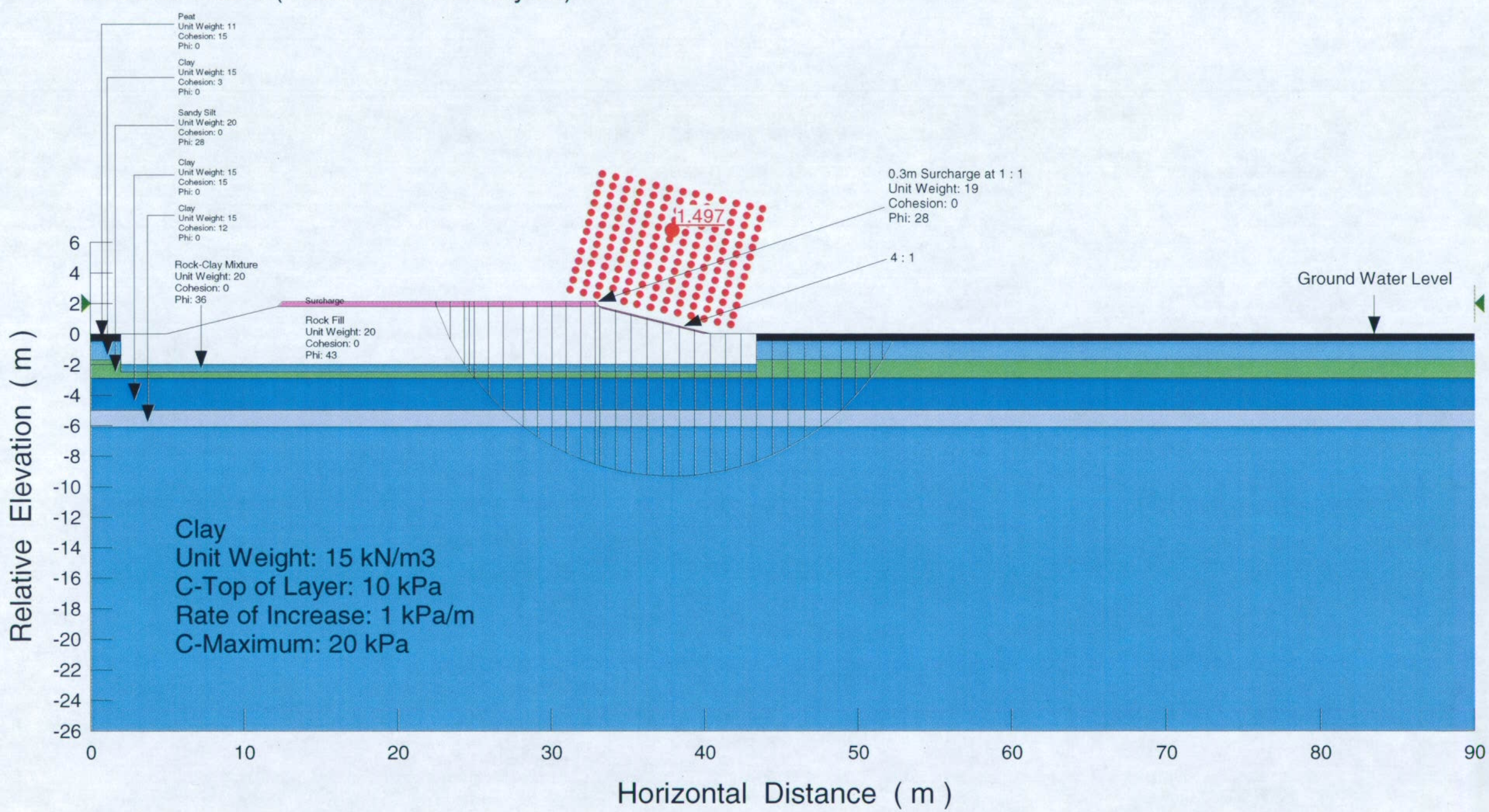
SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, Station 12+842
1.6m High, Rock Fill Embankment (Plus 0.5m Surcharge)
Undrained Case (Total Stress Analysis)

2.0 m Sub-Excavation
0.5 m Rock Penetration



SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, Station 12+855
1.7m High, Rock Fill Embankment (Plus 0.3m Surcharge)
Undrained Case (Total Stress Analysis)

2.0 m Sub-Excavation
0.5 m Rock Penetration



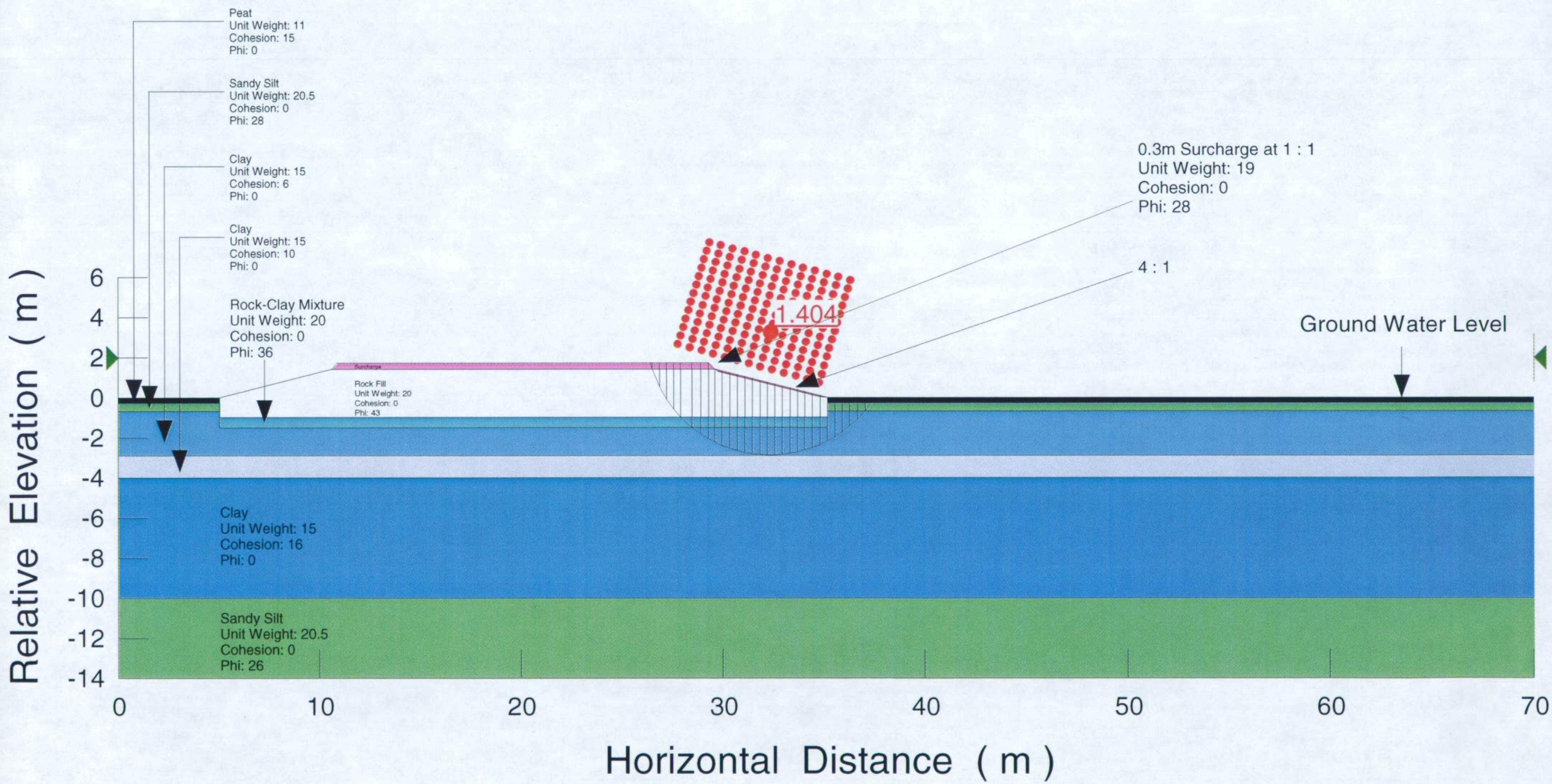
APPENDIX E5-4

Highway 638 Slope Stability Analysis Results (Based on Original Proposed Grade)

SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, PR6
1.4m High, Rock Fill Embankment (Plus 0.3m Surcharge)
Undrained Case (Total Stress Analysis)

Platform Width = 19m

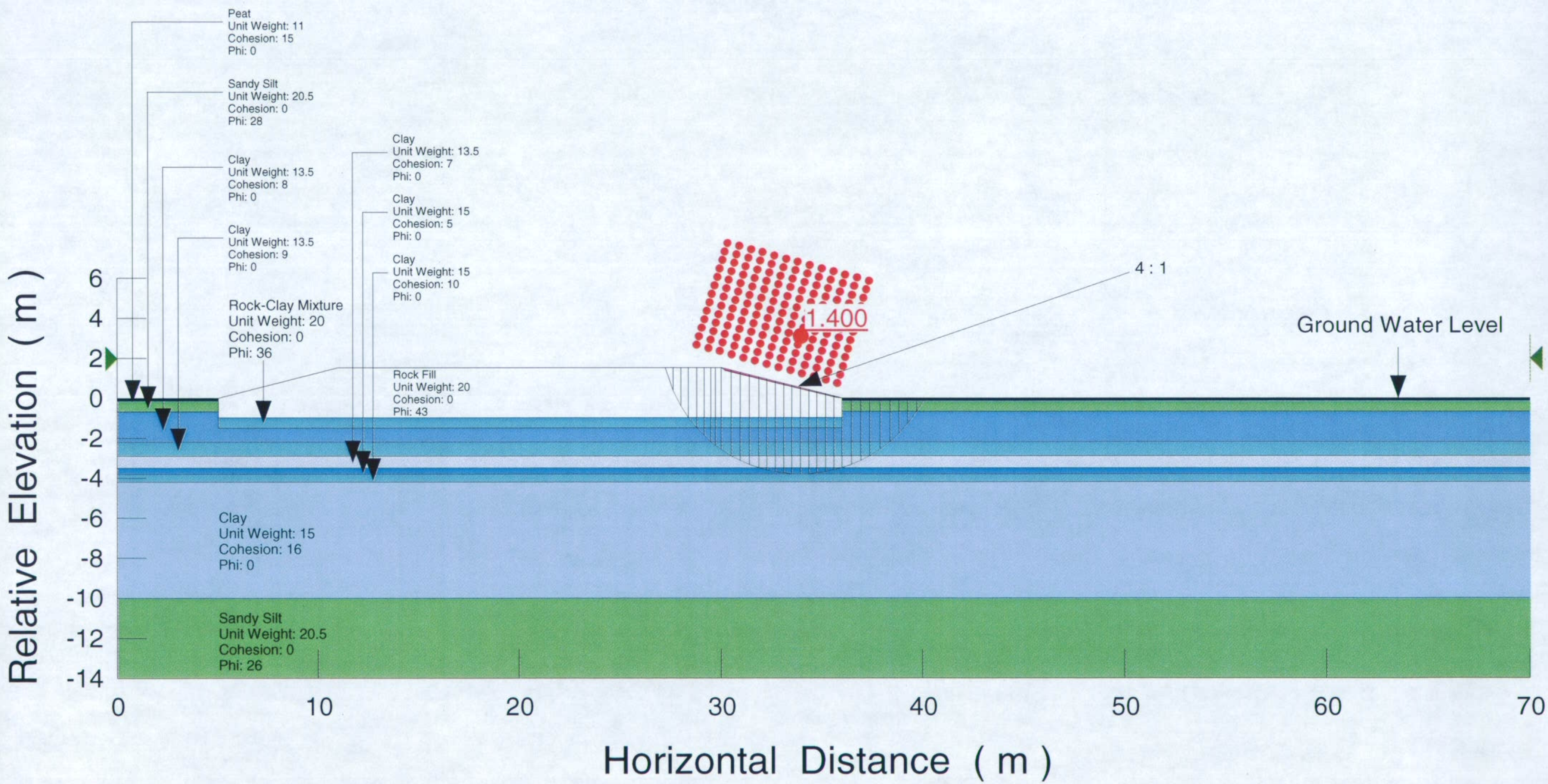
1.0 m Sub-Excavation
0.5 m Rock Penetration



SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, PR7
1.5m High, Rock Fill Embankment (NO Surcharge)
Undrained Case (Total Stress Analysis)

Platform Width = 19m

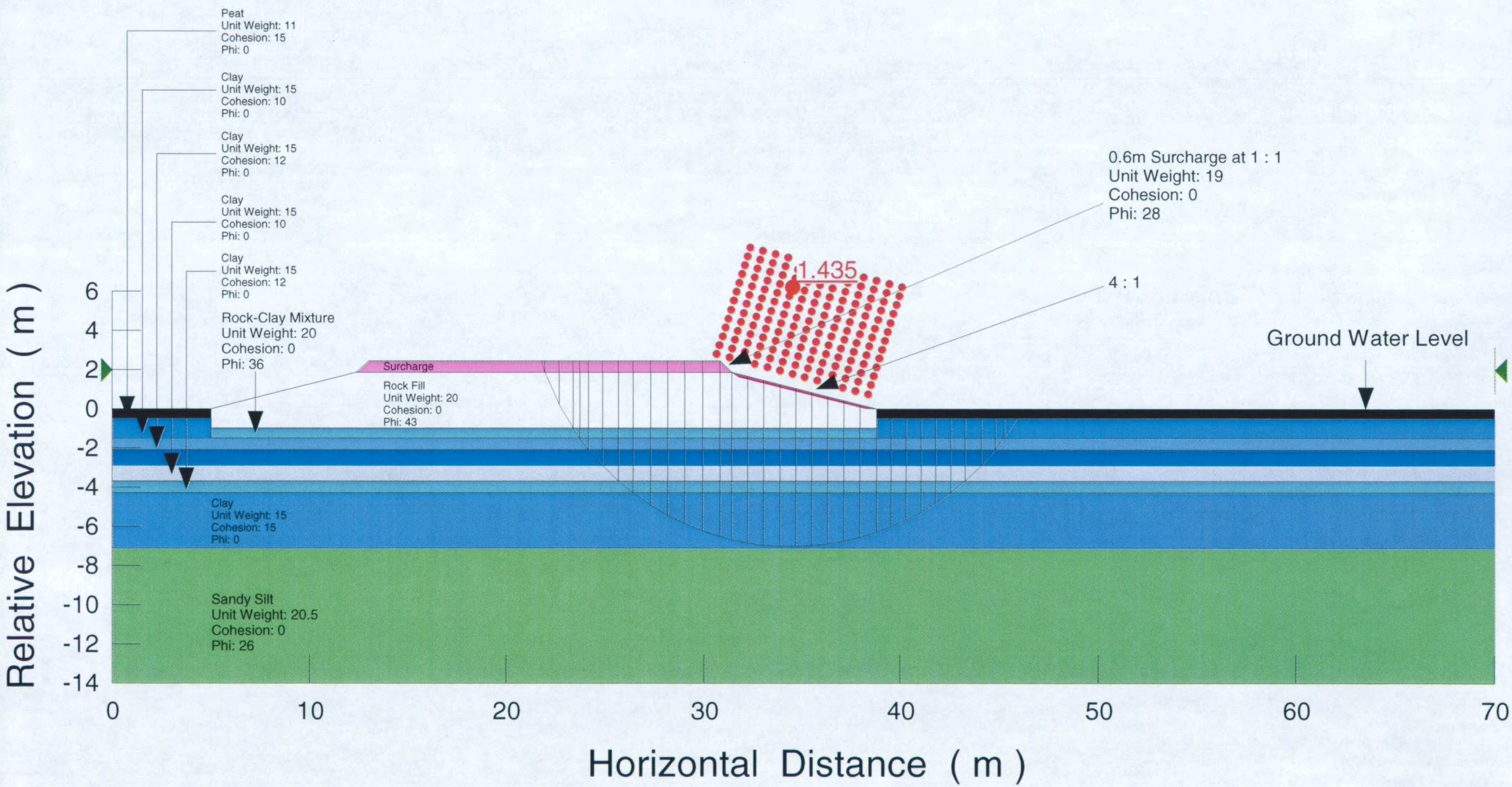
1.0 m Sub-Excavation
0.5 m Rock Penetration



SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, PR9
1.85m High, Rock Fill Embankment (Plus 0.6m Surcharge)
Undrained Case (Total Stress Analysis)

Platform Width = 19m

1.0 m Sub-Excavation
0.5 m Rock Penetration



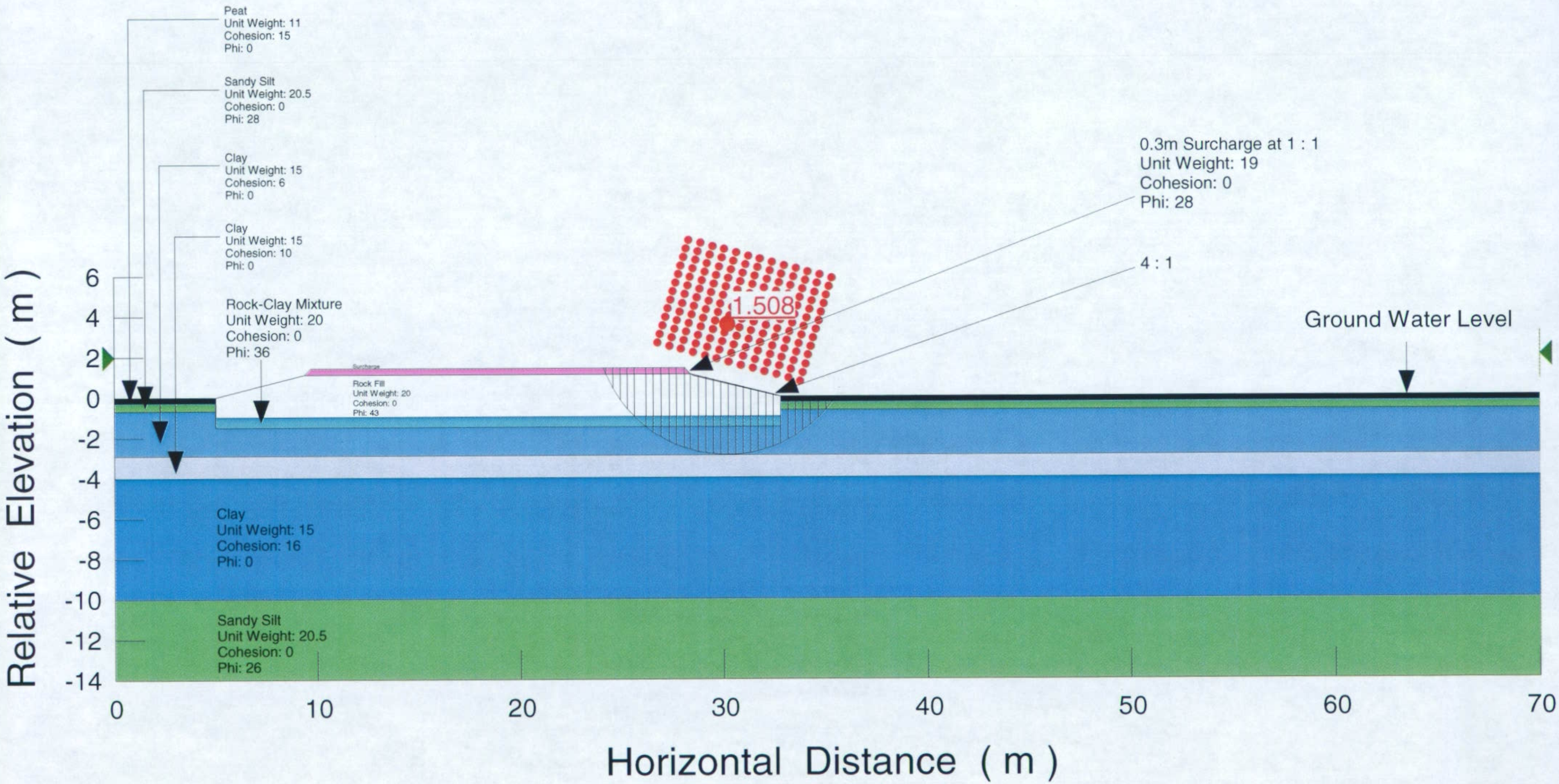
APPENDIX E5-5

Highway 638 Slope Stability Analysis Results (Based on Revised Proposed Grade)

SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, PR6
1.1m High, Rock Fill Embankment (Plus 0.3m Surcharge)
Undrained Case (Total Stress Analysis)

Platform Width = 19m

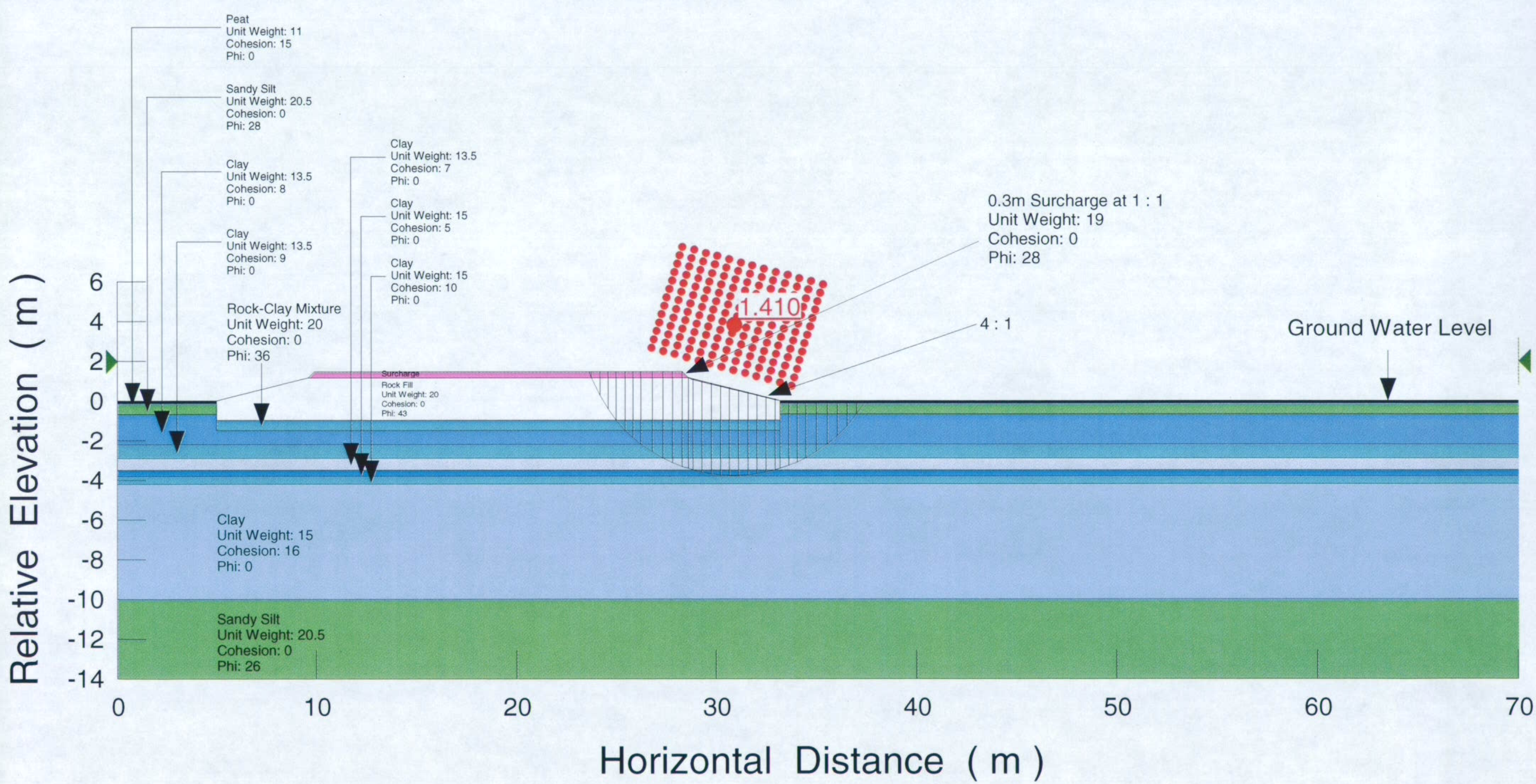
1.0 m Sub-Excavation
0.5 m Rock Penetration



SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, PR7
1.15m High, Rock Fill Embankment (Plus 0.3m Surcharge)
Undrained Case (Total Stress Analysis)

Platform Width = 19m

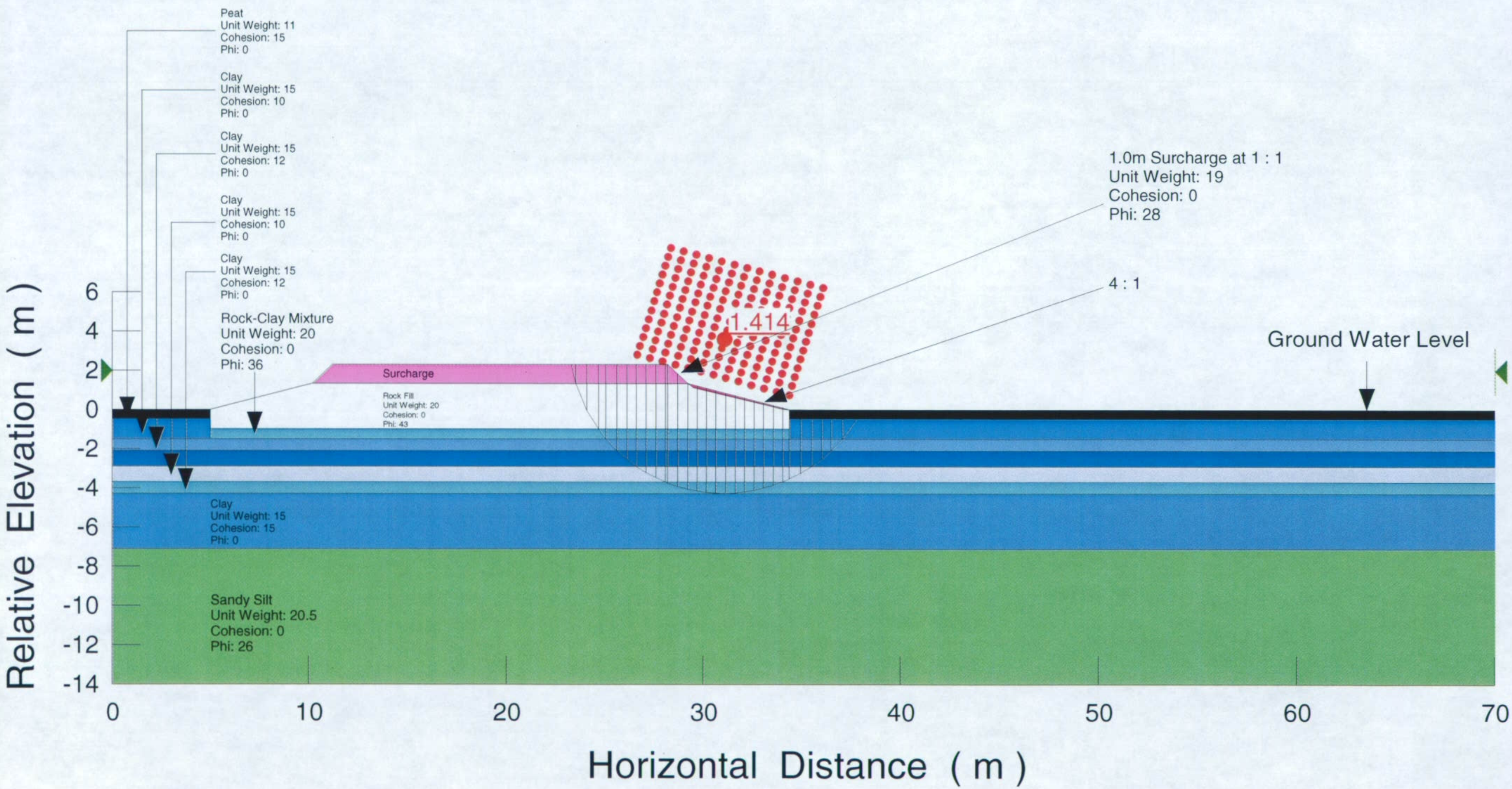
1.0 m Sub-Excavation
0.5 m Rock Penetration



SPT 1055, Highway 17 (New), Sault Ste.Marie
Swamp Crossing, PR9
1.3m High, Rock Fill Embankment (Plus 1.0m Surcharge)
Undrained Case (Total Stress Analysis)

Platform Width = 19m

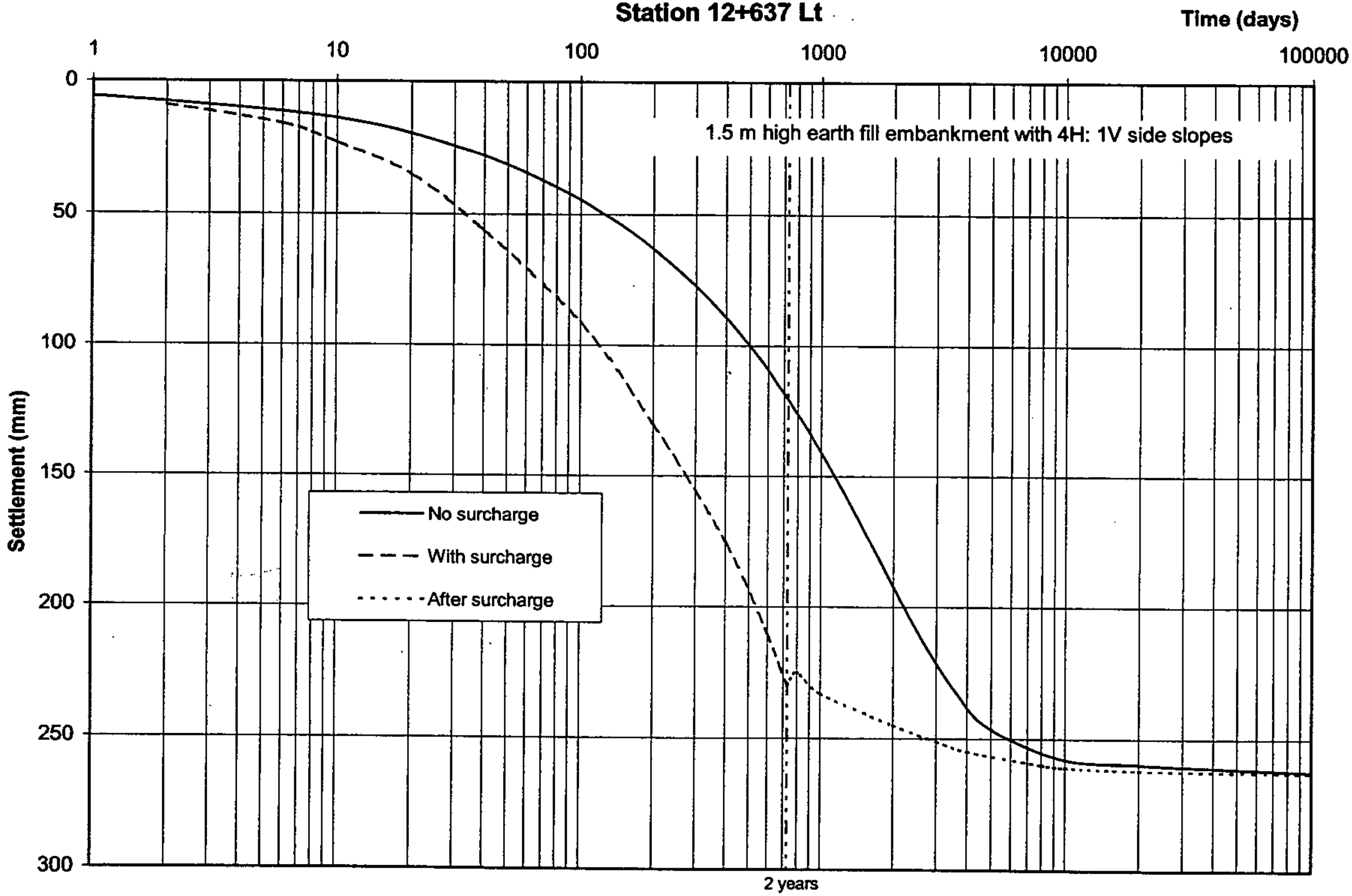
1.0 m Sub-Excavation
0.5 m Rock Penetration



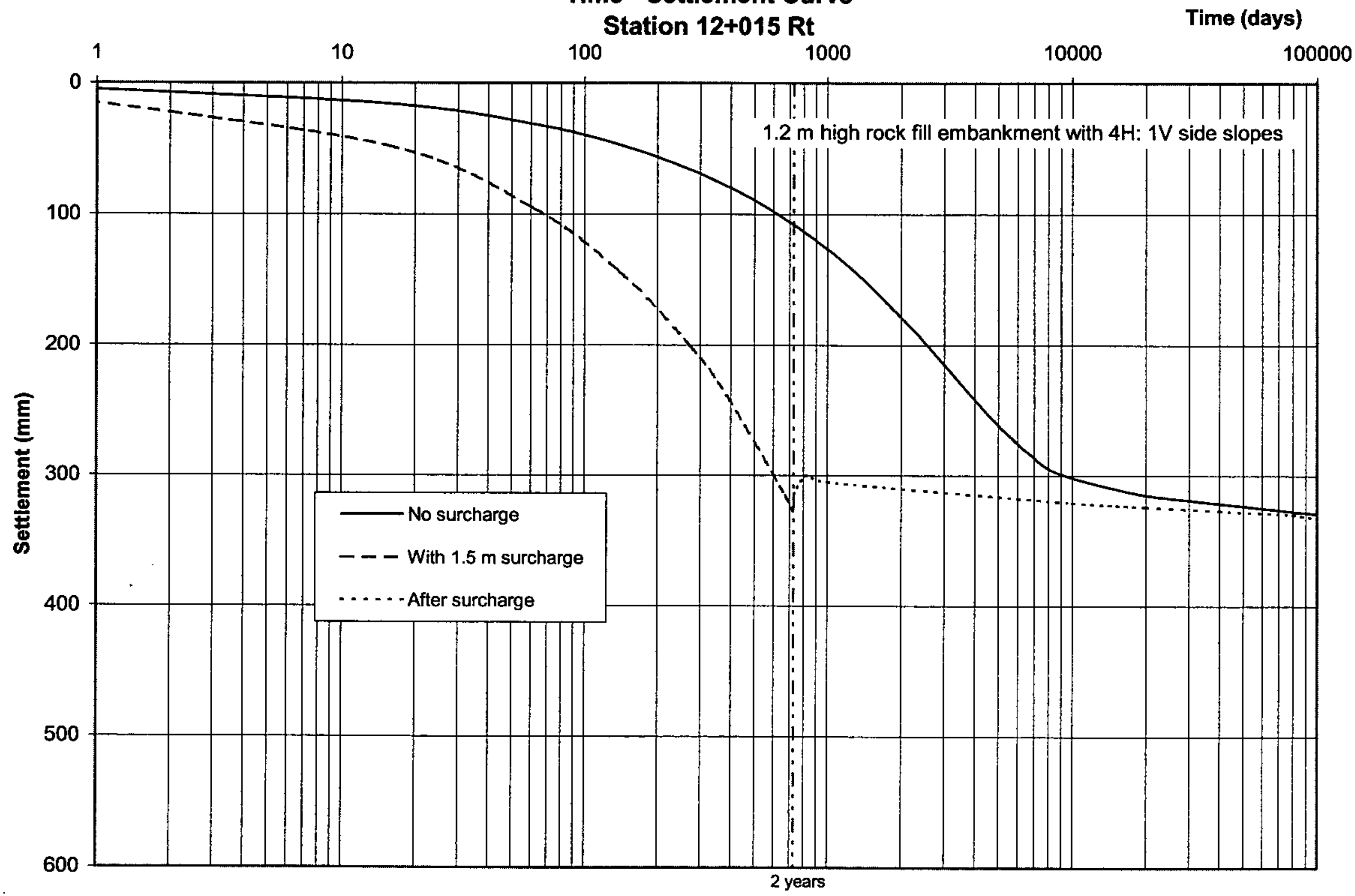
APPENDIX F5

Time-Settlement Curves

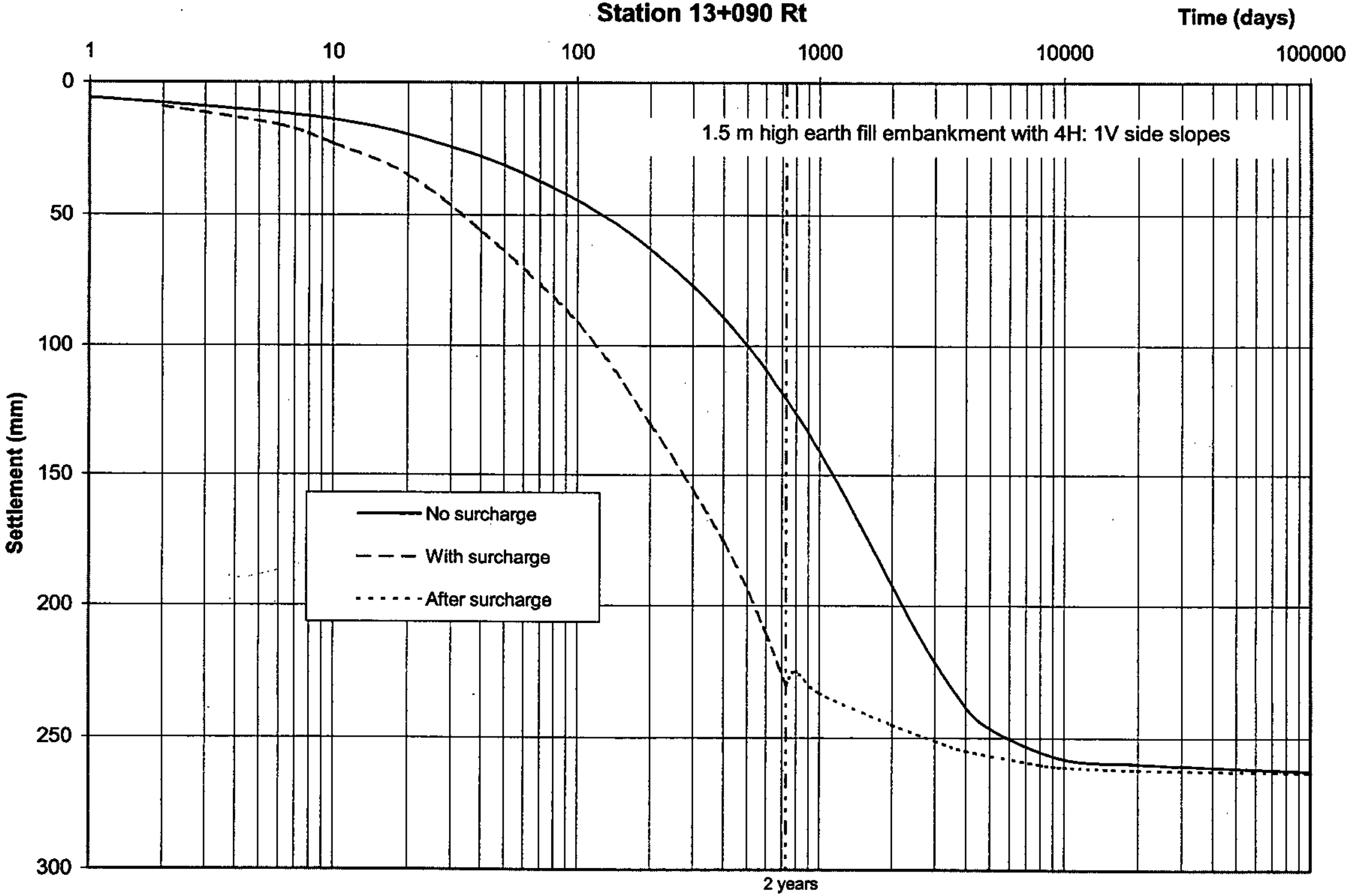
PROFILE 'B'
Time - Settlement Curve
Station 12+637 Lt



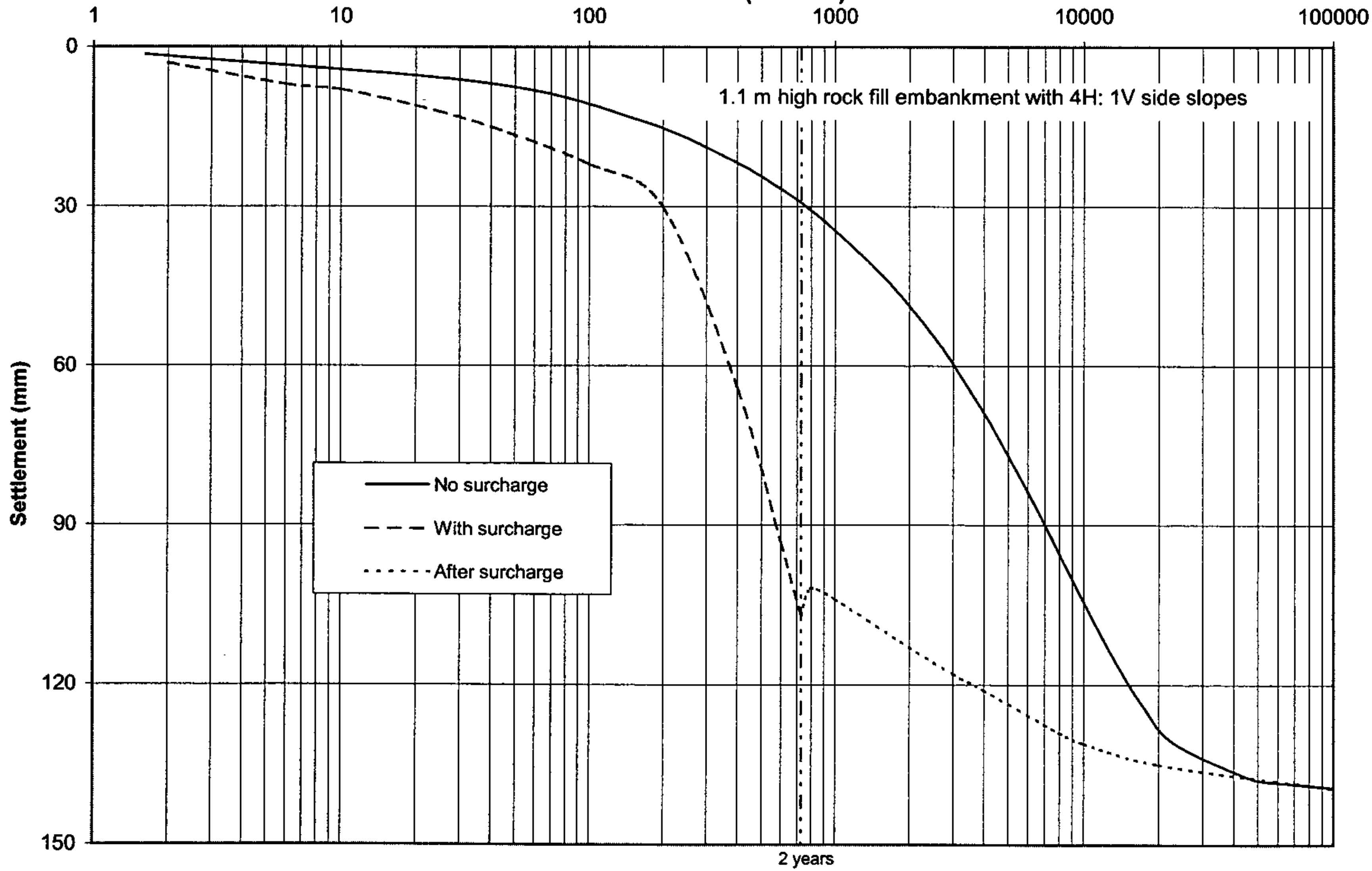
PROFILE 'B'
Time - Settlement Curve
Station 12+015 Rt



PROFILE 'B'
Time - Settlement Curve
Station 13+090 Rt



PROFILE 'D'
Time - Settlement Curve
Station PR 7 (11+645)



5.6 SITE NO. 6: HIGHWAY 17 (NEW) FILL SECTION BETWEEN STATIONS 15+470 AND 15+670 WESTBOUND LANES, AND BETWEEN STATIONS 15+470 AND 15+690 EASTBOUND LANES

A total of ten boreholes was put down for this 170 m stretch of alignment between Station 15+470 and 15+640. The boreholes showed the presence of 0.1 m to 0.25 m thick layer of topsoil or peat. The organic soils are underlain at most of the borehole locations by clay deposits at depths ranging between 0.1 and 2.0 m below the ground surface. The clay is irregularly layered with occasional thin silty clay to silt interlayers. Particle size distribution analyses show that the material has a high clay-size particle content and is of high to medium plasticity, with Liquid Limit values generally in excess of 50%. Frequently, the measured Liquidity Indices are greater than 1. The in-situ shear strengths, measured by means of Field Vane tests, range from generally 20 to 80 kPa, as shown in Figure C6-1, in Appendix C6. In some of the boreholes, the clay deposit is underlain by competent granular soils, at depths ranging from 2.3 to 7.8 m below the ground surface.

Upon their completion, water levels in the boreholes were measured at depths between 3.7 and 5.5 m below the ground surface. These water levels are believed to be unstabilized due to the practically impervious nature of the clay deposit. From the colour of the soil, site observations and the moisture contents of the soil samples, the groundwater table is believed to be at or very near the ground surface level.

5.6.1 EMBANKMENT STABILITY

In this section, the height of the proposed embankments ranges from about 3.1 to 6.3 m for the east bound lanes and 2.4 to 4.6 m for the west bound lanes. The foundation stability of the embankments was analyzed by the limit equilibrium method, utilizing Bishop's Simplified method of analysis. For this purpose, the computer program Slope/W was utilized. Bishop's Simplified Method is known to be slightly on the conservative side because of the fact that the side forces on the slices are ignored, as opposed to more rigorous methods.

For the undrained (short-term) stability analyses, undrained shear strengths (c-values) of the clay were based on the Field Vane test results at each borehole location and assuming a ϕ-value of zero. The c-values used in our analyses ranged from 20 to 80 kPa. No correction factor to the Field Vane tests was applied (e.g. Bjerrum, Aas, etc. correction). Because of this and due to irregularly layered, fat (highly plastic) nature of the clay deposit, a minimum safety factor of 1.40 was deemed necessary. In addition to this, the conditions of the site are variable in relation to the thickness of the clay deposit and to a certain extent its shear strength. The latter can partially be attributed to the layered nature of the clay.

Long-term (drained) analyses were also performed at some selected locations and, as was expected, these were not found to be critical.

In our analyses, a maximum surcharge of 1.5 m was added to the height of the embankments, in order to deal with the settlement aspects, as will be discussed.

The soil parameters used in the slope stability analyses are presented in Table 5.6.1.1. Typical embankment slope stability sections are presented in Appendix E6.

Table 5.6.1.1 Soil Parameters Used in Slope Stability Analyses

Material Type	Short-Term Analysis			Long-Term Analysis		
	ϕ (degrees)	c (kPa)	γ (kN/m^3)	ϕ' (degrees)	c' (kPa)	γ (kN/m^3)
Embankment Fill (select subgrade material)	30	0	21.5	30	0	21.5
Sand Backfill (used to replace existing peat and other surficial unsuitable soils)	30	0	20.0	30	0	20.0
Surficial Granular Soils	30	0	20	30	0	20
Clay	0	20-80	16.0-18.3	24	2	16.0-18.3
Lower Glacial Till	33	0	21.5	33	0	21.5

The analyses results show that earth fills with 2H:1V side slopes provide a minimum safety factor of 1.40 for the proposed height of embankments, except at about Station 15+630 (EBL). Earth fills can be built to 2H:1V slopes from Stations 15+470 to 15+670 along the WBL and from Stations 15+470 to 15+580 along the EBL. At about Station 15+580 along the EBL, the side slopes should be gradually flattened from 2H:1V to reach 4H:1V side slopes at Station 15+610. This 4H:1V side slope should be maintained between Stations 15+610 and 15+650. The remaining side slopes along the EBL can be gradually returned to the normal 2H:1V side slopes from Station 15+650 to the southern end of this stretch.

It should be pointed out that in our analyses, we have assumed that all the existing peat, topsoil and other unsuitable soils will be removed and replaced with suitable granular soils, such as Granular 'B' materials.

5.6.2 SETTLEMENT OF EMBANKMENTS

From the profile drawings, the maximum height of fill along the EBL is about 3.1 to 6.3 m, while along WBL, the maximum height is about 2.4 to 4.6 m. Our analyses show that under the proposed height of the embankments (as detailed in Table 5.6.2.1, presented below) the anticipated settlements range from about 100 mm to 250 mm.

Table 5.6.2.1 Fill Heights at Station Centreline

Station Number	Proposed Embankment Height (m) **WBL**	Proposed Embankment Height (m) **EBL**
15+469	2.36	2.44
15+499	3.00	3.14
15+501	3.05	3.08
15+531	3.65	3.70
15+556	4.18	4.28
15+559	4.25	4.28
15+561	4.29	4.41
15+593	4.55	5.16
15+621	3.13	5.86
15+630	2.68	6.29
15+676	0.52	3.31
15+680	0.11 (cut)	2.94

The highest settlement estimate values were obtained along the EBL Stations 15+630 where the height of the proposed embankment is 6.29 m and the thickness of the clay deposit is about 5.3 m and also at Station 15+561 where the thickness of the clay is about 6.5 m and the embankment height is 4.41 m. At Station 15+469 where the embankment height is only about 2.4 m, the estimated settlement is about 220 mm and can be expected to take place over a period of about two years.

In order to alleviate the effects of these settlements, we recommend surcharging. Our stability analyses show that the maximum height of surcharge that can be applied at most critical locations is 1.5 m.

To minimize the required surcharge materials, a 1.2 m high surcharge can be employed in this site. The height of surcharge can gradually decrease towards the southern limit of this stretch (i.e. from Station 15+590 to Station 15+660 along the WBL, and from Station 15+650 to Station 15+690 along the EBL).

With this approach, the anticipated settlements of the subject site after one year of surcharging are reduced to about 80%. The residual settlements are expected to take place gradually within one year and should therefore not cause major concern.

Recommendations for further to the north of Station 15+470 are highly dependent to the findings of the foundation investigation in those areas and therefore, will not be included in the present discussions.

In any event, it is recommended that surcharging be carried out with proper instrumentation for field monitoring. It is furthermore recommended that the surcharge be placed gradually (i.e., preferably at least 3 layers, starting from one end of the site and proceeding to the other end), to allow excess pore pressures to dissipate.

Embankments should be provided with a widened cross-section to allow for settlements of the underlying soils and any possible future minor grade raises. In this case, the road platform should be widened by at least 1 m on each side of the centerline. However, in accordance with the Northern Region Engineering Directive NRE-98-200, the road platform could be widened by 2 m on each side (total 4 m). In addition, certain amount of regrading may be necessary due to settlements, including the side slopes after the surcharging period.

5.6.3 CONSTRUCTION

For embankment construction, all organic or otherwise unsuitable materials should be removed from the toe of the proposed embankment. Based on the available borehole data, the average values of the unsuitable soils to be stripped can be expected to be variable but for preliminary estimating purposes, it can be assumed to be about 0.2 m on the average.

After stripping, the exposed subgrade should be inspected and approved by the Quality Verification Engineer appointed by the Contract Administrator. The subgrade may need to be compacted from the surface using a suitable compactor. This should, however, only be done at the discretion of the Quality Verification Engineer, as it may not be feasible to effect surface compaction due to site conditions (i.e. high water table), weak nature of the clay, etc. It is believed that because of the presence of high water table at the site dewatering will be required to facilitate the stripping and backfilling operations. This is normally achieved by gravity drainage and pumping from open sumps. Construction procedures will have to be cognizant of the soft nature of the clay at most borehole locations.

The type of material to be used for soil replacement (i.e. to replace the stripped soils to the existing original ground surface) should be compatible with the proposed embankment fills.

For example, where the embankment is to consist of earth fill, granular fill, which is compactable, should be used (e.g. Granular 'B' material). In this instance, the first one to two lifts of the fill may need to be thicker than normal (i.e. thicker than 300 mm lifts).

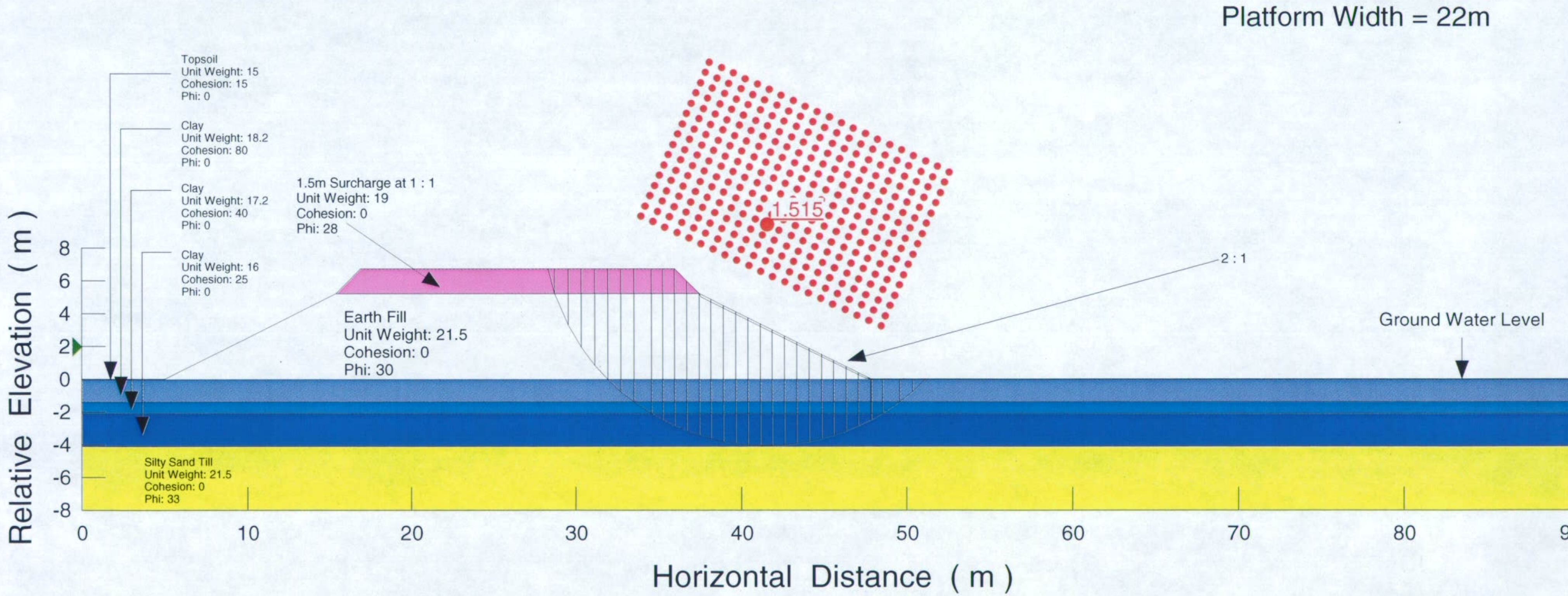
The materials used for the construction of the embankment should consist of approved acceptable earth fill. The earth fills should be placed in lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density (SPMDD) (OPSS 206 and OPSS 501).

Proper erosion control measures should be implemented both during construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

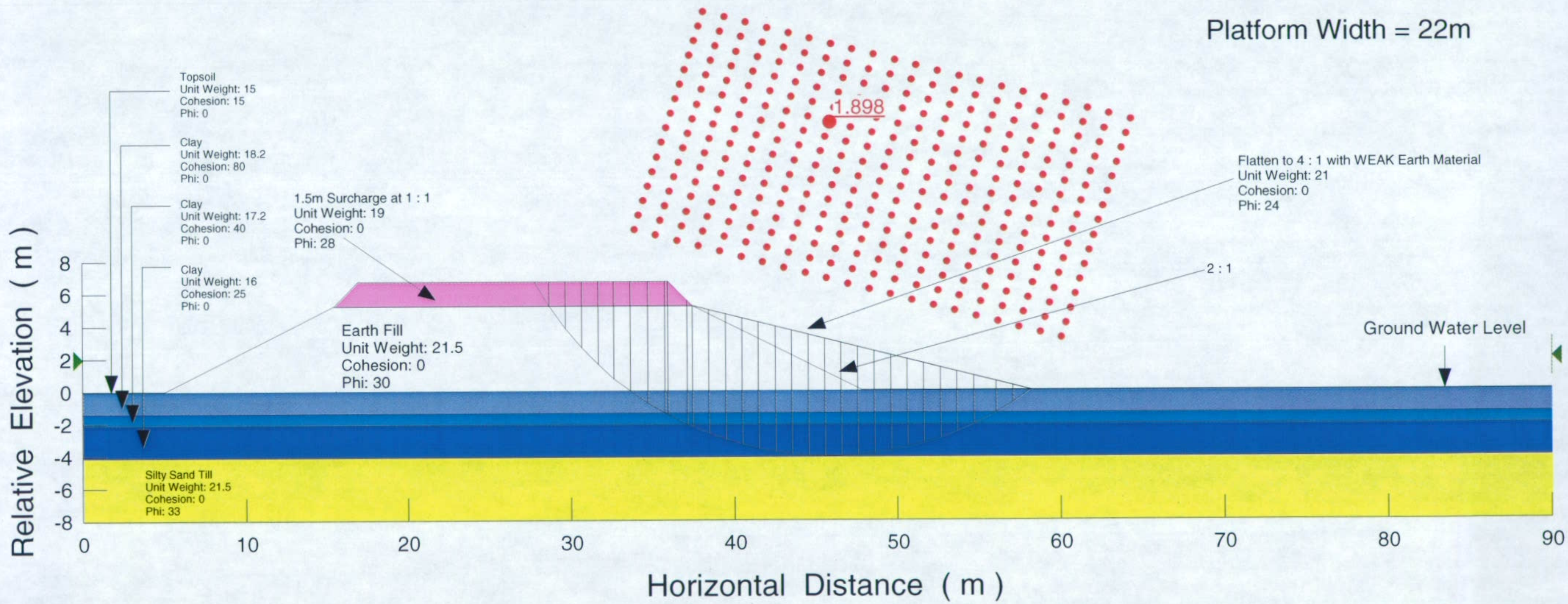
APPENDIX E6

Slope Stability Analysis Results

SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 15+593, 5.2m High, Earth Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)

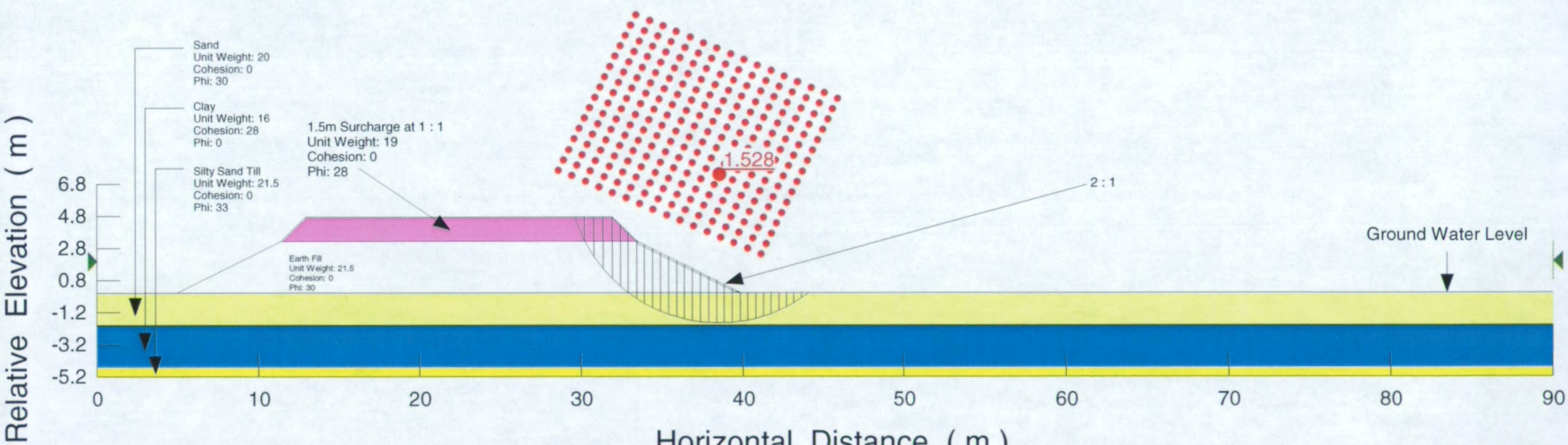


SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 15+593, 5.2m High, Earth Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)



SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 15+621, 3.2m High, Earth Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)

Platform Width = 22m



Ministry of Transportation
Ontario

Foundation Design

SPT 1055

RECORD OF BOREHOLE No 10+923; 19 m Rt

METRIC

GWP 354-94-00
DIST 62 HWY 17 (New)
DATUM Geodetic

LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 151 307.6; E 301 623.5
BOREHOLE TYPE Hollow Stem Augers
DATE 4/3/2002

ORIGINATED BY G.I.
COMPILED BY M.L.
CHECKED BY R.A.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
189.6	Ground Surface									
0.0	0.2 m Topsoil	1	SS	4						
189.2 / 0.4	SILTY SAND occasional topsoil inclusions grey, wet, very loose									
	very stiff	2	SS	4						
	firm	3	SS	2						
		4	SS	2						
		5	SS	2						
		6	SS	2						
	soft to firm	7	TW	PH		14.8	0	0	15	85
		8	SS	2	▼ *					
		9	SS	2						
		10	SS	2						
179.4 / 10.2	End of borehole									

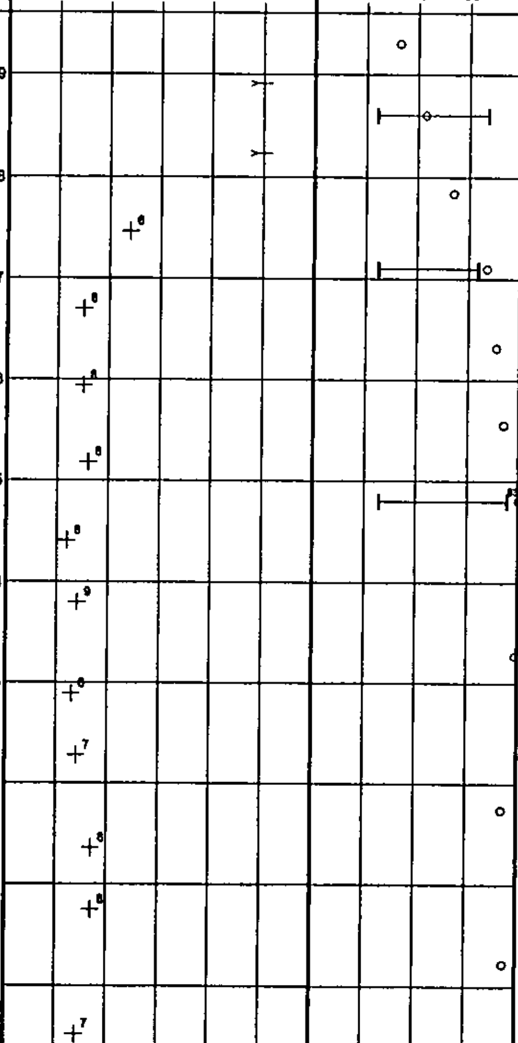
* Water level at 6.7 m (not stabilized) and hole open to 7.6 m on completion.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

ELEVATION SCALE: 189, 188, 187, 186, 185, 184, 183, 182, 181, 180



+³, ×³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

SPT 1055

RECORD OF BOREHOLE No 10+923; 19 m Rt

METRIC

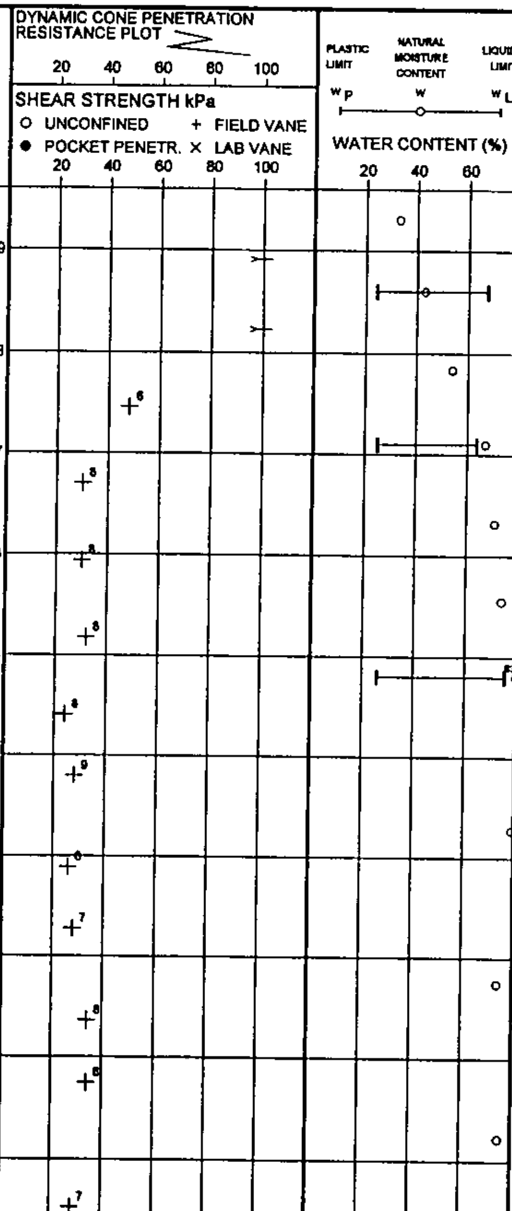
GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 151 307.6; E 301 623.5 ORIGINATED BY G.I.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers COMPILED BY M.L.

DATUM Geodetic DATE 4/3/2002 CHECKED BY R.A.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
189.6	Ground Surface							
0.0	0.2 m Topsoil	1	SS	4				
189.2 0.4	**SILTY SAND** occasional topsoil inclusions grey, wet, very loose					189		
	very stiff	2	SS	4				
	firm	3	SS	2		188		
		4	SS	2		187		
		5	SS	2		186		
		6	SS	2		185		
	soft to firm	7	TW	PH			14.8	0 0 15 85
						184		
		8	SS	2	▼*	183		
						182		
		9	SS	2		181		
		10	SS	2		180		
179.4 10.2	End of borehole							

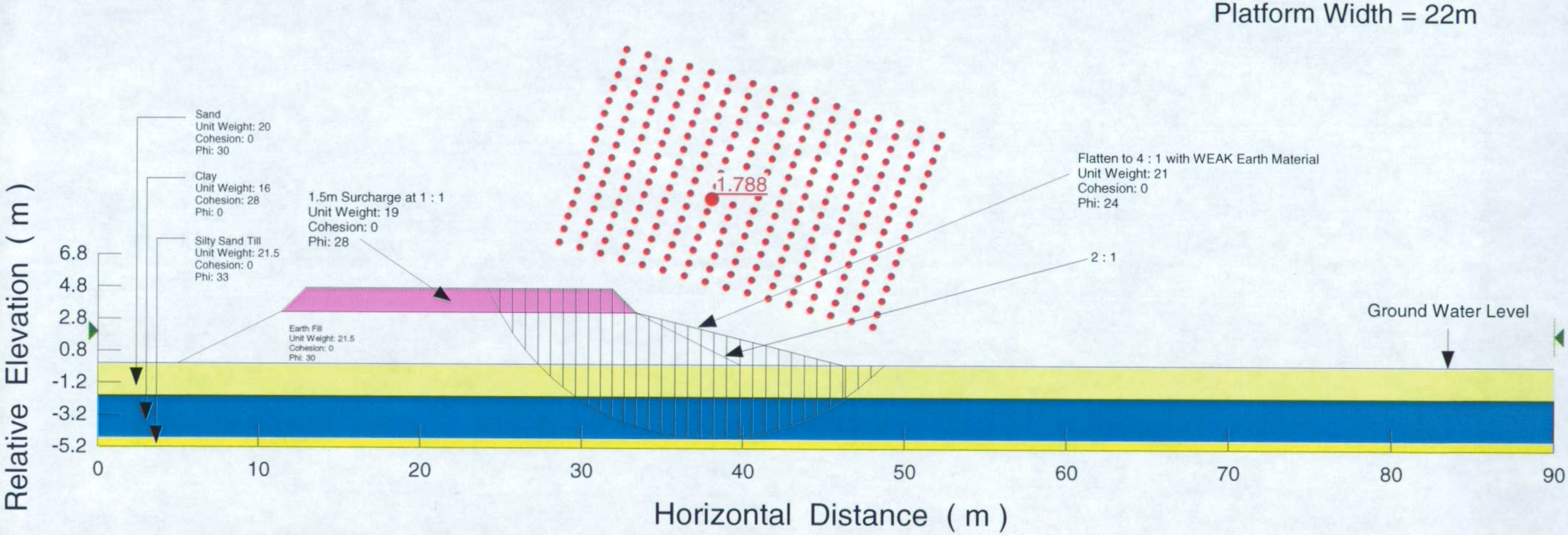
* Water level at 6.7 m (not stabilized) and hole open to 7.6 m on completion.



+³, ×³: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

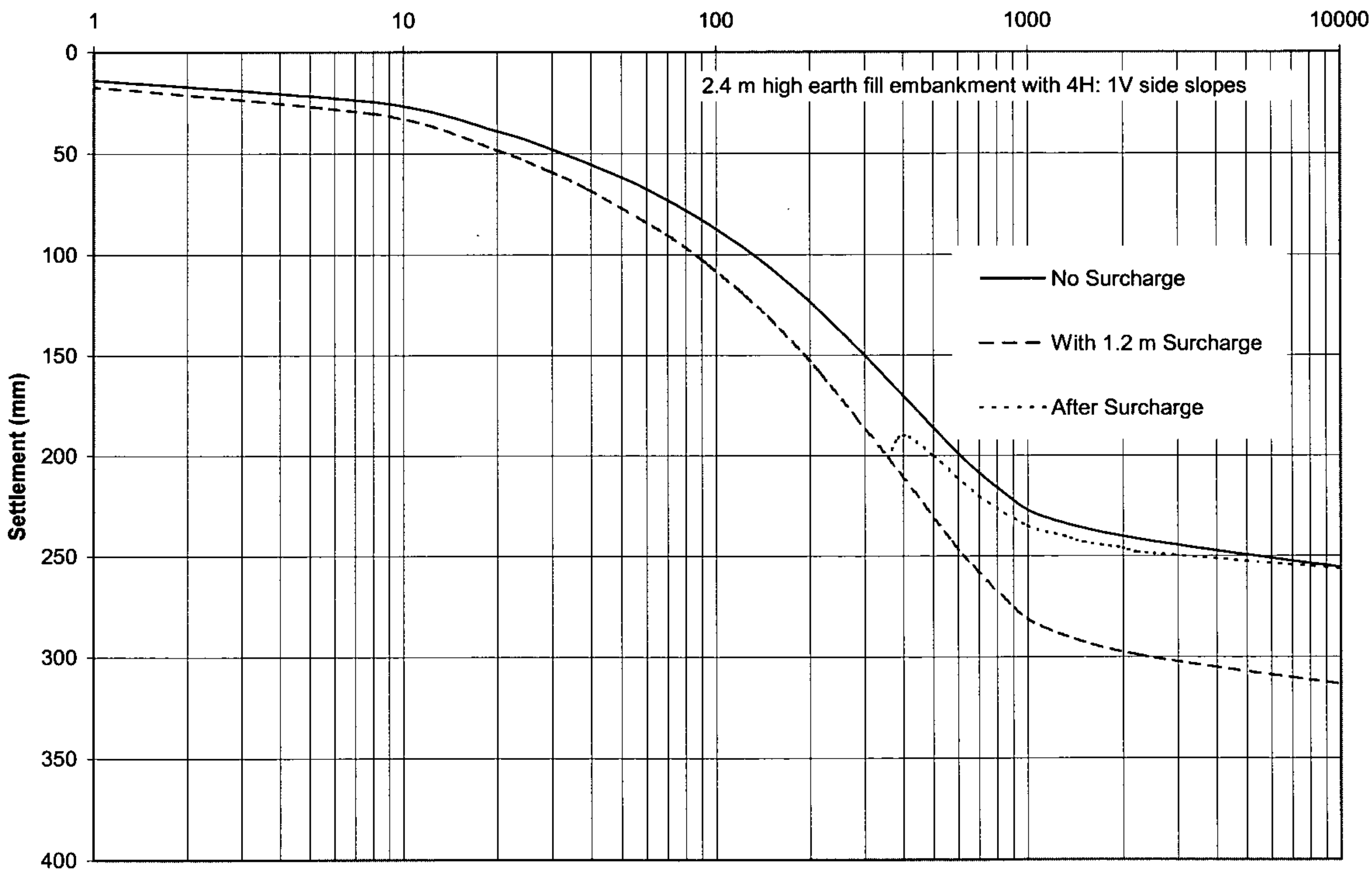
SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 15+621, 3.2m High, Earth Fill Embankment (Plus 1.5m Surcharge)
Undrained Case (Total Stress Analysis)



APPENDIX F6

Time-Settlement Curve

Time - Settlement Curve
Station 15+469 CL



5.7 **SITE NO. 7 :** HIGHWAY 17 (NEW) CUT SECTION BETWEEN STATIONS 15+670 AND 15+850 WESTBOUND LANES, AND FILL AND CUT SECTION BETWEEN STATIONS 15+690 AND 15+850 EASTBOUND LANES

5.7.1 CUT SECTION BETWEEN STATIONS 15+670 AND 15+850 WBL

The profile drawing indicates that up to about 16 m of rock cut is anticipated along the centerline of the WBL road. We understand that the rock between Highway 17 (New) and Government Road alignments will be essentially removed and up to about 3 m of cut could be expected to the left of Highway 17 (New). The cut material consists of surficial topsoil, sand and gravel with occasional cobbles and boulders, and bedrock.

5.7.1.1 STABILITY OF CUT

From the cross-section in Drawings 7C and 7D, the cut slope to the left of the road could consist mainly of bedrock (as shown for Sections 15+730 and 15+760). The bedrock at this site consists of both the quartzite and the sandstone.

The quartzite bedrock in this section is considered stable at steep cut slope of 0.25H:1V, provided that the rock within the cut zone is not steeply jointed; or if steeply jointed, the joints are not dipping towards the highway. The sandstone bedrock is relatively weaker than the quartzite bedrock but it is still considered 'strong' and therefore, a 0.25H:1V rock cut slope could also be utilized, provided that a wider clear zone or larger catchment area is provided for cuts higher than about 3 m. It is recommended that the exposed rock should be inspected by a Rock Engineer during construction to assess its stability and provide any corrective measures, if required. It should be ensured that no rocks are overhanging along the steep slope. If this rock is found to be fractured and/or unfavourably jointed or unstable, corrective measures such as grouted dowels and/or rock anchors should be implemented.

If a Rock Engineer or Geologist will not be utilized to inspect the exposed rock, it is recommended that reinforcing of joints with dowels or rock bolts should be provided in the Contract Documents through an NSSP. Any rock bolt design and/or grouting should be illustrated on the contract drawings in addition to including specifications in the contract documents. Alternatively, the rock could be cut at a flatter slope such as 0.5H:1V even for the quartzite rock and/or clear zone could be increased.

5.7.1.2 CONSTRUCTION

Considering that the groundwater table was generally not encountered at this site, no major problem with groundwater seepage is anticipated. Surface water run-off should however be controlled by interceptor ditch along the top of the slope.

Based on the measured moisture contents, the cut materials are expected to be generally damp and are therefore suitable for re-use for fills.

Blasting of the rock should be carefully controlled so as not to produce an over-break which could cause instability of the rock face. As mentioned before, the stability of the rock should be assessed by an experienced Rock Engineer during construction.

5.7.2 FILL AND CUT SECTION BETWEEN STATIONS 15+690 AND 15+850 EBL

The boreholes and test pits which explored this fill area show the presence of probably dense to very dense sand and gravel with cobbles and boulders, over bedrock.

5.7.2.1 EMBANKMENT STABILITY

Based on the profile drawings, the grades along this section are expected to be raised to a maximum height of about 5.7 m at Station 15+850 EBL.

Embankment stability analyses were performed using limit state equilibrium (Bishop's Simplified Method by the computer program Slope/W). A minimum factor of safety of 1.40 is considered sufficient.

The following soil parameters were used for the stability analyses:

Table 5.7.2.1 Soil Parameters Used in Slope Stability Analyses

Material Type	Short-Term Analysis			Long-Term Analysis		
	ϕ (degrees)	c (kPa)	γ (kN/m^3)	ϕ' (degrees)	c' (kPa)	γ (kN/m^3)
Embankment Fill (Select Subgrade Material)	30	0	21.5	30	0	21.5
Sand and Gravel or silty sand till	35	0	21.5	35	0	21.5

Typical embankment slope stability sections are presented in Appendix E7.

Based on the above and our analyses, no foundation failures are anticipated for earthfill embankments up to 6 m high and with normal 2H:1V side slopes, provided that all organic, weak or otherwise unsuitable materials are removed as per MTO standards before placing the fill, and that the fill consists of properly compacted, acceptable inorganic material (e.g., non frost susceptible soils or SSM). If necessary and to keep the embankment slopes uniform across this area, the slopes could be flattened to 4H:1V using inorganic unsuitable soils (e.g., wet and weak soils).

5.7.2.2 SETTLEMENT OF EMBANKMENTS

From the profile drawings, the maximum height of fill is about 5.7 m.

Our analyses show that the expected settlements of the founding soils under the weight of the proposed embankment are in the range of 50 to 150 mm. It is anticipated that these settlements will occur rapidly and will probably be complete a few weeks after the full height of the embankment is reached.

No surcharging is required within this section.

Fills at the above section should be provided with a widened cross-section to allow for settlements of the underlying soils and a future grade raise. For Highway 17 (New), we recommend that the road platform should be widened by at least 2.0 m on each side of the centreline (total of 4 m), to conform with the Northern Region Directive.

5.7.2.3 CONSTRUCTION

For embankment construction, all organic and other unsuitable soils should be removed within an envelope given by an imaginary slope no steeper than 1:1 from the toe of the proposed embankment. Based on the available borehole data, the average values of the unsuitable soils to be stripped can be expected to be variable but for preliminary estimating purposes, it can be assumed to be about 0.3 m on the average.

After stripping, the exposed subgrade should be inspected, approved and properly compacted from the surface, using a suitable compactor. This should, however be done at the discretion of the Quality Verification Engineer.

The materials used for the construction of the embankment should consist of approved, acceptable earth or rock fill. The earth fills should be placed in lifts not exceeding 300 mm before compaction and each lift of earth fill should be uniformly compacted to at least 95%

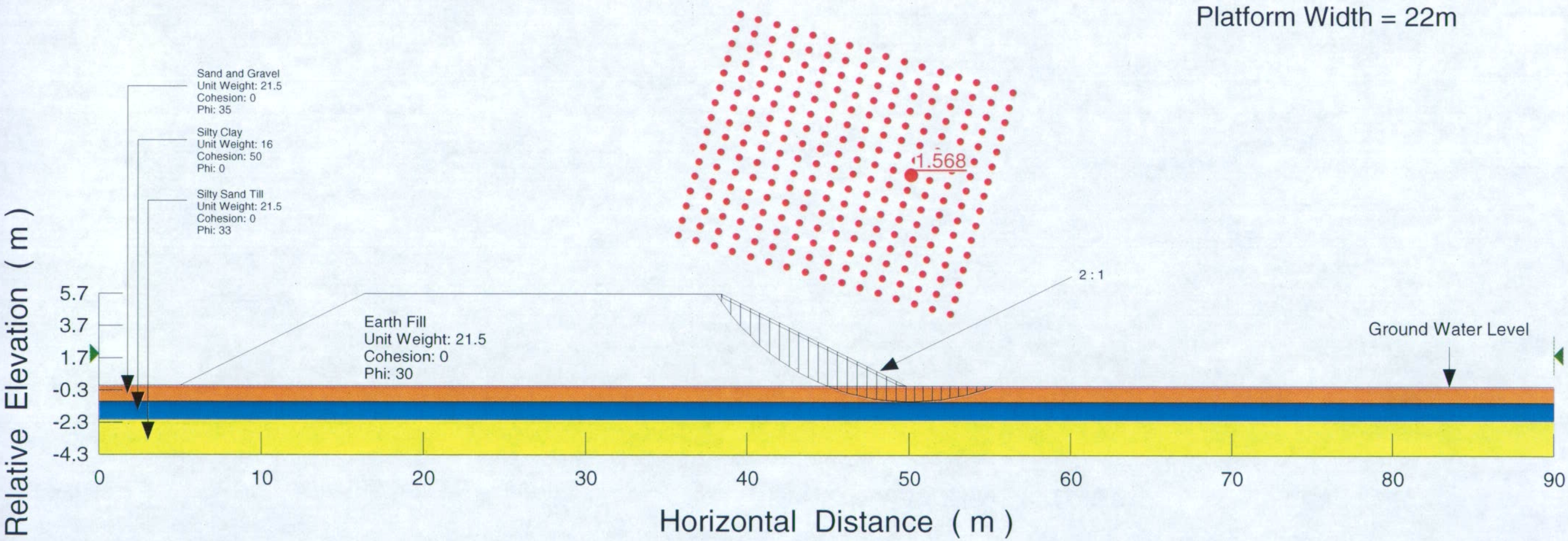
of the material's Standard Proctor Maximum Dry Density (SPMDD) (OPSS 206 and OPSS 501).

Proper erosion control measures should be implemented both during construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

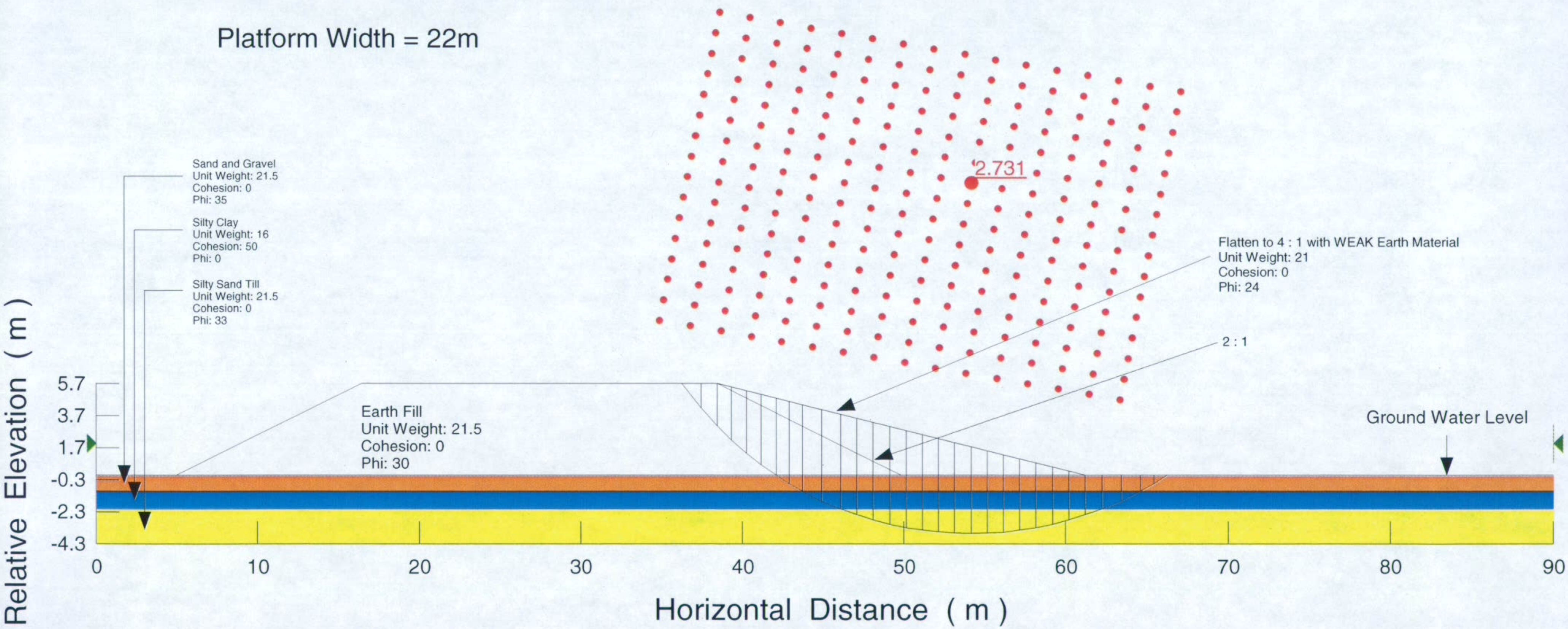
APPENDIX E7

Slope Stability Analysis Results

SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 15+850, 5.7m High, Earth Fill Embankment
Undrained Case (Total Stress Analysis)



SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 15+850, 5.7m High, Earth Fill Embankment
Undrained Case (Total Stress Analysis)



5.8 SITE NO. 8 : HIGHWAY 17 (NEW) FILL SECTION BETWEEN STATIONS 15+850 AND 16+600

A total of 41 boreholes was put down for the approximately 0.7 km of alignment between Stations 15 + 850 and 16 + 560. The surface of the bedrock beyond the northern limit of the site dips from a massive bedrock outcrop in the south, south-west direction. The thickness of the overburden increases from about 1 to 5 m at Station 15 + 910 to about 10 to 12 m near the CPR crossing at about Station 16 + 100. The depth to the surface of the bedrock (i.e. overburden thickness) increases southerly beyond this station. In the northerly section of the site, to about Maple Leaf Road crossing, the boreholes revealed the presence of a 0.1 to 0.3m thick topsoil layer underlain by a sand deposit which ranges in composition from fine sand with a variable silt content to well-graded sand and coarser gravelly sand. The relative density of the sand ranges from very loose to very dense but is generally in the loose to loose-compact range. The sand is underlain by silty sand till and/or a zone of very coarse overburden, which consists of cobbles and boulders within a sand and gravel matrix, generally overlying bedrock.

A weak and compressible clay deposit underlain by fine sand was encountered near the Maple Leaf Road crossing. The thickness of the clay increases from less than 1 m, southerly to in excess of 10 m near the south limits of the site. Also towards the south end of the project, where the ground is relatively depressed and water-logged, a 0.4 to 0.6 m thick layer of surficial peat was contacted overlying the massive clay deposit.

Groundwater is generally about 1 m below the ground surface near the northern part and within 0.5 m to the south of Maple Leaf Road.

5.8.1 EMBANKMENT STABILITY

As shown on the profiles presented, the height of the embankment at the site reaches a high point of about 10 to 11 m at the CPR crossing, then gradually decreases to about 7.5 m at the Maple Leaf Road crossing. Beyond this point, the height of the proposed embankments gradually decreases to about 3 m at the south limit of the project at Station 16 + 560.

The foundation stability of the proposed embankment was analyzed by the limit equilibrium method, utilizing Bishop's Simplified method of analysis and Slope/W computer software.

Where the overburden consists of entirely granular deposits, long and short-term analyses are essentially the same. Where cohesive deposits were also encountered (i.e. generally near and south of Maple Leaf Road), for the undrained (short-term) stability analyses, undrained shear strengths (c-values) of the clay were utilized, based on Field Vane Test results. The c-values used in our analyses ranged from 10 to 50 kPa. As no correction factor (e.g. Bjerrum, Aas, etc.) was applied to the Field Vane Test results and considering the layered and sensitive nature of the clay, a minimum factor of safety of 1.40 was sought, for the undrained analyses involving these clays.

In our analyses, surcharge loads of between 0.6 and 1.2 m were added (depending on the location), in order to deal with settlement issues, as will be discussed in Section 5.8.2 of this report.

The soil parameters used in the slope stability analyses are given in Table 5.8.1.1.

Table 5.8.1.1 Soil Parameters Used in Slope Stability Analyses

Soil Type	Short-Term Analyses			Long-Term Analyses		
	ϕ (degrees)	c (kPa)	γ (kN/m^3)	ϕ' (degrees)	c' (kPa)	γ (kN/m^3)
Embankment Fill (approved earth material)	30	0	21.5	30	0	21.5
Embankment Fill (Rock Fill)	43	0	18.0-20.0	43	0	18.0-20.0
Sand Backfill (used to replace existing peat and other surficial unsuitable soils)	30	0	20.0	30	0	20.0
Rock Backfill (used to replace existing peat and other surficial unsuitable soils)	43	0	18.0-20.0	43	0	18.0-20.0
Sand	30-33	0	20-21	30-33	0	20-21
Silty Sand Till	33-34	0	21-22	33	0	21-22
Cobbles & Boulders in a gravel & sand mixture, some silt	35	0	21	35	0	21
Clay	0	10-50	16-17	24	2	16-17

Typical stability calculations for the embankments are presented in Appendix E8.

We understand that the use of earth fill embankments is preferred at this site. The results of our analyses show that where the subsurface conditions consist of granular deposits (i.e. generally south of Maple Leaf Road), normal 2H:1V side slopes can be utilized. To the south of Maple Leaf Road, the presence of the weak clay necessitates the use of flatter slopes, including a mid-height berm in one section along the proposed WBL.

Details of the recommended side slopes, mid-height berm and surcharge configurations are presented in Table 5.8.1.2.

Table 5.8.1.2 Recommended Embankment Side Slopes

Station	Design Height of Embankment	Recommended Side Slopes	Mid-Height Berm Width	Recommended Surcharge Height	Note
EAST BOUND LANES					
15 + 850	5.7 m	2H:1V			Start applying surcharge to bring its height to 0.6 m @ Station 15 + 880
15 + 880 to 16 + 330	6.5 – 10.5 m	2H:1V*	0	0.6 m	
16 + 330	6.5 m	2H:1V	0	0.6 m	Start flattening side slopes from 2H:IV to 4H:IV @ Station 16 + 400
16 + 400	4.8 m	4H:1V	0	0.6 m	
16 + 450	4.2 m	4H:1V	0	0.6 m	Start increasing surcharge height to reach 1.0 m @ Station 16 + 480
16 + 480	3.8 m	4H:1V	0	1.0 m	
16 + 540	2.9 m	4H:1V	0	1.0 m	
16 + 600	2.0 m	4H:1V	0	1.0 m	
WEST BOUND LANES					
15 + 850 to 15 + 950	2.0 – 5.5 m	2H:1V	0	0	
15 + 950	5.5 m	2H:1V	0	0	Start applying surcharge to bring its height to 0.6 m @ Station 16 + 000
16 + 000 to 16 + 260	6.6 – 10 m	2H:1V*	0	0.6 m	
16 + 260	8.0 m	2H:1V*	0	0.6 m	Start flattening side slopes from 2H:1V to 4H:1V @ Station 16 + 290

16 + 290	7.4 m	4H:1V	0	0.6 m	Start increasing surcharge height from 0.6 to 1.0 m @ Station 16 + 350
16 + 350	6.3 m	4H:1V	0	1.0 m	Start mid-height berm to reach 5.0 m width @ Station 16 + 400
16 + 400	5.0 m	4H:1V	5.0 m	1.0 m	
16 + 450	4.5 m	4H:1V	5.0 m	1.0 m	
16 + 500	4.2 m	4H:1V	5.0 m	1.0 m	Start reducing mid-height berm to zero @ Station 16+540 and increasing surcharge height to 1.2 m @ Station 16+540
16 + 540	2.9 m	4H:1V	0	1.2 m	
16 + 600	2.0 m	4H:1V	0	1.2 m	

* Provide a minimum 2 m-wide mid-height berm as per OPSD 202.010.

In essence, the slopes composed of suitable earth fill can be constructed using normal 2H:1V side slopes (providing an at least 2 m wide mid-height berm for embankment heights in excess of 8 m), from the north end of the project to Station 16 + 330 along the EBL and to Station 16 + 260 along the WBL. At these stations, the slopes should be gradually flattened to reach 4H:1V side slopes at Stations 16 + 400 and 16 + 290 for EBL and WBL, respectively. 4H:1V side slopes continue to the end of this stretch of the project.

In addition, our calculations show that a 5.0 m wide mid-height berm is required along the WBL between Stations 16 + 400 and 16 + 500. The construction of this berm should be started at Station 16 + 350, reaching the required 5.0 m width at Station 16 + 400; remaining 5.0 m wide between Stations 16 + 400 and 16 + 500 and gradually reducing in width to zero at Station 16 + 540.

In making these recommendations, we have assumed that all the peat, topsoil, otherwise organic or unsuitable soils will be removed and replaced with compacted suitable fill under the footprint of the embankment.

5.8.2 SETTLEMENT OF EMBANKMENTS

Foundation settlements for granular overburden soils can be expected to be completed relatively rapidly, while the clay deposit can be expected to undergo consolidation settlements that are in excess of the settlements for granular soils both in magnitude and time rate (i.e. consolidation of the clay will take a much longer period of time to substantially complete). As mentioned in earlier sections of this report; the anticipated settlements for the clay deposit will depend on mainly the following.

- Thickness of the clay deposit
- Stresses imposed by the embankments (i.e. height and side slopes of the embankment and type of material used, etc.)
- Relative thickness of the relatively more compressible zones/layers in the clay deposit

The time rate of settlement is influenced mainly by the thickness of the clay that determines the length of drainage path, hence the rate of settlement and the permeability of the deposit.

Our calculations show that to the north of the site, where the depth of the overburden is very shallow or non-existent, the settlement of the embankments will primarily be due to the settlement of the embankment under its own weight. South of Station 15 + 950 both the depth of the overburden and the height of the proposed embankments increase, thereby increasing the magnitude of expected settlements both due to foundation settlement and the settlement of the embankment fills under their own weight. The magnitude and time rate of settlements of the embankments under their own weight will depend to a large extent on the type of materials used to build the embankments and effectiveness of the compaction applied during the construction. In any event, assuming suitable, well compacted fill, the settlement of embankments under their own weight is estimated to range between about 20 mm on the north and sides (i.e. low embankment heights) to about 60 mm where the height of the embankments reaches 10 to 11 m near the CPR crossing area. The settlement of the foundation materials range from zero where there is no overburden (i.e. over rock outcrop) or 20 mm where the overburden depth is shallow (i.e. Station 15 + 850 EBL) to about 170 mm near the CPR crossing, bringing the magnitude of total settlements (combined foundations and embankment self weight) to in excess of 200 mm. To the south of the CPR crossing the height of the embankments gradually decrease, while the thickness of the overburden increases. The calculated total settlements in this general area ranges from about 200 mm at Station 16 + 180 to about 110 mm near the Maple Leaf Road crossing, where the height of the embankments drop to about 7.5 m.

As the subgrade soils to the north of Maple Leaf Road consist of essentially granular soils, these settlements can be expected to take place rapidly, some of which will be completed during construction of the embankments, while much of the remainder within two months thereafter. To effect these settlements prior to paving, we recommend a surcharge period of two months, together with a surcharge of 0.6 m.

South of Maple Leaf Road, where the presence of a weak and compressible clay deposit (with increasing thickness towards the south) was recorded, both the magnitude and the time rate of the foundation settlements increase, in spite of the fact that the height of the embankments decrease rather rapidly from about 7.5 m at the Maple Leaf Road crossing to about 2.9 m at Station 16 + 540. In this section the magnitude of the anticipated settlements along the WBL range from about 300 mm at Station 16 + 300 to a maximum of about 400 mm further south. A similar trend emerges along the EBL beyond about Station 16 + 350.

The time rate of settlement increases from several months to more than ten years (near the south limit) for substantial completion. In order to effect much of the settlements prior to paving of the highway, surcharging is recommended. However, due to the possibility of a foundation failure, only a limited surcharge height can be applied. For this reason, the height of the surcharge should be limited to about 1.0 m, where the combined height of the surcharge and the embankment will be about 4.0 m or less, as detailed in Table 5.8.1.2. With this approach if the surcharge is applied for a minimum period of six months between Maple Leaf Road and Station 16 + 350, the residual settlements should not exceed 50 mm. South of Station 16 + 350, after a surcharge application of one year, the residual settlements should not exceed 100 mm between Stations 16 + 350 and 16 + 450 and 150 mm to the south of Station16 + 450. These values can be decreased to 50 mm and 80 mm, respectively, if the surcharge is maintained for about two years. It should be pointed out that these residual settlements will take place gradually over a period of years and should therefore not represent a major concern. Alternatively, wick drains may be used to speed up the rate of settlements. The use of lightweight fill is another option but is not recommended based on cost and reliability.

It is recommended that surcharging be carried out with proper instrumentation for monitoring settlements in the areas south of Station 16 + 290 (WBL) and 16 + 370 (EBL). It is furthermore recommended that the surcharge be placed gradually (i.e. preferably at least 3 layers, starting from one end of the site and proceeding to the other end), to allow excess pore pressures to dissipate.

Embankments should be provided with a widened cross-section to allow for settlements of the underlying soils and any possible future minor grade raises. In this case, we recommend that the road platform should be widened by at least 2 m on each side of the centreline (total 4 m). This is also in accordance with the Northern Region Engineering

Directive NRE-98-200. In addition, certain amount of regarding may be necessary due to settlements, including the side slopes after the surcharging period.

5.8.3 CONSTRUCTION

For embankment construction, all organic or otherwise unsuitable materials should be removed within an envelope given by an imaginary slope no steeper than 1:1 under the entire embankment width including flattening to 4H:1V.

After stripping, the exposed subgrade should be inspected and approved by the Quality Verification Engineer appointed by the Contract Administrator. The subgrade may need to be compacted from the surface using a suitable compactor. This should, however, only be done at the discretion of the Quality Verification Engineer, as it may not be feasible to effect surface compaction due to site conditions (i.e. high water table), weak nature of the clay, etc. It is believed that because of the presence of high water table at the site dewatering will be required to facilitate the stripping and backfilling operations. This is normally achieved by gravity drainage and pumping from open sumps. Construction procedures will have to be cognizant of the soft nature of the clay at most borehole locations south of Station 16 + 300 (WBL) and 16 + 330 (EBL).

The type of material to be used for soil replacement (i.e. to replace the stripped soils to the existing original ground surface) should be compatible with the proposed embankment fills. For example, where rock fill is to be used for embankment construction, the existing soils beneath the existing ground surface should be replaced with rock fill. In this case, it is important that the rock fill be penetrated the exposed subgrade where the subgrade consists of weak clay (i.e. very soft to firm consistency) and the size of the rock fill should be limited to 400 mm.

Similarly, where the embankment is to consist of earth fill, granular fill, which is compactable, should be used (e.g. Granular 'B' material). In this instance, the first one to two lifts of the fill may need to be thicker than normal (i.e. thicker than 300 mm lifts).

The materials used for the construction of the embankment should consist of approved acceptable earth or rock fill. The earth 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 (SPMDD) (OPSS 206 and OPSS 501).

Rock fill construction should be as per OPSS 206, except it should be placed in lifts with thickness not exceeding 1.5 m. The final surface of rock fill material should be compacted by at least two additional passes and should be blinded with compacted fine fill material (or chinked) prior to installation of the road subbase layer.

Rock fill material of maximum nominal size of 400 mm could be used and should consist of pieces of hard and durable rock with no sign of decomposition. Concrete, masonry, brick and similar materials should not be used.

A geotextile separator is recommended between rock fill and any native soil surcharge in order to prevent infiltration of fine soils into the rock fill. Any silty soils left at the surface of the rock fill, after the surcharge is removed, could be potential cause of possible frost heaving of the pavement. The separator should comprise of a Class II non-woven geotextile as per OPSS 1860 with a Filtration Opening Size (FOS) of 50 to 100 μm or 150 mm of Granular 'B' layer.

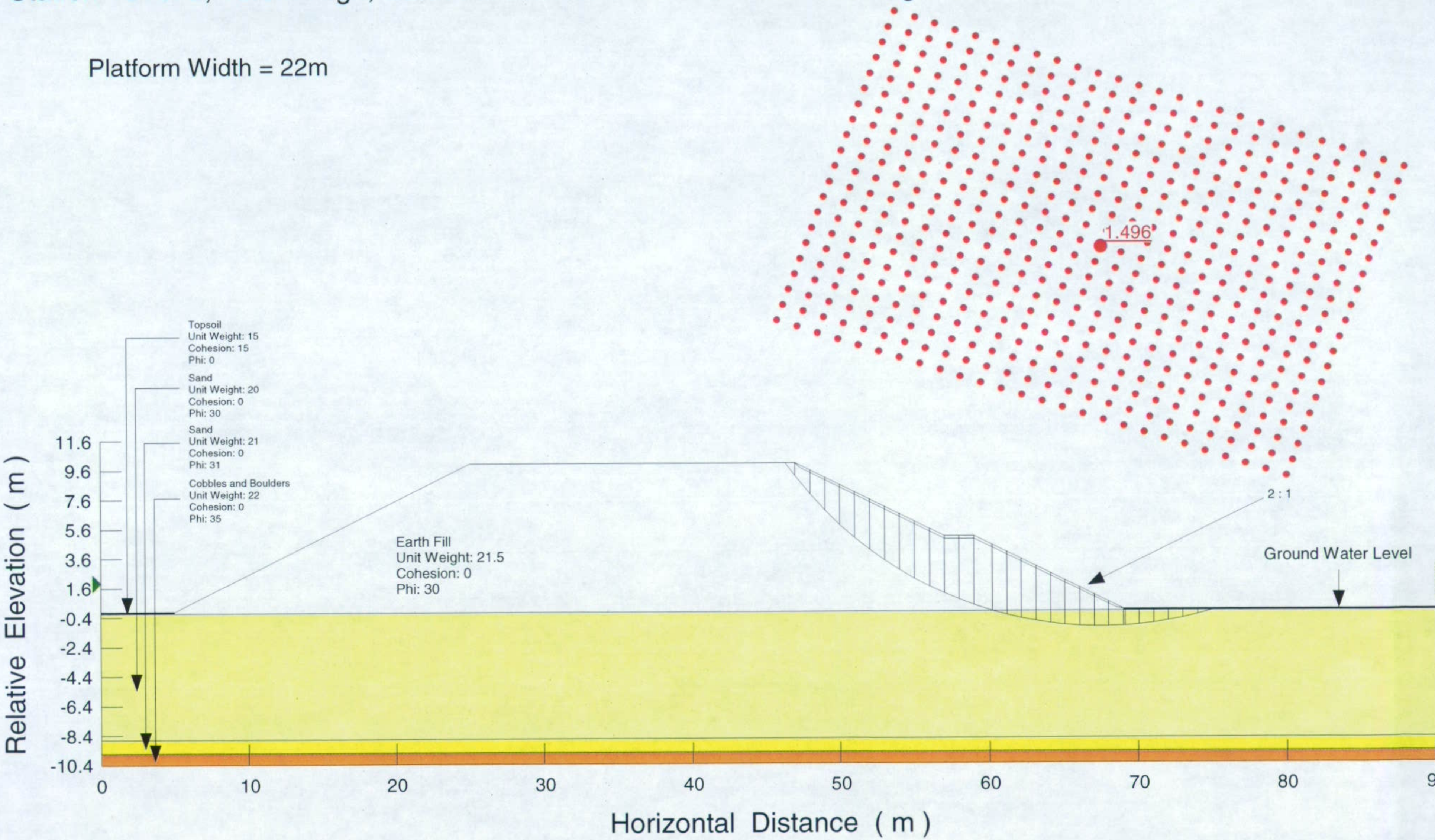
Proper erosion control measures should be implemented both during construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

APPENDIX E8

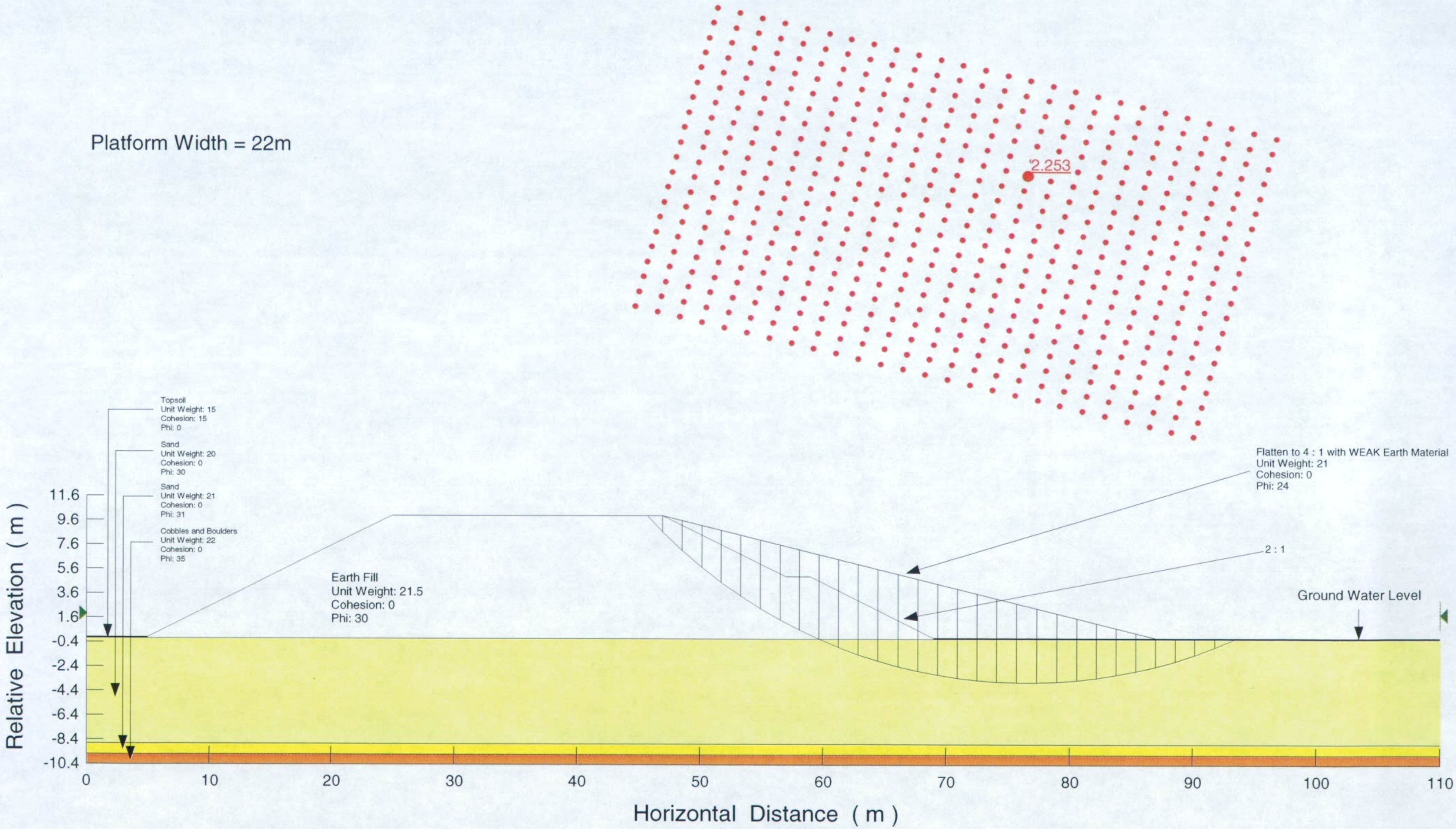
Slope Stability Analysis Results

SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 16+073, 10.0m High, Earth Fill Embankment with 2.0m Midheight Berm

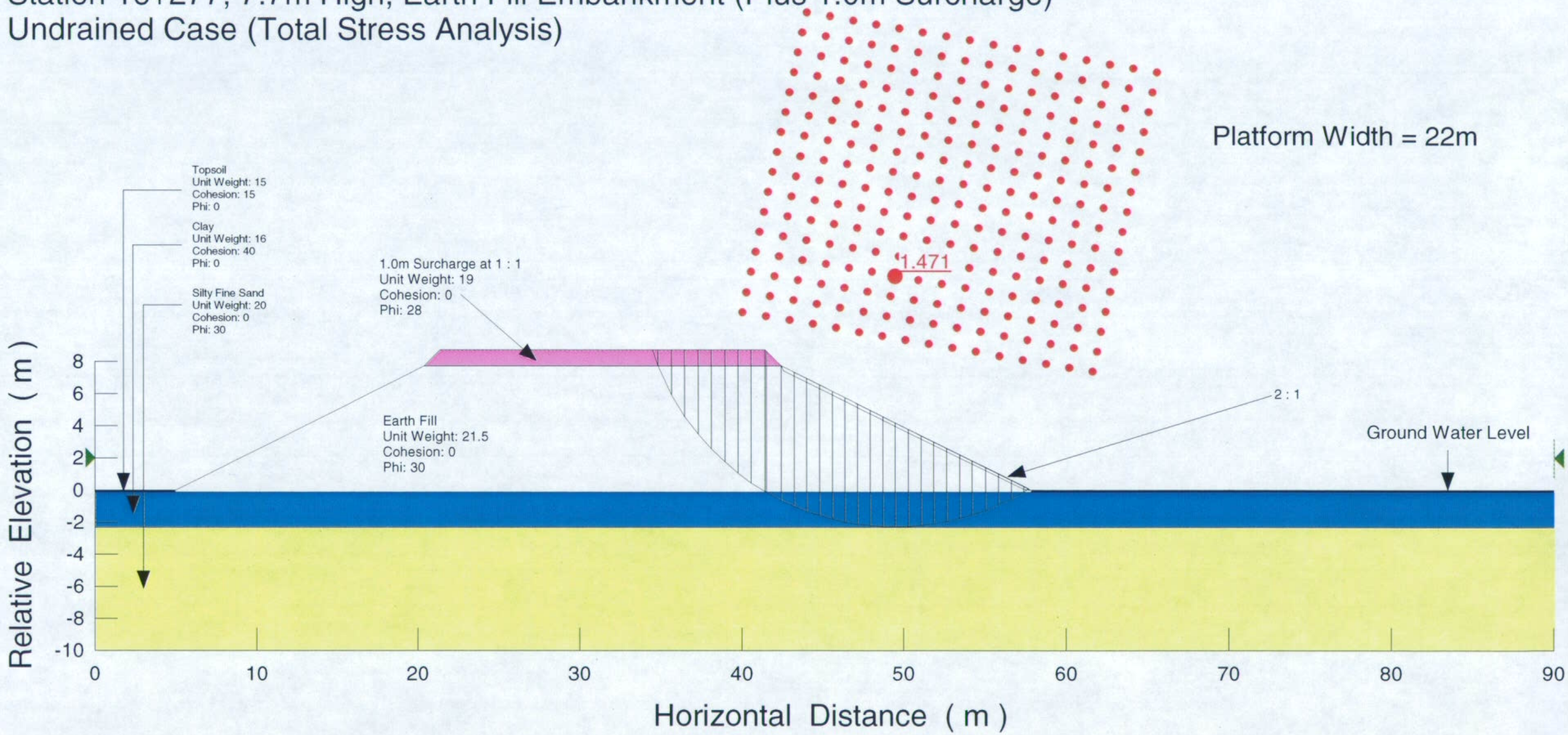
Platform Width = 22m



SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 16+073, 10.0m High, Earth Fill Embankment with 2.0m Midheight Berm

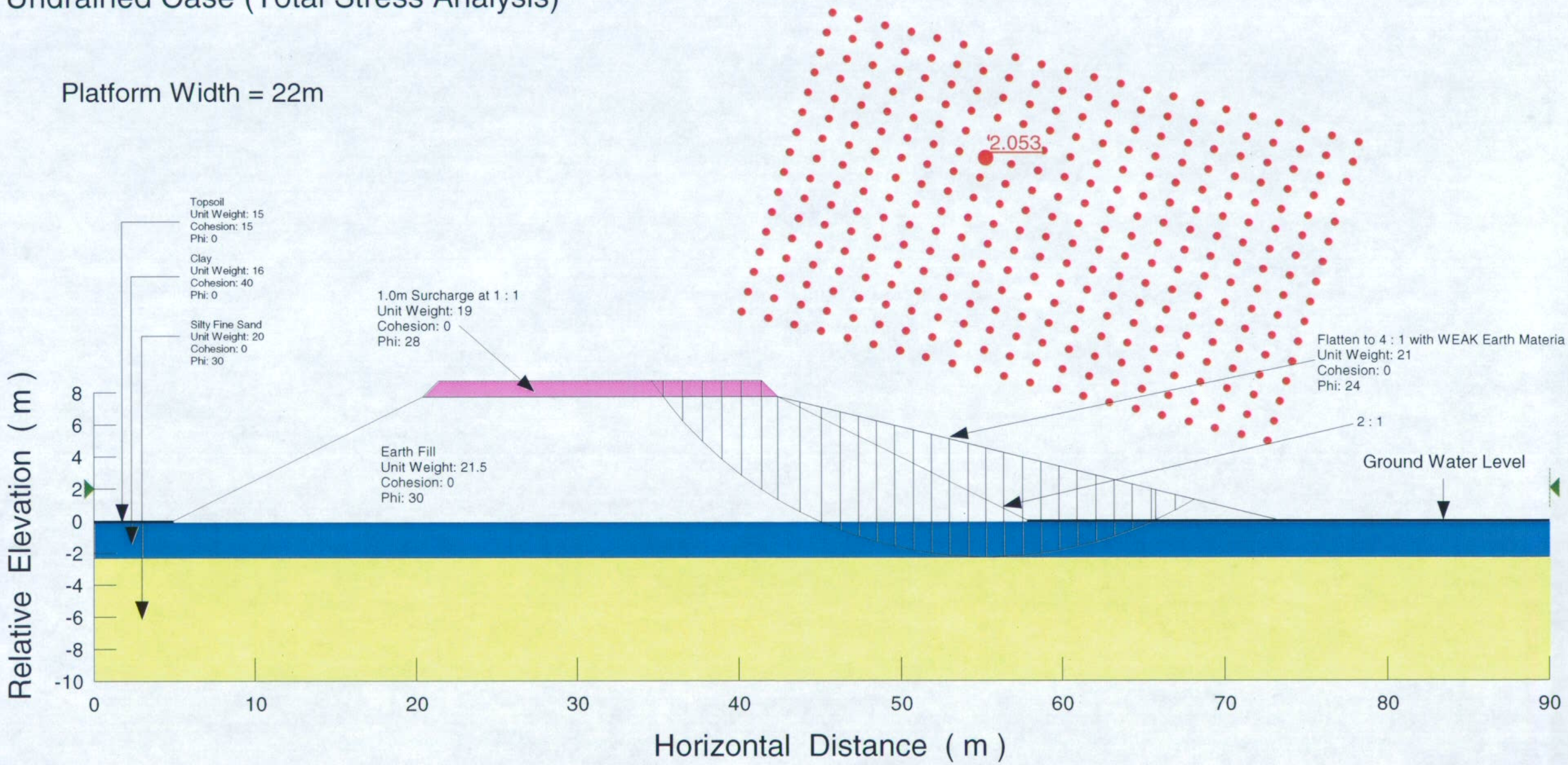


SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 16+277, 7.7m High, Earth Fill Embankment (Plus 1.0m Surcharge)
Undrained Case (Total Stress Analysis)

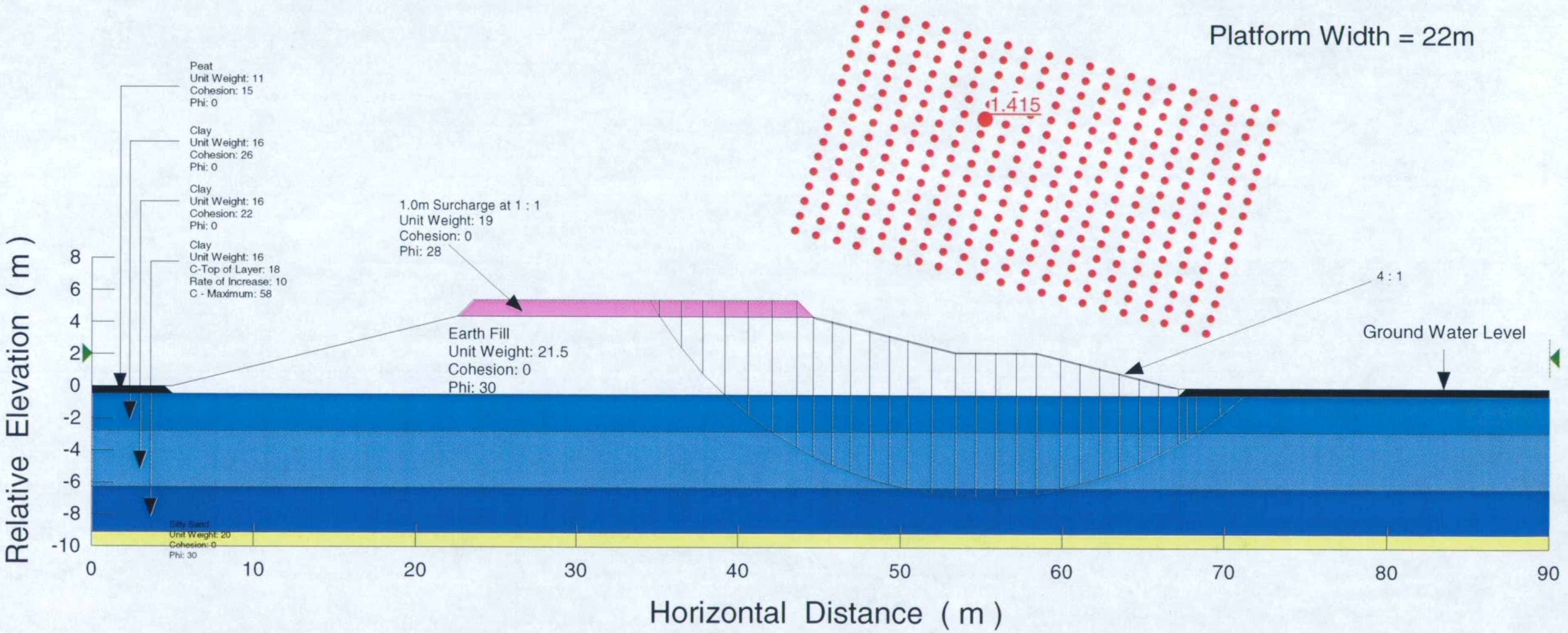


SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 16+277, 7.7m High, Earth Fill Embankment (Plus 1.0m Surcharge)
Undrained Case (Total Stress Analysis)

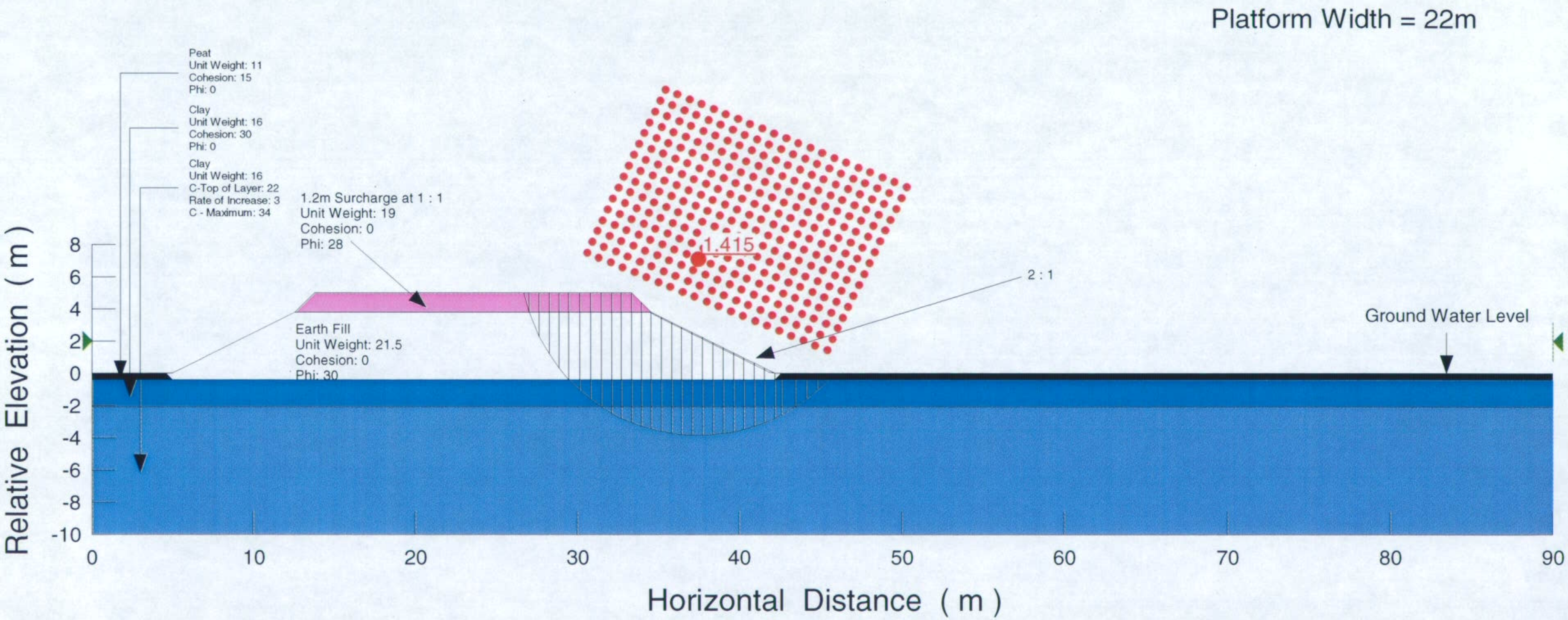
Platform Width = 22m



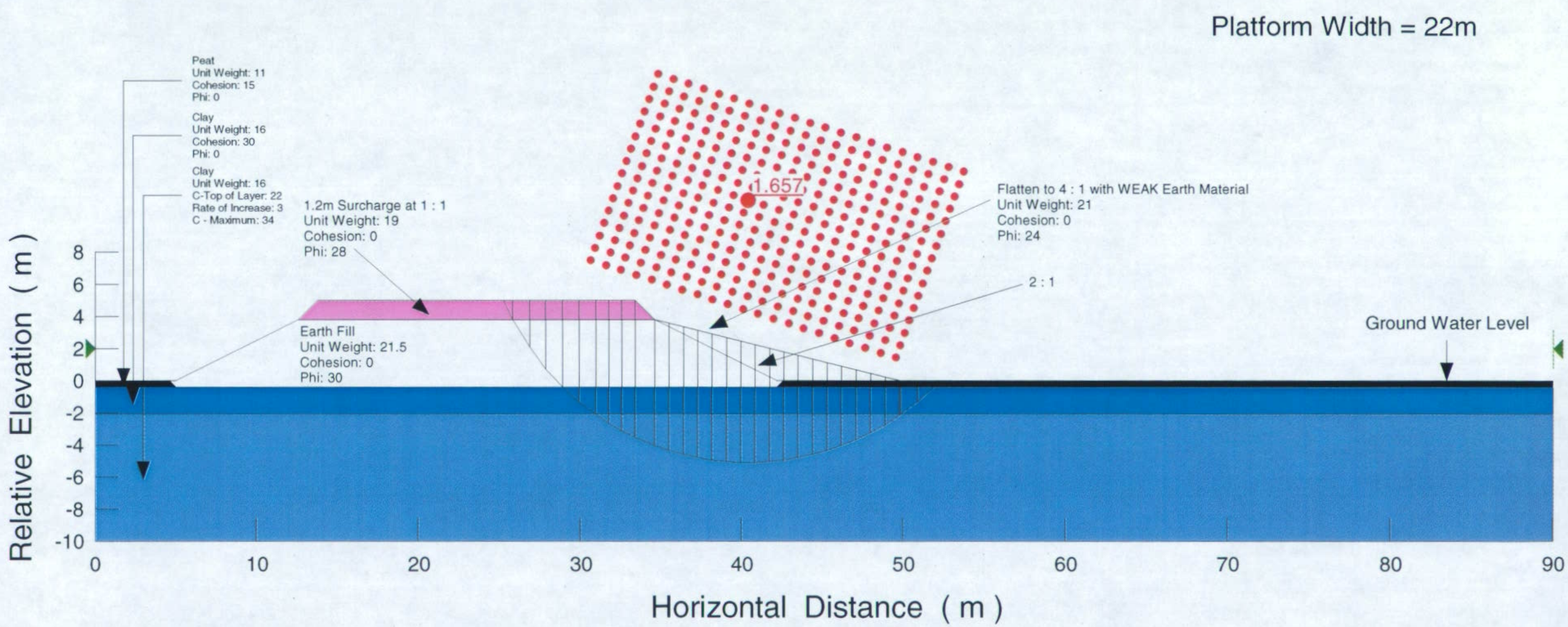
SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 16+470, 4.4m High, Earth Fill Embankment with 5.0m Mid-height Berm (Plus 1.0m Surcharge)
Undrained Case (Total Stress Analysis)



SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 16+480, 3.8m High, Earth Fill Embankment (Plus 1.2m Surcharge)
Undrained Case (Total Stress Analysis)



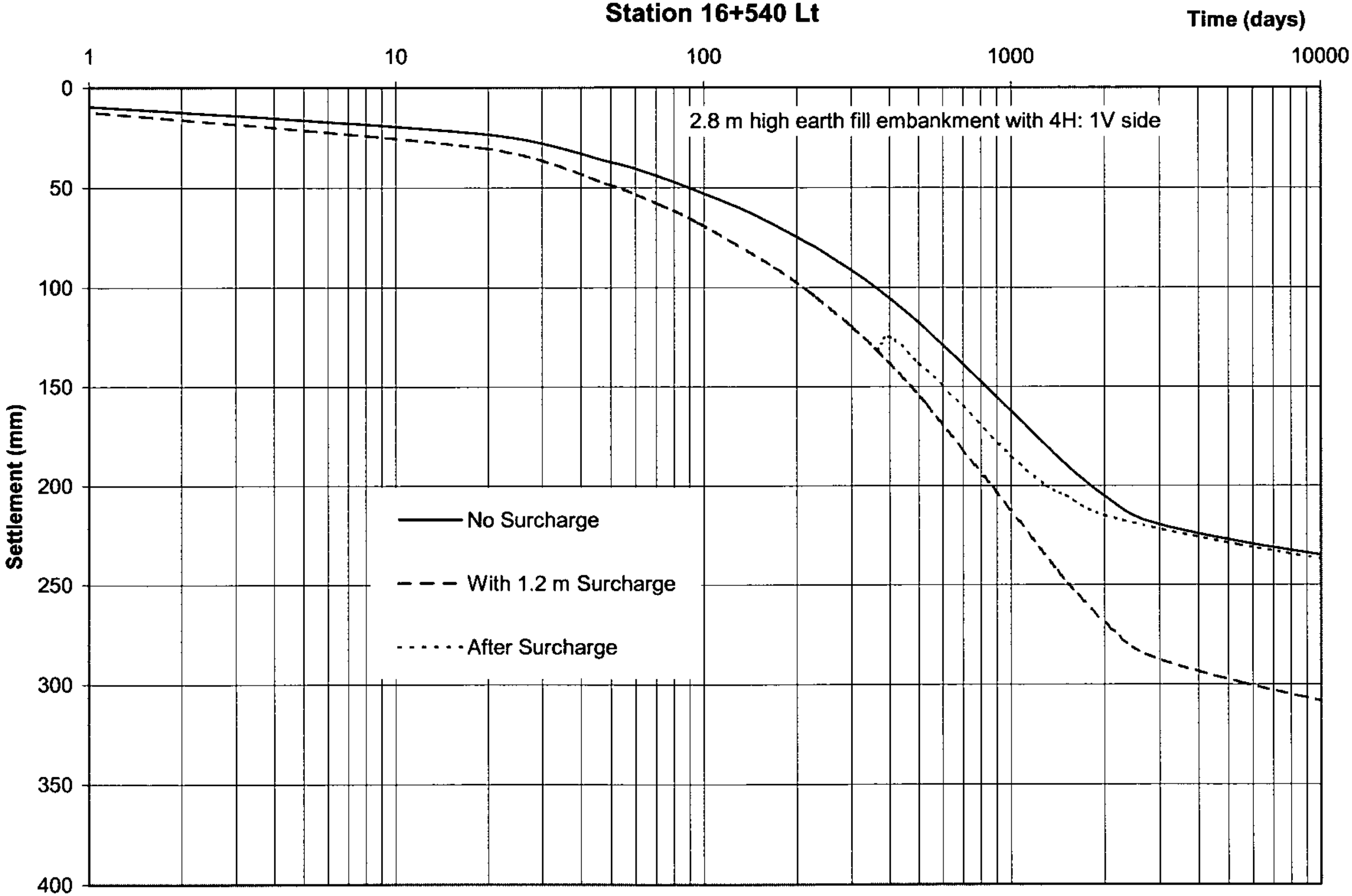
SPT 1055, Highway 17 (New), Sault Ste.Marie
Station 16+480, 3.8m High, Earth Fill Embankment (Plus 1.2m Surcharge)
Undrained Case (Total Stress Analysis)



APPENDIX F8

Time-Settlement Curve

Time - Settlement Curve
Station 16+540 Lt



5.9 SITE NO. 9 : GOVERNMENT ROAD CUT SECTION BETWEEN STATIONS 11+000 AND 11+220

Between about Stations 10+965 and 11+220, the profile drawing indicates that up to about 6 m of earth cut is anticipated along the centerline of the road, but only up to about 5 m of cut could be expected to the left of the road on a sloping rock. The cut material consists of surficial topsoil, clayey silt, sand and gravel and/or silty sand till with occasional cobbles and boulders, and bedrock. We understand that the rock between Highway 17 (New) and Government Road alignments will be essentially removed.

5.9.1 STABILITY OF CUT

From the cross-section in Drawing 9D, the cut slope to the left of the road could consist mainly of compact to dense silty sand till wedge over the bedrock (as shown for Station 11+120).

Slope stability analyses were conducted on a 5 m high earth cut slope consisting of sand and gravel / silty sand till with cobbles and boulders on the left side of Station 11+120. The bedrock surface is estimated to be at 5H : 1V slope or flatter, dipping to the right.

The analyses were performed using limit state equilibrium (Bishop's Simplified Method by the computer program Slope/W) and the following soil parameters were used:

Table 5.9.2.1 Soil Parameters Used in Slope Stability Analyses

Material Type	Short-Term Analysis			Long-Term Analysis		
	ϕ (degrees)	c (kPa)	γ (kN/m^3)	ϕ' (degrees)	c' (kPa)	γ (kN/m^3)
Sand and Gravel or Silty Sand Till	35	0	21.5	35	0	21.5

Due to the height of cut, the recommended minimum factor of safety for a stable slope in this cut area is 1.40. Typical cut slope stability sections are presented in Appendix E9.

Based on the above, in the case of a soil wedge over the bedrock, our slope stability analysis shows that the soil wedge is stable at 2H : 1V side slope, assuming no groundwater is present along the slope. If, however, groundwater (possibly from surface water infiltration) is present and assuming that the water level within the soil wedge is at about 0.5 m above the bedrock surface, the cut slope is expected to be stable at 2.5H:1V. Therefore, to account for this condition, we recommend a cut slope of 2.5H:1V in this section.

The quartzite bedrock in this section is considered stable at steep cut slope of 0.25H:1V, provided that the rock within the cut zone is not steeply jointed; or if steeply jointed, the joints are not are not dipping towards the highway. The sandstone bedrock is relatively weaker than the quartzite bedrock but it is still considered 'strong' and therefore, a 0.25H:1V rock cut slope could also be utilized, provided that a wider clear zone or larger catchment area is provided for cuts higher than about 3 m. It is recommended that the exposed rock should be inspected by a Rock Engineer during construction to assess its stability and provide any corrective measures, if required. It should be ensured that no rocks are overhanging along the steep slope. If this rock was found to be fractured and/or unfavourably jointed or unstable, corrective measures such as grouted dowels and/or rock anchors should be implemented.

If a Rock Engineer or Geologist will not be utilized to inspect the exposed rock, it is recommended that reinforcing of joints with dowels or rock bolts should be provided in the Contract Documents through an NSSP. Any rock bolt design and/or grouting should be illustrated on the contract drawings in addition to including specifications in the contract documents. Alternatively, the rock could be cut at a flatter slope such as 0.5H :1V even for the quartzite rock and/or clear zone could be increased.

5.9.2 CONSTRUCTION

Considering that the groundwater table was not encountered at this site, no major problem with groundwater seepage is anticipated. Surface water run-off should however be controlled by interceptor ditch along the top of the slope.

Based on the measured moisture contents, the cut materials are expected to be generally damp and are therefore suitable for re-use for fills.

Blasting of the rock should be carefully controlled so as not to produce an overbreak which could cause instability of the rock face. As mentioned before, the stability of the rock should be assessed by an experienced Rock Engineer during construction.

6. CLOSURE

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

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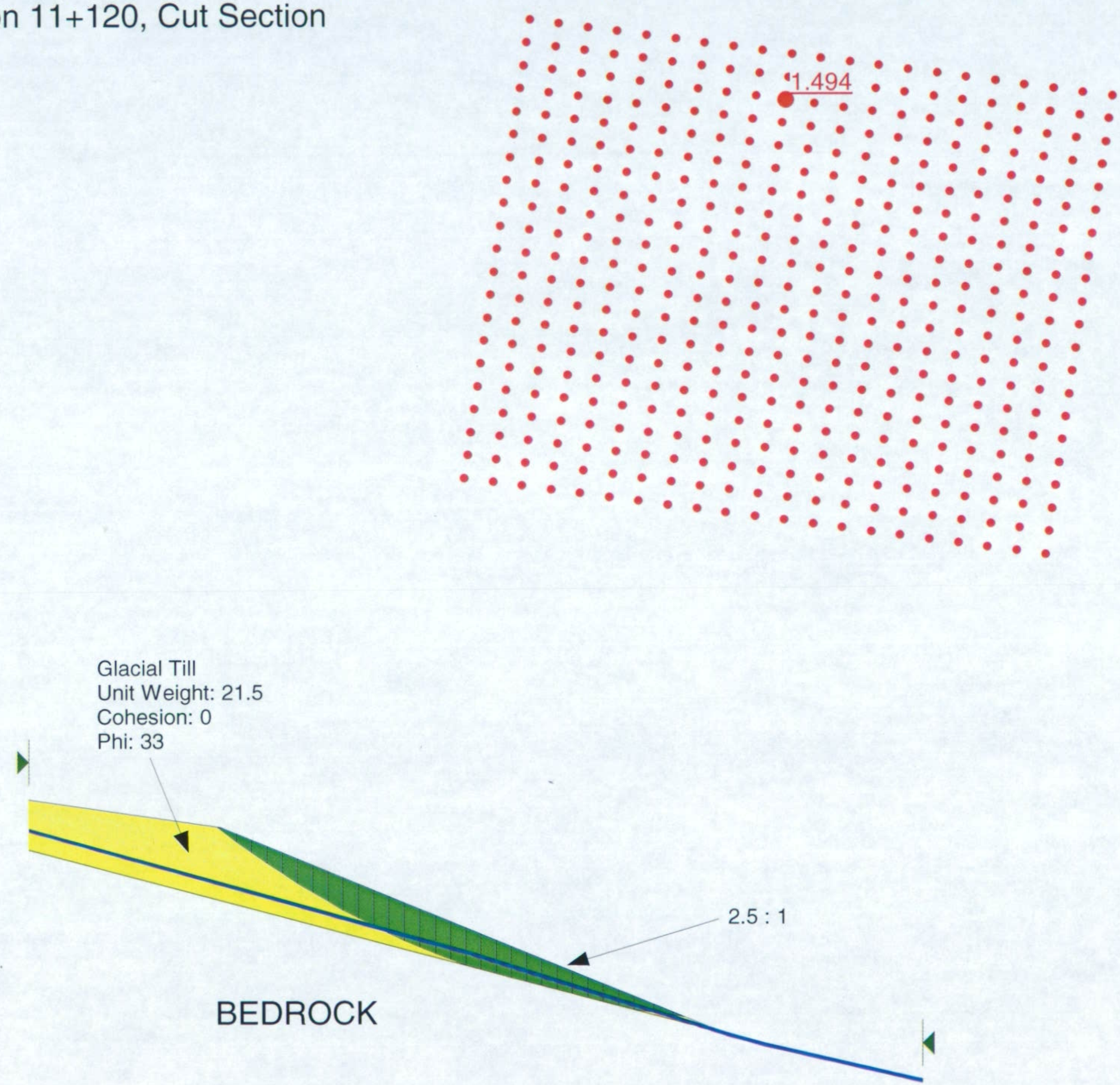
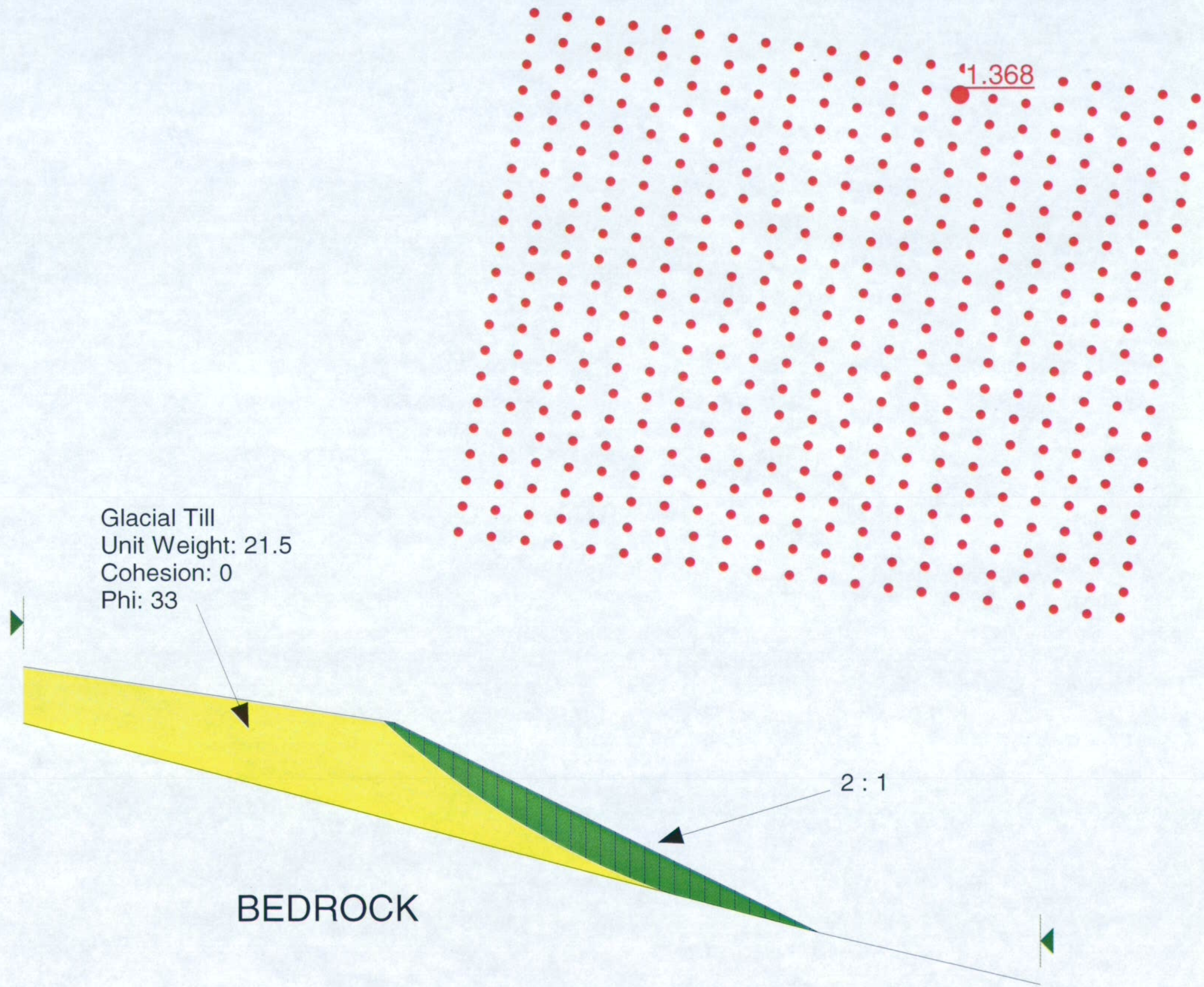
APPENDIX E9

Slope Stability Analysis Results

SPT 1055, Highway 17 (New), Sault Ste.Marie (for Government Road)
Station 11+120, Cut Section



SPT 1055, Highway 17 (New), Sault Ste.Marie (for Government Road)
Station 11+120, Cut Section



APPENDIX G

Limitations of Report

LIMITATIONS OF REPORT

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.