FOUNDATION INVESTIGATION AND DESIGN REPORT
HWY 11/17, PROPOSED WESTBOUND PASSING LANE AT GOLF COURSE ROAD, TOWNSHIP OF NIPIGON
MINISTRY OF TRANSPORTATION (MTO), NORTHWESTERN REGION
GWP NUMBER 6057-07-00 / W.P. 521-00-06
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MTO GEOCRES No. 52A-135

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ADDENDUM

1.0 INTRODUCTION

AMEC Earth & Environmental, a Division of AMEC Americas Limited (AMEC), Consulting Geotechnical, Construction Quality Control and Environmental Engineers, was retained by the Ministry of Transportation (Northwestern Region) to conduct a foundation investigation for the construction of a proposed new westbound passing lane along Highway 11/17 at its intersection with Golf Course Road, west of Nipigon, Ontario as indicated on Drawing No. 1. The project included widening of the highway to accommodate a westbound passing lane and left and right turn lanes at Golf Course Road.

Twelve (12) borehole and six (6) Dynamic Cone Penetration Testing (DCPT) locations advanced to the required depths for foundation investigation in the vicinity of the proposed embankment widening across two swamps and two culvert locations were specified by MTO in the Terms of Reference as outlined in the Request for Quotation (Agreement Number: 6008-E-0027 dated March 2008). Authorization to proceed with this investigation was signed by the Regional Director of MTO dated 24 December 2008. The work was carried out by AMEC according to the MTO Terms of Reference and AMEC's Proposal No. P28304 dated 17 December 2008.

Additional information (i.e., new construction plan, profile and previous borehole data) was also forwarded to AMEC. Such information was used in preparing this report, wherever applicable.

The investigation was carried out by means of a limited number of boreholes, in-situ tests and laboratory tests on selected samples. The factual results of the soil conditions encountered in the boreholes and laboratory tests together with design discussion and recommendation are presented in this report.

The factual results of the foundation investigation are presented in a separate report (i.e., Foundation Investigation Report prepared by AMEC - Reference No. TT93000 dated 27 March 2009).

2.0 SITE AND PROJECT DESCRIPTION

The site for the foundation investigation was along the north (left) side of the existing Highway 11/17 at Golf Course Road, approximately 200 m east to 300 m west of the intersection, to the west of Nipigon, Ontario, as shown in the site location plan (Drawing No. 1). Investigation was also completed on the south (right) side of the highway at the locations of two (2) proposed culvert replacements.

A series of site photographs taken during the field investigation works are presented in Appendix C.

At this location, Highway 11/17 is a two-lane asphalt-paved road with gravel shoulders on both sides and on top of an embankment with a height of roughly 2.5 m to 5 m above the surrounding grade. There appeared to be two swamp areas, one in the vicinity of the culvert at approximate Station 18+730, and the other between approximate Station 18+970 and Station 19+050. At the

time of the field investigation, the ground was covered with snow and frozen soils such that visual identification of the swamp areas was difficult. Another culvert was located at approximate Station 19+100.

Between approximate Station 18+850 and Station 18+950, a rock fill berm was present along the toe of the road embankment. Rock outcrops were visible in the area between approximate Station 18+765 and Station 18+010, west of Station 18+700 and east of Station 19+225.

The existing pavement contained numerous longitudinal and transverse cracks as evidenced in Photographs 9 and 10 in Appendix C.

The proposed widening works are planned to accommodate a westbound passing lane and left and right turn lanes at Golf Course Road, between Station 18+700 to Station 19+200.

3.0 QUATERNARY GEOLOGY

Based on Ministry of Northern Development and Mines Quaternary data (OGS Special Volume 4, Part 2), the project site lies within what is known as the Port Arthur Hills formed by the differential erosion of sills, dikes and sediments tilted slightly to the south to form mesas, cuestas and irregular hills up to 500 m in elevation. The height of the features tends to decrease southward from the Lake Nipigon area to the International Boundary. Sills tend to dominate the topography throughout most of the Port Arthur Hills.

4.0 INVESTIGATION PROCEDURES

4.1 Field Investigation

In accordance with the Terms of Reference for this investigation, twelve (12) borehole locations (BH 1 to BH 12) and six (6) DCP testing locations in the vicinity of the proposed widening were to be investigated for foundation investigation, including two (2) boreholes (BH 2 and BH 10) for culvert replacement investigation.

The fieldwork was performed from 12 January 2009 to 18 January 2009, and included staking out the borehole locations, clearing underground utilities, acquiring all necessary permits for road occupancy, preparing access for drilling rig, and drilling the boreholes using a track-mounted drilling rig as supplied and operated by Determination Drilling and Soil Investigations.

The drilled borehole locations were surveyed in the field by AMEC personnel based on a local benchmark (Ministry of Natural Resources benchmark MNR 010806106 with reported geodetic elevation of 269.567 m).

The borehole locations are presented on Drawing No. 2. The coordinates and the geodetic ground surface elevations at the specified borehole locations were surveyed by the on-site AMEC

personnel subsequent to drilling.

The borehole investigation was carried out under the full-time supervision of experienced geotechnical personnel from AMEC (Shami Malla, P.Eng. and Muhammad Saleem, EIT).

The boreholes were sampled to depths ranging from 6.6 m to 11.1 m below the existing ground surface using a combination of solid and hollow stem augers followed by Dynamic Cone Penetration testing beyond the sampled portions to achieve total depths ranging from 8.8 m to 16.7 m below the existing ground surface. Dynamic Cone Penetration test holes (DCPT) locations were advanced to depths ranging from 7.6 m to 11.1 m below the existing ground surface. A summary of the borehole and DCPT locations and achieved depths are presented in the following Table 4.1:

Table 4.1 – Summary of Borehole and DCPT Locations

LOCATION	STATION	OFFSET (m)	NORTHING(1)	EASTING(1)	SAMPLED DEPTH (m)	DCPT(2) DEPTH (m)
BH 1	18+722	21.9 m Left	5428993	207270	8.1	9.9
BH 2	18+731	18.0 m Right	5428973	207305	10.2	15.1
BH 3	18+740	21.2 m Left	5429005	207281	6.6	16.7
BH 4	18+875	20.2 m Left	5429115	207361	10.7	-
BH 5	18+925	22.7 m Left	5429155	207393	11.1	12.0
BH 6	18+970	19.8 m Left	5429188	207422	8.8	-
BH 7	19+010	17.4 m Left	5429215	207445	9.9	12.4
BH 8	19+050	18.3 m Left	5429251	207468	9.4	-
BH 9	19+102	16.8 m Left	5429297	207496	9.8	12.9
BH 10	19+102	13.3 m Right	5429280	207521	10.0	11.1
BH 11	19+150	14.7 m Left	5429336	207517	8.1	9.0
BH 12	19+200	17.2 m Left	5429383	207533	9.6	10.3
DCPT 1	19+176	18.8 m Left	5429361	207523	-	11.0
DCPT 2	19+129	15.9 m Left	5429317	207507	-	10.1
DCPT 3	19+070	16.4 m Left	5429269	207481	-	11.1
DCPT 4	19+033	17.7 m Left	5429236	207459	-	7.6
DCPT 5	18+999	20.0 m Left	5429207	207436	-	10.2
DCPT 6	18+963	20.6 m Left	5429183	207417	-	10.0

Notes: (1) HORIZONTAL DATUM: North American Datum 1983 (NAD83)
3 Degree Modified Transverse Mercator - (MTM) Grid Coordinates, MTM Zone14
(2) DCPT depth beyond sampled portion of boreholes and total depth for DCPT locations

At Boreholes BH 4 and BH 5, where rock fill was present at ground surface, excavation to the bottom of the rock fill (depth of about 4.6 m) was accomplished using an excavator with subsequent

augering completed below these depths through a 12 inch (300 mm) corrugated plastic pipe that was installed and backfilled into the rock fill.

Soil samples were normally taken at 0.75 m intervals up to a depth of 6 m and 1.5 m intervals thereafter, during the performance of Standard Penetration Test (SPT) in accordance with ASTM D1586. This consisted of freely dropping a 63.5 kg (140 lbs.) hammer for a vertical distance of 0.76 m (30 inches) to drive a 51 mm (2 inches) diameter O.D. split-barrel (split spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m (12 inches) was recorded as SPT 'N' value of the soil, which indicated the consistency of cohesive soils or the relative density of non-cohesive soils.

DCPT was carried out at six (6) locations (DCPT 1 to DPCT 6) and below the augered depth at a number of the boreholes locations by advancing a steel cone into the ground with a 63.5 kg (140 lbs.) hammer and a drop height of 0.76 m. The number of blows per 0.3 m required to advance the cone was recorded and presented in the Record of Boreholes / DCPT Test Holes (Appendix A).

Where soft to very soft clayey soils were encountered as indicated by low SPT 'N' values, MTO Field Vane Tests were carried out intermittently in the boreholes drilled by the track-mounted drilling rig.

Soil samples were collected for each soil layer exposed in the boreholes for laboratory inspection and testing. Split-spoon samplers and thin-walled samplers (Shelby tubes) were both used for sampling.

The collected soil samples were screened on site by a gas detector for the total organic vapours (TOV), the results of which are shown in the Record of Boreholes / DCPT Test Holes of Appendix A.

Upon completion of drilling, the boreholes were backfilled with bentonite in accordance with the general requirements of Ministry of the Environment Regulation 903, as indicated in the Record of Boreholes / DCPT Test Holes of Appendix A.

The soil samples were transported to AMEC's Advanced Soil Laboratory in Scarborough (Toronto) for further examination and laboratory soil testing. The program of laboratory testing included grain size analysis, Liquid and Plastic Limit determination, in-situ water content determination, one-dimensional consolidation analysis with creep testing and soil corrosivity testing.

The results of the in-situ and laboratory tests are presented in the corresponding Record of Boreholes / DCPT Test Holes (Appendix A) and Laboratory Test Results (Appendix B).

4.2 Laboratory Tests

Representative soil samples were subject to laboratory testing in AMEC's Advanced Soil Laboratory in Scarborough (Toronto) for soil classification. The following laboratory tests were conducted:

- In-situ water content determination (82);
- Grain size distribution analyses (12);
- Liquid and Plastic Limits (24);
- Laboratory Vane Tests (3);
- One-Dimensional Primary Consolidation tests (2); and
- One-Dimensional Secondary Consolidation tests (2).

The results of the laboratory tests are included in the Record of Boreholes / DCPT Test Holes in Appendix A, where applicable.

The grain size distribution curves, Liquid/Plastic Limits and results of the primary consolidation and secondary consolidation (creep) tests are presented in Appendix B.

5.0 SUB-SURFACE CONDITIONS

In brief, the soil profile typically consisted of either organic materials (peat or topsoil) or fill soils overlying native soils, though native soils were encountered immediately at ground surface at a number of the borehole locations. The native soils consisted of a combination of sand, silty sand / sandy silt, clayey silt / silty clay and sand and gravel, the majority of which were clayey silt/silty clay.

With the exception of Borehole BH 4 that was dry upon completion of drilling, groundwater was encountered in all boreholes ranging in depth from ground surface to about 4.0 m below the existing ground surface. A variance to this general assessment was at Borehole BH 11, where artesian water pressure corresponding to a groundwater level of about 0.3 m above ground surface was encountered after the borehole was drilled through the sand and gravel underlying the clayey silt/silty clay. High groundwater levels observed in the boreholes were typically encountered when the boreholes were drilled through the clayey silt/silty clay.

The results of DCPT as completed at the additional six (6) DCPT locations are presented in the Record of Boreholes / DCPT Test Holes (Appendix A) and are also plotted in Drawing Nos. 3 and 4. The results of the DCPT indicated the presence of very soft/loose soils to depths of from about 7 m to 10.5 m below the existing ground surface with cone refusal encountered at depths of between 7.5 m and 11.0 m below the existing ground surface.

The stratigraphic units and groundwater conditions at the borehole locations are discussed in the following sections. Detailed information is provided in the Record of Boreholes / DCPT Test Holes

(Appendix A) and presented on the longitudinal and transverse soil profiles of Drawing Nos. 3 and 4 respectively. Previous borehole information, as provided by the MTO, is included in Appendix D.

The following summary is to assist the designers of the project with an understanding of the anticipated soil conditions across the site. However, it should be noted that the soil and groundwater conditions may vary between the borehole locations.

5.1 Organic Materials

Deposits of organic materials consisting primarily of peat were encountered immediately at ground surface at Boreholes BH 2, BH 3, BH 6, BH 9 and BH 10, and below a surface deposit of sandy silt (Section 5.3) at Borehole BH 1 to depths ranging from about 0.6 m to 3.7 m below existing ground surface. Topsoil, approximately 0.9 m in thickness, was encountered at the existing ground surface in Borehole BH 7.

The black peat was fibrous and contained some wood fragments.

The thickness of organic materials may vary between boreholes. For accurate quantity estimates, if required, a regular interval of shallow test holes should be excavated to measure the thickness of the organic materials.

The results of laboratory tests conducted on soil samples are as follows:

Natural moisture content (%): 70 to 569

5.2 Rock Fill

Rock fill was encountered at Boreholes BH 4 and BH 5 from ground surface to about 4.6 m below the existing ground surface. The rock fill ranged in sizes from about 150 mm to 1500 mm and was mixed with sand and gravel.

5.3 Sandy Silt / Silty Sand

Native sandy silt / silty sand deposits were encountered immediately at ground surface at Boreholes BH 1, BH 8, BH 11 and BH 12, below the topsoil at Borehole BH 7, and below the peat at Boreholes BH 1, BH 2, BH 6, BH 9 and BH 10. These deposits were present to depths of between 0.6 m and 4.3 m below the existing ground surface. In addition, a deposit of sandy silt was encountered below the clayey silt / silty clay (Section 5.4) in Borehole BH 4 at a depth of 8.8 m below the existing ground surface and present to the termination depth of 10.7 m.

The sandy silt / silty sand was yellowish-brown/brown to grey in colour and contained a trace of gravel and a trace to some clay with occasional rootlets, wood fragments and organic matters.

Some clayey zones were present within the sandy silt / silty sand at Borehole BH 6 as evidenced by the Plastic Limit / Liquid Limit data presented below.

The SPT 'N' values within the sandy silt / silty sand ranged between 0 and 14 blows per 0.3 m with the majority of values being less than 10 blow per 0.3 m indicating a generally loose to very loose relative density.

The results of laboratory tests conducted on soil samples are as follows:

Natural moisture content (%):	15 to 43	
Plastic Limit (clayey sandy silt zone):	15	
Liquid Limit (clayey sandy silt zone):	27	
Grain size (2 samples):	Gravel (%):	2 and 4
	Sand (%):	31 and 63
	Silt (%):	31 and 49
	Clay (%):	4 and 16

The grain size distribution curves are presented in Figure No. B1 with plasticity index data plotted in Figure No. B4 of Appendix B.

5.4 Clayey Silt / Silty Clay

The top surface of native clayey silt / silty clay deposits was encountered underlying the sandy silt / silty sand in Boreholes BH 1, BH 2 and BH 6 to BH 12, and below the rock fill in Boreholes BH 4 and BH 5, at depths ranging from 0.6 m to 4.7 m below the existing ground surface.

With the exception of Boreholes BH 1, BH 4, BH 8, BH 10 and BH 12, where the clayey silt / silty clay was underlain by either sandy silt / silty sand, sand (Section 5.5) or sand and gravel (Section 5.6), the clayey silt / silty clay was present to the maximum sampled depth of the boreholes.

The presence of the clayey silt / silty clay was confirmed by sampling to depths ranging from about 5.9 m to 11.1 m below the existing ground surface. Though not confirmed by sampling, results of DCPT testing indicate the possible presence of clayey silt / silty clay to depths of up to 11 m to 14 m below the existing ground surface.

The clayey silt / silty clay was brown to grey in colour and contained a trace of sand and gravel. A boulder was possibly encountered within the clayey silt / silty clay in Borehole BH 1.

The SPT 'N' values of the clayey silt / silty clay ranged from 0 to 8 blows per 0.3 m with the majority of values being less than 4 blows per 0.3 m. Results of a series of MTO Field Vane Tests completed in the field and laboratory vane tests, as completed on select samples of the clayey silt / silty clay, ranged between 11 kPa and 78 kPa, indicating an overall very soft to stiff consistency. The laboratory vanes tests were carried out on the collected Shelby tube samples using a Pilcon

hand vane tester with circumferential graduated scale for direct reading. The vane shear strengths obtained from laboratory vane ranged between 12 kPa and 75 kPa which are consistent with the field tests. A Bjerrum Correction factor of 0.95 (Plasticity Index range of 20 to 30) was applied to the field vane test results.

The results of laboratory tests conducted on soil samples are as follows:

Natural moisture content (%):	26 to 52	
Plastic Limit:	14 to 18	
Liquid Limit:	26 to 49	
Grain size (8 samples):	Gravel (%):	0 to 1
	Sand (%):	0 to 5
	Silt (%):	35 to 74
	Clay (%):	24 to 65

The grain size distribution curves are presented in Figure No. B2 with plasticity index data plotted in Figure Nos. B4 to B7 of Appendix B.

The results of two (2) one-dimensional consolidation tests and two (2) creep tests conducted on select soil samples are presented in the following Table 5.1:

Table 5.1 – Summary of Consolidation Test Results

Sample ID	Primary Consolidation Test[1]						Creep Test[2]
	c_c	p_c (kN/m^2)	e_0	e_f	c_v (m^2/day)	c_s	c_α
BH 1 - TW7	0.18	80	0.77	0.55	0.01 – 0.03	0.02	not tested
BH 6 - TW5	0.29	160	1.01	0.76	0.01 – 0.03	0.04	0.004 (80 kPa loading) 0.011 (160 kPa loading)

Notes: (1) c_c – compression index, p_c – preconsolidation pressure, e_0 – initial void ratio, e_f – final void ratio, c_v – coefficient of consolidation, c_s – swell index/rebound index
(2) c_α – secondary compression index

The void ratio vs. log pressure plots and coefficient of consolidation vs. log pressure plots for the primary consolidation tests as completed on Samples BH 1 - TW7 and BH 6 - TW5 are presented in Figure Nos. B8 and B9 respectively of Appendix B.

The c_c and c_s values as presented in Table 5.1 were derived from the slopes of the compression and rebound curves of the void ratio vs. log pressure plots with p_c determined using the Casagrande graphical method.

The secondary consolidation (creep) tests were completed on Sample BH 6 - TW5 at constant loadings of 80 kPa and 160 kPa to determine the potential creep settlement rate to be expected if primary settlement to a degree of consolidation of greater than about 90% is obtained. The void ratio vs. log time plots for the tests under 80 kPa and 160 kPa constant loadings normalized to the void ratio at the end of primary consolidation (e_p) are presented in Figure Nos. B10 and B11 respectively of Appendix B.

The c_α values as presented in Table 5.1 were derived from the slopes of the secondary linear portion of these curves using the final void ratios from the respective primary consolidation tests (e_f) as an estimate of e_p.

5.5 Sand

The top surface of native sand was encountered below the clayey silt / silty clay in Boreholes BH 1 and BH 12 at depths of 5.9 m and 7.0 m respectively, and below the peat in Boreholes BH 3 at a depth of 3.7 m. The sand extended to the termination depths of 8.1 m, 6.6 m and 9.6 m in Boreholes BH 1, BH 3 and BH 12 respectively.

The grey to reddish brown sand contained a trace of silt and gravel with a silty clay seam encountered within the sand at Borehole BH 3.

The SPT 'N' values of the sand within Boreholes BH 1 and BH 3 varied from 0 to 28 blows per 0.3 m indicating highly variable very loose to compact conditions. The low SPT 'N' values of the sand measured immediately below the clayey silt / silty clay could possibly be due to the significant flow of sand into the hollow stem augers under high groundwater pressure. The SPT 'N' values of 23 and 26 blows per 0.3 m were measured in the sand within Borehole BH 12 indicating predominately compact conditions.

The results of laboratory tests conducted on soil samples are as follows:

Natural moisture content (%):	11 to 24	
Grain size (2 samples):	Gravel (%):	2 and 9
	Sand (%):	88 and 94
	Silt (%):	3 and 4
	Clay (%):	0

The grain size distribution curves are presented in Figure No. B3 of Appendix B.

5.6 Sand and Gravel

The top surface of native sand and gravel was found underlying the clayey silt / silty clay deposits in Borehole BH 11 at a depth of 7.2 m below the existing ground surface and extending to the termination depth of the borehole of 8.1 m.

Though it could not be confirmed due to the loss of sample, possible sand and gravel was encountered below the clayey silt / silty clay in Borehole BH 8 at a depth of 8.7 m below the existing ground surface and extending to the termination depth of the borehole of 9.4 m.

The grey sand and gravel contained occasional cobbles at Borehole BH 11.

The SPT 'N' values of the sand and gravel of 43 and greater than 50 blows per 0.3 m indicated dense to very dense conditions.

A single water content measured in a sample of the sand and gravel was 12 %.

5.7 Groundwater

The groundwater level in each open borehole was observed during drilling and measured upon completion of drilling and cone penetration testing. The measured groundwater levels are shown in the Record of Boreholes / DCPT Test Holes (Appendix A).

Typically, the high groundwater was encountered when the borehole was drilled through the clayey silt / silty clay into the water-bearing sandy soil.

With the exception of Borehole BH 4 which was dry upon completion of drilling, groundwater was encountered in all the boreholes drilled as indicated in the following Table 5.2:

Table 5.2 – Groundwater Levels

Borehole No.	Groundwater Depth bgs[(1)] (m)	Elevation (m)	Borehole No.	Groundwater Depth bgs[(1)] (m)	Elevation (m)
1	1.7	266.2	**7**	0.6	255.8
2	1.4	265.5	**8**	0.3	255.7
3	0.8	266.7	**9**	0.5	255.5
4	Dry	-	**10**	0.0	255.8
5	4.0	255.9	**11**	0.3 ags[(2)]	256.8
6	0.3	256.4	**12**	0.6	257.1

Notes: (1) bgs – below ground surface
(2) ags – above ground surface (artesian)

It should be noted that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to major weather events.

6.0 LIMITED ENVIRONMENTAL INVESTIGATION

In accordance with the Terms of Reference and AMEC proposal, soil samples obtained during the geotechnical field drilling program were field screened for evidence of environmental impact.

The field screening activities included measuring the combustible organic vapours (COV) in the headspace of samples with a portable hydrocarbon surveyor instrument (Thermo Gastechtor 1238ME). The borehole locations were positioned at the site as required by the geotechnical investigation.

Based on the soil conditions encountered in all the boreholes, the soil profile at the site typically comprised either topsoil, peat or fill soils overlying native soils, though native soils were encountered immediately at ground surface at a number of the borehole locations. The native deposits consisted of a combination of sand, silty sand / sandy silt, clayey silt / silty clay and sand and gravel.

No visual or olfactory evidence of environmental impact was observed in the fill and native soil samples recovered from the boreholes. The measured COV concentrations in all soil samples were relatively low, ranging from non-detect to 140 ppm as shown in the Record of Boreholes / DCPT Test Holes. The COV results are semi-quantitative at best and are generally used only for relative sample comparison purposes when selecting samples for laboratory analysis. Based on the field screening results, evidence of environmental impact is not suspected.

7.0 SOIL CORROSIVITY

One soil sample (BH 9 - SS6) was analysed by AMEC's chemical laboratory in Mississauga to determine the soil corrosivity potential with respect to concrete and steel. The results are presented in the following Table 7.1:

Table 7.1 – Results of Corrosivity Testing

Soil Sample No.	pH	Resistivity (ohms-cm)	Chloride (µg/g)	Sulphate (µg/g)
BH 9 - SS6	7.7	4650	6	26

The tests have shown a sulphate value of 26 ppm (µg/g) and, according to Table 10 - "Requirements for Concrete Subject to Sulphate Attack", Clause 15.5.2, of CSA Standard Specification A23. 1-94, any soil which has sulphate concentrations below 1000 ppm is not considered corrosive with respect to concrete. As such, the general-use Cement Type 10 can be used. The soil resistivity measured is "moderate" while the pH value is close to neutral.

8.0 DISCUSSION AND RECOMMENDATION

8.1 Project Description and Requirements

According to the Terms of Reference for this project provided by MTO, a new passing lane embankment constructed to road subgrade or top of subbase is planned to be completed by 31 October 2009. Placing the Granular A base and paving of the main highway and the new auxiliary lanes have been scheduled for 2010. The Detail Design of the new westbound passing lane is being completed by MTO. Design recommendations are required for embankment widening across two swamps and two culvert replacements.

The project includes detail design for widening of Highway 11/17 west of Nipigon, in the vicinity of Golf Course Road between Station 18+700 to Station 19+200 (Drawing No. 2). A new westbound passing lane and left and right turn lanes at Golf Course Road will be constructed. The elevation of the new westbound passing lane would follow the elevation of the existing highway, resulting in an embankment height of about 2.5 m to 5.0 m above the surrounding grade. The new westbound passing lane would result in widening the existing road embankment and hereinafter will be referred to as “widened embankment”. MTO’s objectives are to produce highway embankment design such that the post-construction settlements are minimal according to the following design criteria:

- Maximum permissible total settlements of 100 mm;
- Differential settlements of 50 mm between the widened embankment and the existing embankment;
- The embankment change in grade due to differential settlements over any 5 m length of pavement must be limited to 0.5 % in both longitudinal and transverse directions of the embankment, i.e., 25 mm in 5 m differential settlement; and
- The design life of 10 years to be assumed for calculating embankment settlements.

The subsoil conditions of the widened embankment and culverts were investigated by a total of 12 boreholes and 6 dynamic cone penetration tests. The borehole locations were specified by MTO. Ten boreholes out of the twelve boreholes were located along the new westbound lane (north of the existing highway) while two boreholes were located at the existing culverts (south of the existing highway). No boreholes were drilled to the native/subgrade underneath the existing paved road surface or shoulder during this investigation. Soil information on the existing embankment is based on the results of previous investigations as presented in Appendix D. As such, slope stability analyses carried out to assess the stability of the widened road embankment assumed that the existing road embankment was composed of compacted granular soils and/or possible rock fill. Similarly, the settlement calculations were carried out for this report by assuming that the existing road embankment constructed some 20 to 30 years ago had completed consolidation settlement (if any). It should be noted that the existing paved road surface contained numerous transverse and longitudinal cracks at the time of investigation.

The results of slope stability analysis for the new widened road embankment and settlement calculations, together with recommendations are presented in the following sections. The slope stability analysis and the settlement calculations were done according to the revised cross section provided by MTO. The cross sections were analyzed considering the geometry revisions as forwarded by MTO to AMEC in their e-mail dated 2 March 2009.

8.2 Subsurface Soil Conditions

Based on the boreholes drilled along the proposed westbound lane, the soil profile consisted mainly of organic matters (peat and topsoil) and/or loose/soft soils overlying a layer of typically very soft to firm clayey silt/silty clay. Hard/very dense soil/possible bedrock indicated by auger/DCPT refusal was found typically underlying a water-bearing granular soil underneath the clayey silt/silty clay. The very soft to firm clayey silt/silty clay was approximately 2 m to 10 m in confirmed thickness. An approximately 4 m thick layer of rock fill berm material was present at the locations of Boreholes BH 4 and BH 5, possibly indicating the presence of historical soft soil/swamp at that location.

There could be a perched groundwater table at some borehole locations due to the presence of granular soils (silty sand/sand) overlying cohesive soils (clayey silt/silty clay). High groundwater level was encountered in the silty sand/sand layer underlying the clayey silt/silty clay which appeared to act as an aquitard. The groundwater level in the silty sand/sand layer indicated an artesian or sub-artesian groundwater condition in the project area.

The SPT 'N' values measured in the clayey silt/silty clay were typically 0 blows/0.3 m which reflected the fact that the split-spoon assembly sank into the clayey silt/silty clay by its own weight. The field and laboratory vane shear strengths measured in the boreholes ranged from about 11 kPa to 78 kPa as shown in Figure No. 1.

8.3 Slope Stability Analysis

From the results of the geotechnical investigation and the planned road embankment and cross-sections provided by MTO in 2 March 2009, slope stability analyses were carried out for design consideration. The slope stability analyses were performed by using the computer program SLOPE/W with the Bishop's Simplified method for circular slip surfaces.

Both undrained and drained analyses were considered using the following soil shear strength:

- Undrained shear strength – Measured field and laboratory vane shear strengths as shown in Figure No. 1 were used together with Bjerrum's Correction Factor (0.95) applied to the field obtained values. Undrained shear strengths of 15 kPa (very soft to soft clayey silt/silty clay) and 25 kPa (firm clayey silt/silty clay) were used, as shown in Figure No. 1, in analyzing the short term stability of the road embankment.
- Drained shear strength – For long term analysis, the clayey silt/silty clay was considered to be normally-consolidated. As such, the cohesion c' was zero and the angle of internal friction Φ' was considered to be 30°. The adopted angle of internal friction is based on the relationship published by Bjerrum and Simons between the soil plasticity index (PI) and typical peak friction angle for clays.

All shear strength parameters used in the slope stability analyses are shown on the results of the analyses (Appendix E).

The shear strengths used in the analyses were justified by the analyses of the existing road embankments.

The following cases were considered in the slope stability analyses:

- Stability of the existing road embankment – This is to determine the validity of the shear strength used in the analysis. The factor of safety (FOS) of the existing embankment against slope instability is calculated using the measured field vane shear strength and drained shear strength. The calculated FOS of the existing embankment will be the reference for calculating the FOS of the widened road embankment. For short-term stability, the FOS of the widened road embankment should be equal to or greater than that of the existing embankment.
- Stability of the widened road embankment at the end of construction – This is the crucial situation for the widened road embankment. The factor of safety of the widened road embankment is theoretically at its lowest at the end of construction – traffic load of 20 kPa is included. Two cases were analyzed, i.e., deep-seated (critical slip surface) and shallow slip surface within the road widening portion (i.e., slip surface that passes through the widened road embankment and the underlying soil subgrade immediately supporting the widened road embankment), for comparison.
- Stability of the widened road embankment configured as per OPSD 203.030 (Embankment Over Swamp – Existing Slopes Maintained) as presented in Appendix G.
- Long term stability of the widened road embankment – This is to assess the long term stability of the road embankment, typically with the assumption of some remaining excess porewater pressure within the 10 year design life.

The results of the slope stability analyses are summarized in Table 8.1 while the graphical presentation of the results is provided in Appendix E.

Table 8.1 - Summary of Slope Stability Analysis Results

Station (m)	Description of Analysis	Reference	Calculated Factor of Safety (FOS)	Remarks ("*Feasible*" is the preferred option.)
18+730 (Based on BH 1, 2 & 3)	***Undrained Analysis***			
	Existing Condition	Figure E1.1	1.09	- Shear strengths used are reasonable. - FOS of the existing road embankment may be higher due to consolidation.

Station (m)	Description of Analysis	Reference	Calculated Factor of Safety (FOS)	Remarks ("*Feasible*" is the preferred option.)
	3 m widening – End of Construction	Figure E1.2a	1.14	FOS slightly increased due to the flatter slope for the road widening (approximately 3.4H:1V) (Deep-seated critical slip surface, instead of shallow slip surface, is considered.) ***Feasible***
	3 m widening – End of Construction	Figure E1.2b	2.02	FOS > 1.5 (Shallow slip surface within the road widening portion is considered.)
	Drained Analysis			
	Existing Condition – Long Term Analysis	Figure E1.3	1.41	- Effective strengths used are reasonable. - FOS of the existing road embankment may be higher due to consolidation.
	3 m widening - Long Term Analysis	Figure E1.4	1.64	Stable
18+875 (Based on BH 4)	***Undrained Analysis***			
	Existing Condition	Figure E2.1	1.54	Stable
	3.5 m widening – End of Construction	Figure E2.2	1.32	FOS is decreased due to the additional embankment load for the road widening

Station (m)	Description of Analysis	Reference	Calculated Factor of Safety (FOS)	Remarks ("*Feasible*" is the preferred option.)
	3.5 m widening with 5 m extension of existing rock fill berm – End of Construction	Figure E2.3a	1.54	FOS is similar to existing case. (Deep-seated critical slip surface, instead of shallow slip surface, is considered.) ***Feasible***
	3.5 m widening with 5 m extension of existing rock fill berm – End of Construction	Figure E2.3b	1.66	FOS > 1.5 (Shallow slip surface within the road widening portion is considered.)
	Drained Analysis			
	Existing Condition – Long Term Analysis	Figure E2.4	2.30	Stable
	3.5 m widening with 5 m extension of existing rock fill berm – Long Term Analysis	Figure E2.5	2.05	Stable
18+975 Based on BH 6 & 7)	***Undrained Analysis***			
	Existing Condition	Figure E3.1	1.04	- Existing embankment has not collapsed. - FOS of the existing road embankment may be higher due to consolidation.
	2.5 m widening – End of Construction	Figure E3.2	1.04	-FOS is almost same as the existing condition due to similar slopes for the existing and widened embankments.

Station (m)	Description of Analysis	Reference	Calculated Factor of Safety (FOS)	Remarks ("*Feasible*" is the preferred option.)
	2.5 m widening with toe berm (4 m width & 1.5 m high) – End of Construction	Figure E3.3a	1.09	-FOS is slightly increased. -Deep-seated critical slip surface, instead of shallow slip surface, is considered. **Feasible**
	2.5 m widening with toe berm (4 m width & 1.5 m high) – End of Construction	Figure E3.3b	1.75	FOS > 1.5 (Shallow slip surface within the road widening portion is considered.)
	Drained Analysis			
	Existing Condition – Long Term Analysis	Figure E3.4	1.59	Stable
	2.5 m widening with toe berm (4 m width & 1.5 m high) - Long Term Analysis	Figure E3.5	1.52	Stable
19+100 (Based on BH 9)	***Undrained Analysis***			
	Existing Condition	Figure E4.1	1.08	- Existing embankment has not collapsed. - FOS of the existing road embankment may be higher due to consolidation.

Station (m)	Description of Analysis	Reference	Calculated Factor of Safety (FOS)	Remarks ("*Feasible*" is the preferred option.)
	3 m widening – End of Construction	Figure E4.2a	1.10	FOS increased slightly due to the flatter slope for the road widening (approximately 3.4H:1V) (Deep-seated critical slip surface, instead of shallow slip surface, is considered.) **Feasible**
	3 m widening – End of Construction	Figure E4.2b	3.17	FOS ≥ 1.5 (Shallow slip surface within the road widening portion is considered.)
	Drained Analysis			
	Existing Condition – Long Term Analysis	Figure E4.3	1.52	Stable
	3 m widening– Long Term Analysis	Figure E4.4	1.31	Stable

From the results of the slope stability analyses as summarized in Table 8.1, the preferred design of the widened road embankment is to construct the widened road embankment with a flattened side slope (i.e., flatter than the typical 2H:1V) or a 2H:1V slope with toe berm for stabilization, except at the road sections where the toe berm should not be needed for slope stability. The recommended embankment configurations are outlined in Table 8.2.

It should be noted that the final recommended embankment configuration and the requirement of toe berm are summarized in the Addendum after considering the calculated settlements.

Table 8.2 - Recommended Embankment Configuration

Station (m)	Height of Existing Road Embankment (m)	Proposed Road Widening Width (m)	Thickness of Soft Compressible Soils[1] (m)	Thickness of Peat (to be removed) (m)	Recommended Embankment Configuration[2]
18+700 to 18+775	2.0 to 5.0	2.5 to 4.0	6.0 to 12.0	2.1 to 3.7	The slope of the widened road embankment should be 2H:1V.

Station (m)	Height of Existing Road Embankment (m)	Proposed Road Widening Width (m)	Thickness of Soft Compressible Soils[1] (m)	Thickness of Peat (to be removed) (m)	Recommended Embankment Configuration[2]
18+775 to 18+825	1.0 to 2.2	3.0 to 3.5	No boreholes	No boreholes	(cut section).
18+825 to 18+950	3.5 to 3.0	2.5 to 3.5	4.5 to 7.0	-	Extend the existing rock fill berm by 2 to 5 m. The slope of the widenedroad embankment should be 2H:1V .
18+950 to 19+050	3.5 to 4.0	2.5 to 3.0	7.5 to 11.5	0.6 to 0.9	Construct a toe berm with 4 m width and 1.5 m high. The slope of the widened road embankment should be 2H:1V.
19+050 to 19+200	2.6 to 4.0	2.5 to 3.5	7.0 to 12.0	1.4 to 1.8	The slope of the widened road embankment should be 2H:1V.

Notes:
(1) Soft compressible soils including existing peat layer
(2) All organic materials underneath the footprint of the widening embankment should be removed and backfilled with rock fill or approved granular soils according to OPSD 203.030. For above water construction, a geotextile (Terrafix 270R) or equivalent should be placed between the inorganic subgrade and rock fill. The 2H:1V slope should be constructed with rock fill or compacted engineered fill. Swamp fill may be added onto the surface of the 2H:1V slope according to OPSD 203.030, if required.

8.4 Settlement Calculations

8.4.1 Method of Settlement Calculations

Settlement calculation procedures and assumptions are included in Appendix F. It has been assumed that no settlement of the embankment under the existing conditions is occurring, since the existing road embankment has already been in service for 20 to 30 years. Furthermore, it has been assumed in the settlement analysis that all compressible peat material within the footprint of the widened embankment has been removed as per OPSD 203.030.

8.4.2 Results of Calculated Settlements

All settlement calculations are based on the following referenced stations and representative borehole information:

Station	Borehole No.	Widened Road Embankment Height (m)	Analyzed Thickness of Clay Layer (m)
18+730	BH 1, 2 & 3	5.0	9.5
18+875	BH 4	2.5	4.5
18+975	BH 6 & 7	3.5	10.0
19+100	BH 9	3.0	10.0

The details of calculation procedures are provided in Appendix F.

8.4.2.1 Immediate Settlement under Widened Road Embankment

The results of immediate (elastic) settlement calculations for construction considerations are summarized in Table 8.3. The immediate settlements occur during construction and as such, the immediate settlements are not considered to be part of the long-term settlements of the widened road embankment.

Table 8.3 – Calculated Volume Loss during Construction

Section (From Station to Station)	Length of Section (m)	Range of Calculated Immediate Settlement (mm)	Calculated Volume Loss (m^3/m)	Calculated Volume Loss over Section (m^3)
18+700 - (18+730)* - 18+775	75	190	92	6,900
18+775 - 18+825	50	-	cut section	-
18+825 - (18+875)* - 18+925	100	30	18	1,800
18+925 - (18+975)* - 19+040	115	90	30	3,450
19+040 - (19+100)* - 19+200	160	90	15	2,400
		Total Volume Loss (m^3)		**14,550**

Note: * Referenced Station where the immediate settlement is calculated. The calculated volume loss from one station to another station (located both sides of the Referenced Station) is based on the weighted average of the height of the new road embankment.

It should be noted that the calculated volume loss could be significantly different from the actual volume loss measured during construction. This is due to the facts that the calculated volume loss is based on theoretical considerations for vertical settlement and lateral movement of the soft soils

supporting the widened road embankment is not included in the calculated volume loss. **For contractual purposes, the quantity for volume loss during construction should be considerably higher than the calculated volume loss.** For construction contract, it is suggested that the contract allows for a volume loss of up to 20,000 m^3 subject to actual volume measurement and unit rate payment.

The immediate settlement contour plots as generated by SIGMA/W for the analyzed sections are presented on Figure Nos. F1.1 to F1.4 for the as-planned loading scheme and Figure Nos. F2.1 to F2.4 for the additional surcharge loading scheme of Appendix F.

8.4.2.2 Primary Consolidation Settlement

Based on the two consolidation test results (Figure Nos. B8 and B9 in Appendix B), the variation of overburden pressure (p_o), pre-consolidation pressure (p_c) and overconsolidation ratio (OCR), with depth, are presented in Figure No. 2. From the two consolidation test results, a representative variation of p_c and OCR with depth for the site can not be established. The high OCR values may reflect the high content of silt in the test samples.

Settlements due to consolidation are calculated considering the following four cases:

CASE I — Normally consolidated condition with C_c = 0.18 as determined by the consolidation testing on Sample TW7 (Borehole BH 1).

CASE II — Normally consolidated condition with C_c = 0.29 as determined by the consolidation testing on Sample TW5 (Borehole BH 6).

CASE III — Over-consolidated condition with a preconsolidation pressure (p_c) of 80 kPa and C_c = 0.18 as determined by the consolidation testing on Sample TW7 (Borehole BH 1).

CASE IV — Over-consolidated condition with a preconsolidation pressure (p_c) of 160 kPa and C_c = 0.29 as determined by the consolidation testing on Sample TW5 (Borehole BH 6).

In general, Cases III and IV should be representative of the calculated settlements since they are based on actual laboratory test results. Cases I and II should be considered as conservative, leading to relatively high consolidation settlements. It should be noted that the normally-consolidated condition considered in Cases I and II is based on the soil condition investigated which indicates that the natural water content of the soft clay is higher than its liquid limit.

As derived from Figure Nos. F3.1 to F3.4 of Appendix F, the embankment widening results in final effective overburden pressures (p_f) ranging from 53 kPa to 162 kPa at the middle of the clayey silt/silty clay layer.

The results of the total, transverse and longitudinal calculated settlements at the end of a period of ten (10) years after construction for Cases I to IV are presented in Tables F1 to F4 of Appendix F and summarized in Tables 8.4a, 8.4b and 8.4c respectively.

Based on the calculations, the degree of primary consolidation is typically at or greater than 90% before the end of ten (10) years after construction and, accordingly, secondary (creep) consolidation values are included in the final settlement estimates. The total settlements of the widened embankment during the design life of 10 years are therefore due to both primary and secondary consolidation.

Table 8.4a – Total Settlements: 10 years after Construction (without Preloading)

Station (m)	Range of Calculated Total Settlements at Points along Embankment Sections (mm)[1]					Remarks (Criteria - Total <100 mm at Points A, B and C)
	A min - max	B min - max	C min - max	D min - max	E min - max	
18+730	**20 - 60**	**20 - 110**	**35 - 200**	*55 - 330*	*60 - 370*	***min values OK max values fail at B & C***
18+875	**5 - 10**	**10 - 25**	**10 - 40**	*10 - 55*	*15 - 70*	***OK***
18+975	**10 - 25**	**20 - 65**	**25 - 110**	*40 - 230*	*45 - 270*	***min values OK max value fails at C***
19+100	**10 - 20**	**20 - 85**	**30 - 140**	*40 - 210*	*35 - 190*	***min values OK max value fails at C***

Notes: 1) Point A – Centerline of Existing Embankment, Point B – Crest of Existing Embankment adjacent to Widened Embankment, Point C – Crest of Widened Embankment, Point D – Mid-Slope of Widened Embankment, Point E – Toe of Widened Embankment (See Figure No. F3.0 in Appendix F and as follows).

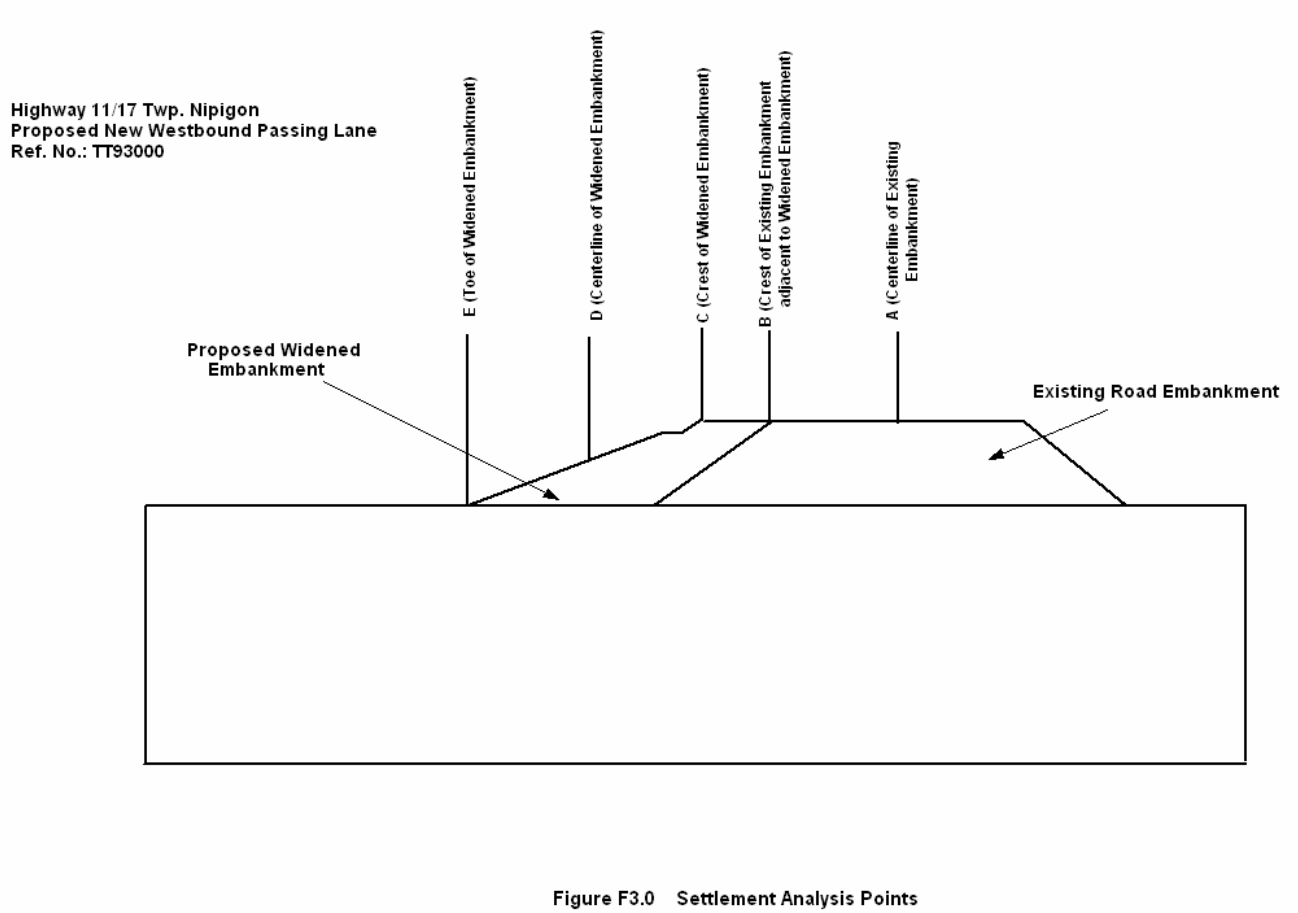


Figure F3.0 Settlement Analysis Points

Table 8.4b – Transverse Differential Settlements: 10 years after Construction (without Preloading)

Station (m)	Range of Differential Settlements between Points A and C (mm) min - max	Distance between Points A and C (m)	Transverse Grade Change (%) min - max	Remarks (Criteria - Differential <50 mm; Grade Change <0.5%)
18+730	15 - 140	9	0.2 – 1.6	***min values OK*** ***max values fail both criteria***
18+875	5 - 35	13	0 – 0.3	***OK***
18+975	10 - 85	11	0.1 – 0.8	***min values OK*** ***max values fail both criteria***
19+100	15 - 120	11	0.1 – 1.1	***min values OK*** ***max values fail both criteria***

Table 8.4c – Longitudinal Differential Settlements: 10 years after Construction (without Preloading)

Station (m)	Range at Top of Widened Road Embankment (mm) min - max	Distance between Stations (m)	Grade Change (%) min - max	Remarks (Criteria - Grade Change <0.5%)
18+730 to 18+875	15 - 160	145	0.01 – 0.11	***OK***
18+875 to 18+975	5 - 70	100	0.01 – 0.07	***OK***
18+975 to 19+100	0 - 35	125	0 – 0.02	***OK***

Based on these results, further analysis considering pre-loading periods of three (3) months and one (1) year are completed for the total and transverse differential settlements using the as-planned loading to determine if the remaining settlements after the three (3) month and one (1) year pre-loading periods could satisfy the settlement criteria.

The three month period is the minimum period required by the TOR and the one year period is based on MTO's plan that the new road embankment will be completed to the top of subbase or subgrade in October 2009 and the base and asphalt surface will be completed in 2010.

The results of the calculated settlements are included in Tables F1 to F4 of Appendix F with total and transverse settlements summarized in Tables 8.5a and 8.5b (3 months) and Tables 8.6a and 8.6b (1 year). Preloading in these tables refers to the completion of the widened road embankment to the subbase level (as-planned load) after which it is left in place for preloading of the clay layer.

Table 8.5a – Remaining Total Settlements after 3 months Pre-loading (As-Planned Load)

Station (m)	Range of Calculated Total Settlements at Points along Embankment Sections (mm)					Remarks (Criteria - Total <100 mm at Points A, B and C)
	A min - max	B min - max	C min - max	D min - max	E min - max	
18+730	**20 – 50**	**20 - 90**	**30 - 170**	*45 - 275*	*50 - 305*	***min values OK max value fails at C***
18+875	**5 - 10**	**10 - 20**	**10 - 30**	*10 - 40*	*10 - 50*	***OK***
18+975	**10 - 20**	**20 - 60**	**20 - 90**	*35 - 190*	*40 - 230*	***OK***
19+100	**10 - 20**	**20 - 70**	**25 - 115**	*35 - 175*	*30 - 160*	***min values OK max value fails at C***

Table 8.5b – Remaining Transverse Settlements after 3 months Pre-loading (As-Planned Load)

Station (m)	Range of Differential Settlements between Points A and C (mm) min - max	Distance between Points A and C (m)	Transverse Grade Change (%) min - max	Remarks (Criteria - Differential <50 mm; Grade Change <0.5%)
18+730	10 - 120	9	0.1 – 1.3	***min values OK max value fails both criteria***
18+875	5 - 20	13	0 – 0.2	***OK***
18+975	10 - 70	11	0.1 – 0.6	***min values OK max values fail both criteria***
19+100	15 - 100	11	0.1 – 0.9	***min values OK max values fail both criteria***

Table 8.6a – Remaining Total Settlements after 1 year Pre-loading (As-Planned Load)

Station (m)	Range of Calculated Total Settlements at Points along Embankment Sections (mm)[1]					Remarks (Criteria - Total <100 mm at Points A, B and C)
	A min - max	B min - max	C min - max	D min - max	E min - max	
18+730	**20 - 40**	**20 - 75**	**30 - 135**	*40 - 220*	*40 - 250*	***min values OK max value fails at C***
18+875	**5 - 10**	**5 - 10**	**10 - 20**	*10 - 20*	*10 - 25*	***OK***
18+975	**10 - 20**	**15 - 50**	**20 - 80**	*30 - 155*	*35 - 185*	***OK***
19+100	**10 - 20**	**20 - 60**	**25 - 95**	*30 - 145*	*30 - 130*	***OK***

Table 8.6b – Remaining Transverse Settlements after 1 year Pre-loading (As-Planned Load)

Station (m)	Range of Differential Settlements between Points A and C (mm) min - max	Distance between Points A and C (m)	Transverse Grade Change (%) min - max	Remarks (Criteria - Differential <50 mm; Grade Change <0.5%)
18+730	10 - 95	9	0.1 – 1.0	***min values OK*** ***max values fail both criteria***
18+875	5 - 10	13	0 – 0.1	***OK***
18+975	10 - 60	11	0.1 – 0.5	***min values OK*** ***max value fails differential***
19+100	10 - 80	11	0.1 – 0.7	***min values OK*** ***max values fail both criteria***

Based on these results, further analyses considering an additional 1 m high pre-load surcharge above the as-planned load over periods of three (3) months and nine (9) months beyond the initial as-planned three (3) month pre-loading period, corresponding to six (6) month and one (1) year after initial completion of the widened road embankment, are completed for the total and transverse differential settlements to determine if the remaining settlements satisfy the settlement criteria.

As derived from Figure Nos. F4.1 to F4.4 of Appendix F, the additional 1 m surcharge height results in final effective overburden pressures (p_f) ranging from 56 kPa to 163 kPa at the middle of the very soft to soft clayey silt/silty clay layer.

The results of the total and transverse estimated settlement calculations with the additional surcharge pre-loading are included in Tables F5 to F8 of Appendix F and summarized in Tables 8.7a and 8.7b (6 months after initial construction) and Tables 8.8a and 8.8b (1 year after initial construction).

Table 8.7a – Remaining Total Settlements after 6 months Pre-loading (i.e., 3 months of As-Planned Load plus another 3 months with 1 m thick Surcharge Load)

Station (m)	Range of Calculated Total Settlements at Points along Embankment Sections (mm)[1]					Remarks (Criteria - Total <100 mm at Points A, B and C)
	A min - max	B min - max	C min - max	D min - max	E min - max	
18+730	**20 - 45**	**30 - 90**	**25 - 115**	*40 - 230*	*45 - 250*	***min values OK*** ***max value fails at C***
18+875	**5 - 10**	**5 - 20**	**10 - 20**	*10 - 25*	*10 - 30*	***OK***
18+975	**10 - 25**	**20 - 60**	**20 - 92**	*35 - 180*	*40 - 205*	***OK***
19+100	**10 - 25**	**20 - 80**	**25 - 120**	*35 - 170*	*30 - 155*	***min values OK*** ***max value fails at C***

Table 8.7b – Remaining Transverse Settlements after 6 months Pre-loading (i.e., 3 months of As-Planned Load plus another 3 months with 1 m thick Surcharge Load)

Station (m)	Range of Differential Settlements between Points A and C (mm) min - max	Distance between Points A and C (m)	Transverse Grade Change (%) min - max	Remarks (Criteria - Differential <50 mm; Grade Change <0.5%)
18+730	5 - 70	9	0 – 0.8	***min values OK max values fail both criteria***
18+875	5 - 15	13	0 – 0.1	***OK***
18+975	10 - 70	11	0.1 – 0.6	***min values OK max values fail both criteria***
19+100	15 - 95	11	0.1 – 0.9	***min values OK max values fail both criteria***

Table 8.8a – Remaining Total Settlements after 1-year Pre-loading (i.e., 3 months of As-Planned Load plus another 9 months with 1 m thick Surcharge Load)

Station (m)	Range of Calculated Total Settlements at Points along Embankment Sections (mm)[1]					Remarks (Criteria - Total <100 mm at Points A, B and C)
	A min - max	B min - max	C min - max	D min - max	E min - max	
18+730	**20 - 40**	**25 - 80**	**20 - 95**	*35 - 190*	*40 - 205*	***OK***
18+875	**5 - 10**	**5 - 10**	**5 - 10**	*5 - 10*	*5 - 15*	***OK***
18+975	**10 - 20**	**15 - 55**	**20 - 80**	*30 - 150*	*35 - 170*	***OK***
19+100	**10 - 20**	**20 - 70**	**25 - 100**	*30 - 145*	*30 - 130*	***OK***

Table 8.8b – Remaining Transverse Settlements after 1-year Pre-loading (i.e., 3 months of As-Planned Load plus another 9 months with 1 m thick Surcharge Load)

Station (m)	Range of Differential Settlements between Points A and C (mm) min - max	Distance between Points A and C (m)	Transverse Grade Change (%) min - max	Remarks (Criteria - Differential <50 mm; Grade Change <0.5%)
18+730	5 - 60	9	0 – 0.6	***min values OK max values fail both criteria***
18+875	0 - 5	13	0	***OK***
18+975	10 - 60	11	0.1 – 0.5	***min values OK max value fails differential***
19+100	10 - 80	11	0.1 – 0.7	***min values OK max values fail both criteria***

Settlement versus time plots for the critical point (Point C – Crest of Widened Embankment) at each referenced station considering the four analyzed cases (Cases I to IV) are presented on Figure Nos. F5.1 to F5.4 of Appendix F.

8.4.3 Conclusions for Calculated Settlements

Based on the calculations of immediate and primary consolidation settlements in Sections 8.4.2.1 and 8.4.2.2, the following conclusions should be considered by the designer:

- From the calculation of immediate settlements during construction, the calculated volume loss of the fill materials is 14,450 m^3. Due to the nature of the calculation, the volume loss to be considered in the construction contract should be substantially higher than the calculated volume loss. Suggested volume loss for the construction contract is provided in Section 8.4.2.1.
- For the design life of 10 years, it is likely that the widened road embankment will have reached a degree of primary consolidation of at least 90% (±) at or before the end of the 10 year design life. As such, secondary settlement (creep) has been considered in the settlement calculation.
- Based on the representative laboratory consolidation test results which yield the minimum calculated settlements, the total settlement of the widened road embankment and the differential settlement between the widened and the existing road embankments should meet the criteria required by MTO. However, due to the theoretical nature of settlement calculations, the actual settlements of the widened road embankment may be significantly different from the minimum calculated settlements.
- It is recommended that the widened road embankment be constructed as planned by MTO, i.e., to the subbase level. Subsequently, the widened road embankment should be monitored for its settlement and excess porewater pressure dissipation for a period of 1 to 3 months. If the measured settlement and/or the dissipation of the excess porewater pressure are equal to or less than the calculated minimum settlements and/or the degree of consolidation, there will be no need for additional surcharge. On the contrary, an additional 1 m thick surcharge can be placed on top of the widened road embankment to accelerate the consolidation settlement. The recommended widened road embankment configuration and monitoring program are provided in the Addendum, respectively.

8.5 Comparison of Ground Improvement Methods

Based on the facts that the widened road embankment could be constructed to the design height with a toe berm for slope stabilization (where required) and there should be sufficient time period for preloading to reduce future settlement (from approximately 31 October 2009 to spring of 2010 – minimum of about 6 months available), the preferred ground improvement method for the widened road embankment is preloading with the majority of the widened road embankment weight itself.

Preloading is defined as applying the design load onto the soft subgrade soil for a certain period in order to reduce post-construction settlement after the construction of the new lane has been completed.

The main purpose of the ground improvement for the widened road embankment is to reduce post-construction settlements and enhance the slope stability. As such, there are a number of feasible ground improvement methods as compared in Table 8.9.

Table 8.9 – Comparison of Feasible Ground Improvement Methods

Ground Improvement Method	Description	Advantages	Disadvantages	Risks / Consequences	Cost Comparison
OPTION 1: Preloading without surcharge in one stage	Construct the widened road embankment to the design height (top of subbase) as planned by MTO and allow it to settle with time (minimum 3 months).	- Construction in one stage prior to placing base and hot mix.	- Preloading embankment height is slightly lower than the load from the base and traffic. - Additional load from base and traffic may cause post-construction settlement (to be closer to the tolerable settlement of 50 mm).	- Additional settlement due to placing base and traffic load. - Potential cracks in new paved road surface. - Potential differential settlement between the widened and existing road embankment.	- Low
OPTION 2: Preloading with surcharge in one stage	Construct the widened road embankment to base and place additional surcharge of 1 m to 2 m high.	- Reduce future settlement of the new paved lane.	- Additional fill is required for surcharge. - Removal of the additional fill after preloading. - Additional settlement to the existing road embankment.	- Reduce stability of the widened road embankment. Additional berm height and width may be required.	- Medium to high
OPTION 3: Preloading in stages	Construct the widened road embankment to subbase. Place additional fill on top of the subbase if the settlement rate is too slow and/or higher settlement is required.	- Reduce future settlement of the new paved lane.	- Additional placement of fill is required in stages. - Increase in construction cost.	- If fill higher than the finished grade is required, embankment stability will be reduced.	- Medium to high
OPTION 4: Preloading with prefabricated	Construct the toe berm with granular soils, install PVD (wick drains),	- Lower the moisture content of the clayey soil throughout the depth of PVD	- A working platform is required to support the PVD equipment during installation. - A sand blanket layer	- High surcharge fill may be required, leading to risk of slope instability.	- High

Ground Improvement Method	Description	Advantages	Disadvantages	Risks / Consequences	Cost Comparison
vertical drains (wick drains)	and place the widened road embankment to a sufficient height. Allow sufficient preloading time to dissipate the excess porewater pressures from the PVD (wick drains).	(wick drains), thereby possibly increasing the soil shear strength. - More effective in reducing settlement and preloading time than preloading without PVD (wick drains). - Reduce future settlement of the widened road embankment.	is required to allow drainage of water from PVD. - A surcharge load higher than the preconsolidation pressure is required, probably resulting in relatively high preloading fill embankment.	- Poor design and installation of the PVD system may result in poor dissipation of excess porewater pressure.	
OPTION 5: Lightweight fill	Construct the widened road embankment with lightweight fill, e.g., expanded polystyrene (EPS).	- Low additional load on the existing road embankment and ground. - Toe berm for stabilizing the road embankment may not be required where recommended. - Low road settlement.	- Long term integrity of lightweight fill is uncertain. For example, EPS may be damaged by oil spill, rodent activity, etc.	- Lightweight fill may be damaged in long term, resulting in high settlement or collapse of the new road embankment.	- High to medium
OPTION 6: Geosynthetic reinforcement / Retained Soil System (RSS)	Construct the widened road embankment with geosynthetic (e.g., geogrid, geotextile, etc.) at the base and/or within the embankment.	- Reduce side slope of the widened road embankment. - Possibly more uniform settlement. - Possibly preventing cracks within the widened road embankment.	- Unnecessarily increase cost and construction time in the event that typical 2H:1V side slope can be constructed within right-of-way. - The widened road embankment built with geosynthetic will still settle, possibly the same amount as the road embankment without geosynthetic.	- Steep slope of road embankment built with geosynthetic may cause abrupt settlement, leading to separation between the widened and the existing road embankment.	- High to medium
OPTION 7: Stone columns (e.g., Geopier)	Install columns of stone/gravel into the soil subgrade by displacement or non-displacement	- High compressive strength at the locations of stone columns. - Stone columns will possibly act as drainage path	- Displacement method for installing stone columns will significantly reduce the shear strength of the soft clay, thereby lowering the shear strength of the soft	- Disturb the native soft clay (particularly by the displacement method) and significantly reduce its shear strength.	- Medium to high

Ground Improvement Method	Description	Advantages	Disadvantages	Risks / Consequences	Cost Comparison
	methods, while compacting the stone/gravel.	for dissipating excess porewater pressure (similar to PVD – wick drains).	clay between the stone columns. - A large number of closely-spaced stone columns will be required to significantly increase the shear strength of the soft clay in order to enhance the slope stability of the road embankment. - Installing stone columns to a sufficient depth in a thick soft clay layer may be difficult or may not be achievable (e.g., Geopier length is normally less than 8m). - A hard layer of soil or equivalent may be required on top of the stone columns to distribute the applied load onto the soil subgrade.	- Installing too few stone columns may actually lower the overall shear strength of the soft clay layer, thereby leading to lowering its factor of safety against slope instability. - Significant difference in compressive strength at the locations of stone columns and the soft clay between the stone columns may lead to significant differential settlement in the road surface (mushroom-like settlement).	

8.6 Proposed Preloading Scheme

Based on the results of slope stability analyses (Table 8.1) and settlement calculations (Tables 8.4 to 8.8), preloading by the widened road embankment, i..e, Option 1 - Preloading without surcharge in one stage (Table 8.9), is the preferred ground improvement method. The recommendations for the design of the widened road embankment configurations together with associated analyses are summarized in the Addendum. All organic matters (e.g., peat) underneath the widened road embankment should be removed according to OPSD 203.030. The following procedures for preloading should be considered in the design and construction.

a. Construct the widened road embankment and the toe berm (Figure No. 3a), where recommended (Addendum), according to the procedures in Sections 8.7 to 8.11. **The bottom layer of the widened road embankment and the toe berm, where recommended, should consist of a minimum 1 m thick free-draining granular layer in order to facilitate draining of porewater from the subsurface soils during preloading. The free-draining layer should be designed and constructed such that it provides positive drainage for any free water entering underneath the widened road embankment and toe berm (where recommended) from the subgrade, and at the**

same time, prevents any water from the surroundings (e.g, creek/swamp water) to enter the toe berm (where recommended) and the widened road embankment. A side ditch located below the edge of the widened road embankment and the toe berm (where recommended) may be required.

b. Construct the widened road embankment and the toe berm (where recommended) to the design height, in one stage.
c. Construct the widened road embankment to the design height, i.e., top of subbase.
d. Monitor the porewater pressures in the subsurface soil underneath the widened road embankment through a series of standpipe piezometers, together with settlement points as recommended in the Addendum. If the installation of standpipe piezometers is not practical, only settlement points should be sufficient for monitoring the settlement of the widened road embankment.
e. Evaluate the rate of porewater pressure dissipation (if monitored) and consolidation settlement and compare the monitoring results with the design requirements as shown in the Addendum. If the settlements of the preloading widened road embankment are less than those required, increasing road embankment height and thickening side slopes as shown in Figure No. 3b may be required prior to completing the preloading period.

It should be noted that the calculated settlements and the calculated rates of settlement are based on the one-dimensional consolidation theory and laboratory test results. The actual field settlements and the actual field rates of settlement may be different than those calculated. As a result, it may be necessary to revise the design and construction schedule accordingly in order to satisfy the post-construction settlement criteria.

8.7 Construction of Widened Road Embankment

The construction of the widened road embankment should follow OPSS 209 (Construction Specification for Embankment over Swamps and Compressible Soils). In particular the embankment construction should follow the “Excavation Method” in OPSS 209.

All organic matters should be removed from the footprint of the widened road embankment, as per Ministry of Transportation of Ontario’s current practice, with an envelope given by a gradient not steeper than 1H:1V away from the toe of the embankment. According to MTO’s planned cross sections, the existing slopes will be maintained. In this case, OPSD 203.030 (Embankment Over Swamp – Existing Slopes Maintained) should be used (Appendix G). In order to use the excavated swamp material as slope flattening material as specified in OPSD 203.030, the material should be dried and sloped at no steeper than 3H:1V. Otherwise OPSD 203.02 and/or OPSD 203.010 should be considered. Under the toe berm (where recommended), the organic matters may be left in place provided that the toe berm is constructed to the elevation and extent as recommended. The toe berm (where recommended) should consist of rock fll.

For above water construction, the exposed subgrade should be inspected, approved and properly compacted from the surface, to a minimum of 95 % Standard Proctor Maximum Dry Density (SPMDD), under the supervision of qualified geotechnical personnel. Any soft spots identified during stripping and/or re-compacting should be sub-excavated and replaced with compacted engineered fill. Care should be exercised to minimize disturbance to the subgrade during preparation and the construction of embankment.

For under water construction (e.g., in the area of culverts/swamps), all organic matters and/or soft/loose soils should be removed and backfilled with rock fill or approved granular soils until the backfill level is above the water level. Otherwise, dewatering is required for placing an engineered fill.

In the case of soft native clay subgrade, the subgrade preparation should be treated similar to the "under water construction" procedures. In addition, after removing all organic matters, a geotextile (Terrafix 270R or equivalent) should be placed on top of the soft clay subgrade, where practical, prior to placing rock fill or approved granular fill.

The widened road embankment should be constructed with rock fill or compacted engineered fill at 2H:1V side slopes.

For widening the existing road embankment, the new fill should be constructed with benching according to OPSD – 208.010 (Benching of Earth Slopes). The additional slope to the existing slope should be properly benched into the existing slope and the new fill should be properly compacted/constructed.

For an engineered fill, the fill soils used for construction of the conventional earth fill embankment, or for the purposes of backfilling, should consist of approved, clean earth fill (e.g. Select Subgrade Materials – Ontario Provincial Standards Specifications Number: 1010). The existing very soft to soft clayey soils are not suitable for reuse unless their high water contents have been reduced to be close to their optimum water contents. The fill soils should be placed in accordance with Ontario Provincial Standards Specifications Number: 501. Each lift should not exceed 300 mm before compaction and each lift should be uniformly compacted to at least 95 % of the Standard Proctor Maximum Dry Density (SPMDD) of the materials. The degree of compaction within the top 0.6 m of the fill (i.e., the subgrade immediately beneath the granular sub-base) should be increased to 98 % SPMDD. The selection, placement and compaction of the fill should be carried out under a geotechnical control program.

For the fill embankment at this site, using properly compacted and acceptable inorganic fill soils, the side slopes should be 2H:1V for earth fill embankment. Proper erosion control measures should be implemented both during construction and on a permanent basis. This can be achieved by immediate seeding or sodding (Ontario Provincial Standards Specification Number: 572) or equivalent.

8.8 Culvert Replacement

There are two existing culverts located within the project site (Drawing No. 2) that have been planned to be replaced. At the time of investigation, the existing culvert located at approximately Station 18+730 was a 1070 mm in diameter by 29.2 m long CSP and was investigated by Boreholes BH 1 to BH 3. The location of the replaced culvert may be slightly different from the existing culvert at this location. Another existing culvert was located at approximately Station 19+100 and composed of two 1220 mm in diameter by 26.6 m long CSP. This location was investigated by Boreholes BH 9 and BH 10.

The new replacement culverts will have to be extended beyond the widened road embankment. As such, the new culverts underneath the widened road embankment will likely settle. It is therefore preferable to place the new culverts after completion of preloading and at the same time of replacing the existing culverts. Temporary culverts may be used under the widened road embankment during preloading.

The widened road embankment at the extended culverts should be according to OPSD 203.040.

The subgrade for culvert bedding should be cleared of all organic matters and deleterious materials. The exposed subgrade may consist of sandy/silty soils and/or very soft to soft clayey silt/silty clay. Lean concrete (minimum 100 mm thick) or 25 mm minus clear stone overlying a geotextile (Terrafix 270R or equivalent) or equivalent granular soils, should be placed as culvert bedding. At each inlet and outlet of the culverts, a shallow cutoff wall should be embedded in the native soil underneath the culvert in order to prevent the potential loss of the culvert bedding and/or migration of soil subgrade particles. The construction of the cut off wall should be according to OPSD 812.010. Rip-rap treatment (OPSD 804.010) or concrete headwall (OPSD 804.040) may be considered for the culvert outlets.

The inlets and outlets of the culverts should be designed and constructed to prevent erosion from the water flow.

8.9 Construction Staging and Detour

The construction of the widened road embankment should not interrupt the traffic on the existing highway except at the entrance and exit to the area of the widened road embankment. However, the replacement of the two existing culverts will likely require the closure of one lane of the highway at a time, unless a detour is possible.

If a detour is not possible/practical, one lane of the existing two lane highway will have to be closed while keeping the remaining lane open for traffic. This will reduce the traffic lane to be a single lane and all necessary traffic protection will have to be provided. A roadway protection system (i.e., temporary shoring system) will be required,

Replacement of two existing culverts within the project area at stations 18+731 and 19+102 is planned under this project. Based on the cross-sections provided by MTO at locations of the existing culverts, the new culverts will have to be extended to the north at both locations. In addition, the culvert at station 18+731 will have to be extended to the south as well. The subgrade for culvert bedding should be cleared of all organic matters and deleterious materials.

Three boreholes were drilled in the vicinity of the existing culverts at station 18+731, two at the north side (BH 1 and BH 3) and one at the south side (BH 2) of the road. Two boreholes were also drilled in the vicinity of the existing culvert at station 19+102, one at the north side (BH 9) and one at the south side (BH 10) of the road. Based on the borehole information of the current investigation (Appendix A) and the boreholes information provided by MTO (Appendix D), approximate depth of peat/organic in the vicinity of the existing culver at station 18+731 may be up to 3.9 m at the north side and up to 2.1 m at the south side of the road. Similarly, the approximate depth of peat/organic in the vicinity of the existing culver at station 19+102 may be up to 1.4 m at the north side and up to 1.8 m at the south side of the road.

Temporary cantilever/braced sheet piles or soldier piles with wood laggings may be used to support the partially-removed road embankment. The temporary shoring system should be designed using the following soil parameters and surcharge:

Soil bulk unit weight	=	20 kN/m^3
Coefficient of lateral soil earth pressure	=	0.4
Friction Angle for road fill	=	34^o
Traffic load as surcharge	=	20 kPa

It should be noted that there may be rock fill with cobble/boulder sized rock underneath the existing and/or widened embankments. The rock fill, if it exists, may obstruct driving of sheet piles/piles or auguring for caissons. The possible presence of rock fill should be considered in the design of roadway protection system.

Groundwater levels shown in the Record of Boreholes / DCPT Test Holes (Appendix A) should be considered in the design of the temporary shoring system.

Due to the potential settlement of the new road embankment, the replacement of the existing culverts should be carried out after the completion of preloading, i.e., at the same time of placing the granular base on the new road embankment and laying new hot mix on the highway and the new lane. At that time, the new culvert vertical alignment should not be significantly affected by post construction settlement.

8.10 General Construction Comments

All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and its regulations (i.e. Occupational Health and Safety Act O.Reg. 213/91).

The boreholes show that the excavation for the widened road embankment should encounter organic matters (peat), fill soils and very soft to soft clayey soils. These soils should be classified as Type 4 and according to the Occupational Health and Safety Act, temporary open cut slopes should not be steeper than 3H:1V.

In order to prevent slope instability of the existing road embankment, excavation along the toe of the existing embankment should follow OPSS 209, particularly Section 209.07.03. However, the statement in Section 209.07.03 that "*Excavation shall be to the full width and full depth in one operation to backfilling*" should be used only when the excavation will not compromise the stability of the existing road embankment. For organic materials (peat) and soft/loose soils located immediately along the toe of the existing road embankment, the requirement of OPSS 209.07.03 that "*The operations of excavating and backfilling shall be carried out simultaneously*" should be followed. Any excavation beyond the toe of the existing embankment should limit the maximum length, which depends on the actual soil conditions encountered, so that the slope stability of the road embankment will not be compromised at any time. Such an excavation should be backfilled without delay at least to the same original level prior to extending the excavation length. The existing road embankment can not be undermined at any time.

Groundwater may be encountered during excavation due to the presence of sandy soils near the existing ground surface. Excavation into the clayey soils should not encounter significant groundwater seepage within the clayey soils. Other water sources could be creek/swamp water, perched groundwater and/or surface runoff. Dewatering in the excavation within the footprint of the widened road embankment could be carried out by gravity drainage and/or pumping from a properly filtered sump. The base of the excavation should be graded towards a sump in order to drain any surface water inflow into the excavation in order to avoid excessive softening of the road embankment subgrade.

No major excavation difficulties are foreseen but allowance should be made for boulders and cobbles which could be present in fill soils and native soil deposits.

Soft subgrade should be anticipated within the area of the widened road embankment, swamps and the existing culverts. The soft subgrade will not be capable of supporting heavy equipment and construction traffic. Light equipment (e.g, backhoe with a long boom) should first be used for stripping and excavation. The excavation should then be backfilled with rock fill or approved granular fill until it is capable of supporting heavy equipment and construction traffic required for placing fill soils and compaction.

8.11 Construction Inspection

It is recommended that a quality control programme of inspection and testing be carried out during the construction phase of the project to confirm that the conditions encountered are consistent with design assumptions; and to confirm that the various project specifications and material requirements and handling are followed.

9.0 CLOSURE

The sub-soil information and recommendations contained in this report should be used solely for the purpose of foundation assessment of this site.

AMEC should be retained to review the recommendations for this specific applicability, once the details of the proposed works are finalized and prior to the final design stage of the project. Additional investigation may be required to provide geotechnical information for the final design.

This report was prepared by Shami Malla, P.Eng. and Todd Williams, P.Eng. and reviewed by Prapote Boonsinsuk, P.Eng. and George Chow, P.Eng.

The attached Report Limitations is an integral part of this report.

Sincerely,

AMEC Earth & Environmental,
a Division of AMEC Americas Limited

Shami Malla, M.Sc., P.Eng.
Geotechnical Engineer

Prapote Boonsinsuk, Ph.D., P.Eng.
Project Manager and Technical Reviewer

George Chow, P.Eng.
Designated Principal Contact and Project Reviewer

AMEC Earth & Environmental, a division of AMEC Americas Limited

REPORT LIMITATIONS

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 environmental 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. It is recommended practice that the Geotechnical Engineer be retained during the construction to confirm that the subsurface conditions across the site do not deviate materially from those encountered in the testholes.

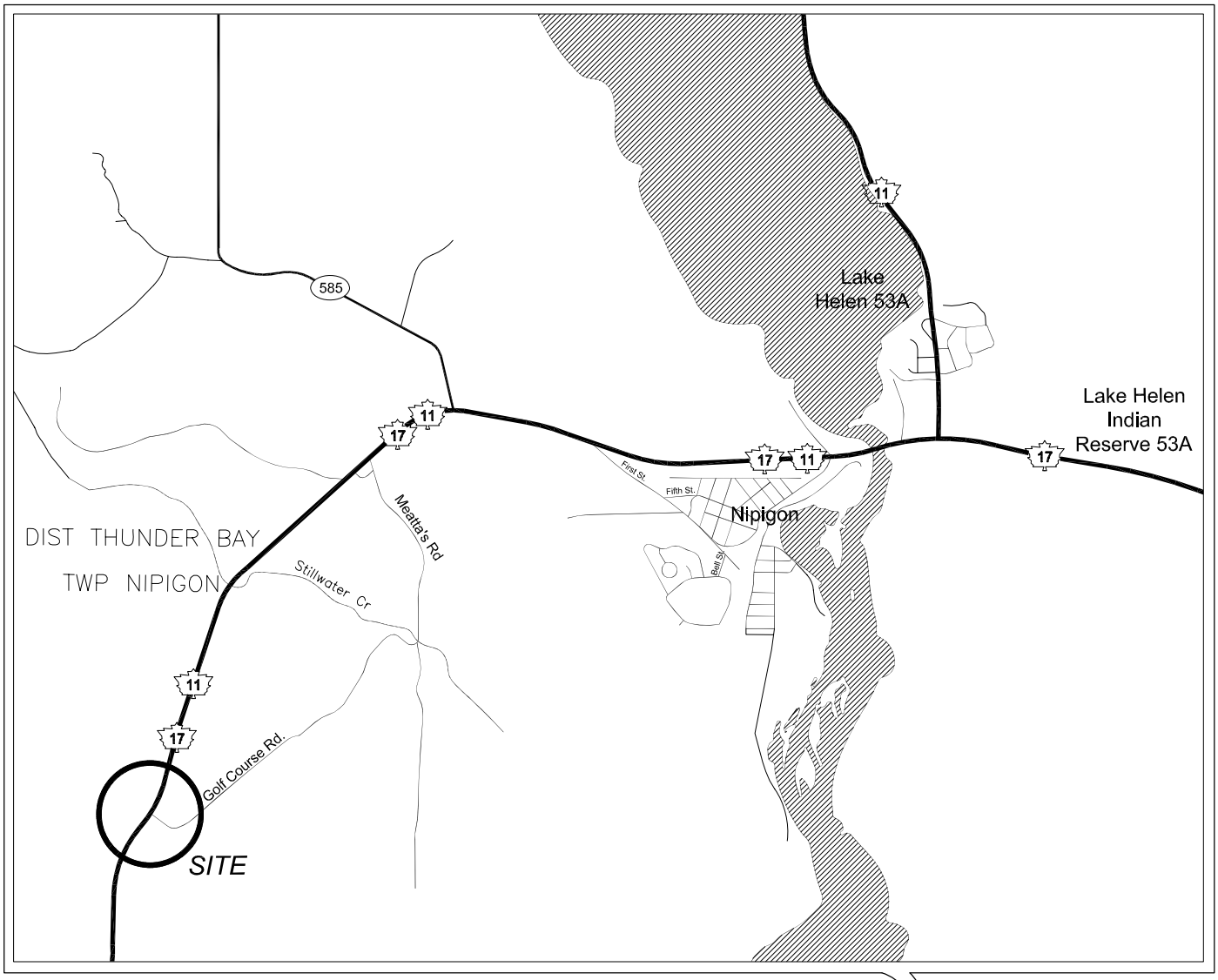
The design recommendations given in this report are applicable only to the project described in the text, and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

The comments made in this report relating to potential construction problems and possible methods of construction 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. No other warranty is expressed or implied.

The benchmark and elevations mentioned in this report were obtained strictly for use by this office in the geotechnical design of the project. They should not be used by any other party for any other purpose.

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. AMEC Earth & Environmental accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

DRAWINGS



SCALE

500m 0 500 1000 1500 2000m

AMEC Earth & Environmental, a Division of AMEC Americas Limited	amec	CLIENT LOGO: Ontario	CLIENT: MINISTRY OF TRANSPORTATION ONTARIO	
TITLE: SITE MAP		DWN BY: KW	DATUM: NAD83	DATE: March 2009
PROJECT: FOUNDATION INVESTIGATION AND DESIGN HIGHWAY 11/17, PROPOSED WESTBOUND PASSING LANE AT GOLF COURSE ROAD, TOWNSHIP OF NIPIGON AGREEMENT NUMBER 6008-E-0027, GWP # 6057-07-00 / W.P. 521-00-26, MTO GEOCRES No.52A-135		CHK'D BY: PB	REV. NO.: A	PROJECT NO: TT93000
		PROJECTION: UTM Zone 16	SCALE: AS SHOWN	DRAWING No. 1

METRIC

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AND/OR MILLIMETRES
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AGREEMENT No.
6008-E-0027

G.W.P. No.
6057-07-00

FOUNDATION INVESTIGATION AND DESIGN
HIGHWAY 11/17, PROPOSED WESTBOUND PASSING LANE AT GOLF COURSE ROAD
TOWNSHIP OF NIPIGON

BOREHOLE LOCATION PLAN

SHEET

AMEC Earth & Environmental,
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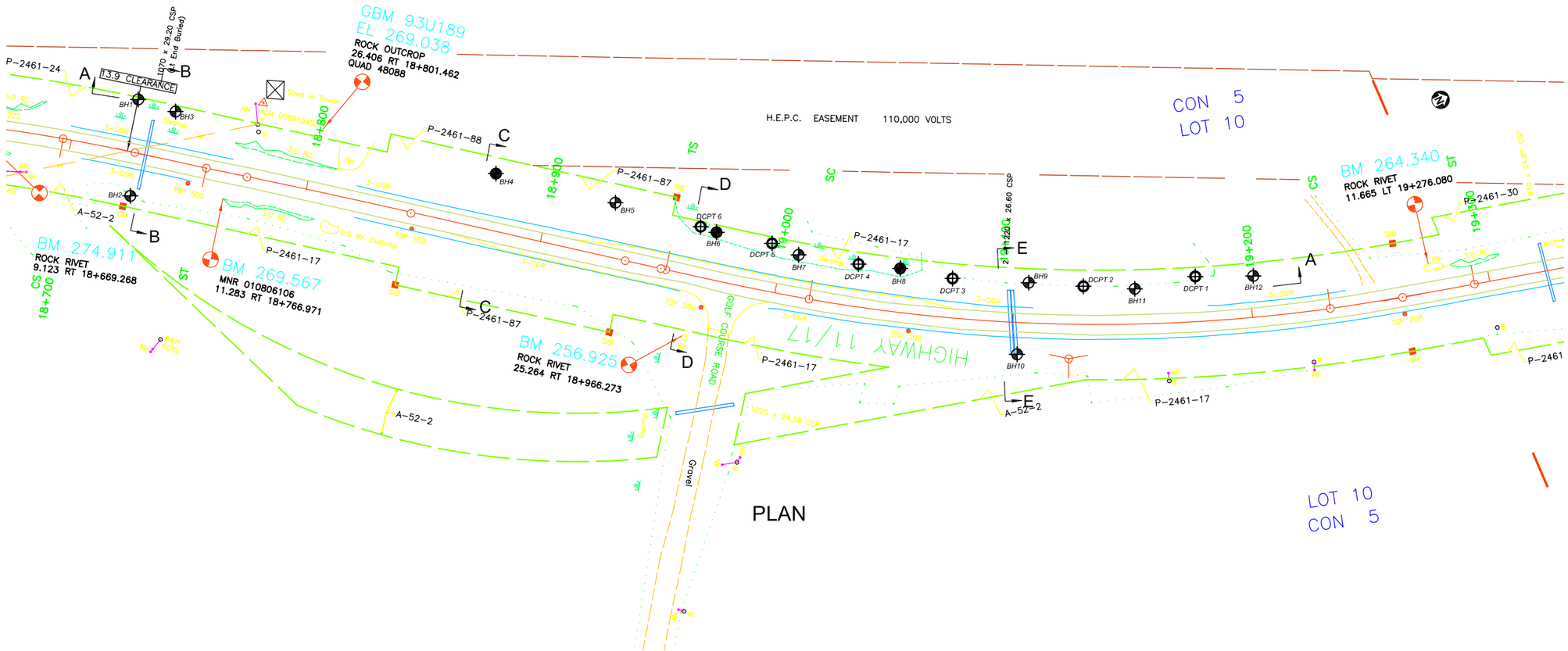
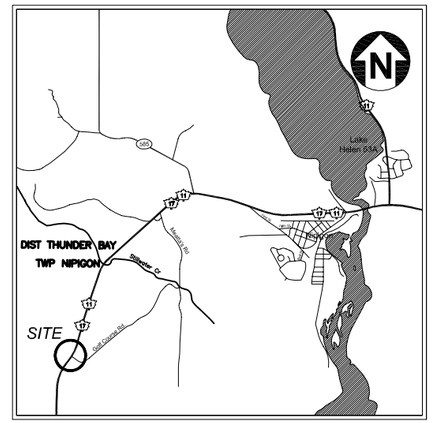


PLAN



KEY PLAN

1000m 0 1000 2000 3000 4000m

Approximate Scale

LEGEND

- BOREHOLE
- DYNAMIC CONE PENETRATION TEST (DCPT)
- BOREHOLE & DCPT

BOREHOLE	MTM COORDINATES		ELEVATION
	NORTHING	EASTING	(m)
BH12	5429383	207533	257.65
DCPT1	5429361	207523	257.37
BH11	5429336	207517	256.87
DCPT2	5429317	207507	256.47
BH9	5429297	207496	255.91
BH10	5429280	207521	255.83
DCPT3	5429269	207481	256.21
BH8	5429251	207468	256.06
DCPT4	5429236	207459	256.17
BH7	5429215	207445	256.45
DCPT5	5429207	207436	256.63
BH6	5429188	207422	256.73
DCPT6	5429183	207417	256.82
BH5	5429155	207393	259.87
BH4	5429115	207361	262.39
BH3	5429005	207281	267.47
BH1	5428993	207270	267.84
BH2	5428973	207305	266.85

0 20 40 60m

SCALE

REVISIONS

DESIGN PB	CHK PB	CODE		DATE MAR. 2009
DRAWN KW	CHK GC	SITE		DWG 2

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LONGITUDINAL SOIL PROFILE (along west existing road embankment toe)

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

AGREEMENT No. 6008-E-0027
G.W.P. No. 6057-07-00

FOUNDATION INVESTIGATION AND DESIGN
HIGHWAY 11/17, PROPOSED WESTBOUND PASSING LANE AT GOLF COURSE ROAD
TOWNSHIP OF NIPIGON

LONGITUDINAL SOIL PROFILE

SHEET

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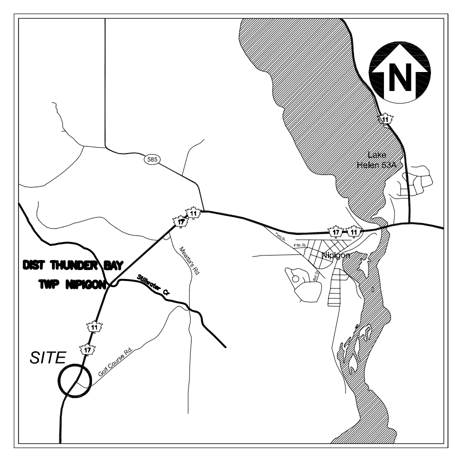

KEY PLAN
1000m 0 1000 2000 3000 4000m
Approximate Scale

LEGEND

- BOREHOLE
- DYNAMIC CONE PENETRATION TEST (DCPT)
- BOREHOLE AND DCPT
- 'N' BLOWS / FT (STD PEN TEST 350ft lbs energy)
- GROUND WATER LEVEL AT TIME OF INVESTIGATION
- ARTESIAN WATER ENCOUNTERED

BOREHOLE	MTM COORDINATES NORTHING	MTM COORDINATES EASTING	ELEVATION (m)
BH12	5429383	207533	257.65
DCPT1	5429361	207523	257.37
BH11	5429336	207517	256.87
DCPT2	5429317	207507	256.47
BH9	5429297	207496	255.91
BH10	5429280	207521	255.83
DCPT3	5429269	207481	256.21
BH8	5429251	207468	256.06
DCPT4	5429236	207459	256.17
BH7	5429215	207445	256.45
DCPT5	5429207	207436	256.63
BH6	5429188	207422	256.73
DCPT6	5429183	207417	256.82
BH5	5429155	207393	259.87
BH4	5429115	207361	262.39
BH3	5429005	207281	267.47
BH1	5428993	207270	267.84
BH2	5428973	207305	266.85

NOTES:
The boundaries between soil strata have been established only at borehole locations. Between boreholes, the boundaries are assumed from geological evidence and may be subject to considerable error.

SOIL STRATIGRAPHY

- SAND
- SANDS & SILTS (Cohesionless Soils)
- CLAYS & SILTS (Clayey Silt / Silty Clay)
- SAND & GRAVEL
- SILTS
- ORGANICS OR TOPSOIL
- FILL
- BEDROCK
- CLAYS

STRATIGRAPHIC PROFILE
0 9 18 27m
SCALE

LICENSED PROFESSIONAL ENGINEER SHAMI MALLA PROVINCE OF ONTARIO
LICENSED PROFESSIONAL ENGINEER G. S. W. CHOW PROVINCE OF ONTARIO

REVISIONS

DESIGN PB	CHK PB			DATE MAR. 2009
DRAWN KW	CHK GC	SITE		DWG 3

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METRIC
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AGREEMENT No. 6008-E-0027

G.W.P. No. 6057-07-00

FOUNDATION INVESTIGATION REPORT
HIGHWAY 11/17, PROPOSED WESTBOUND PASSING LANE AT GOLF COURSE ROAD
TOWNSHIP OF NIPIGON

TRANSVERSE SOIL PROFILES

SHEET

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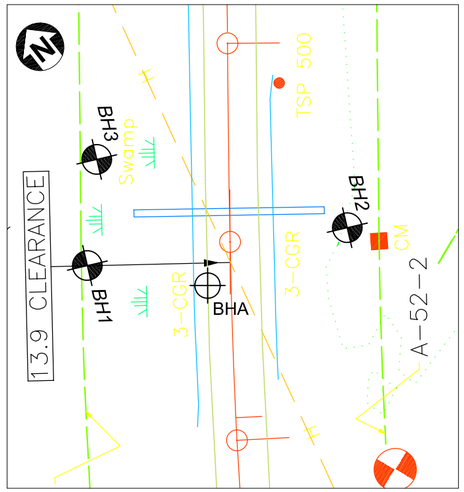
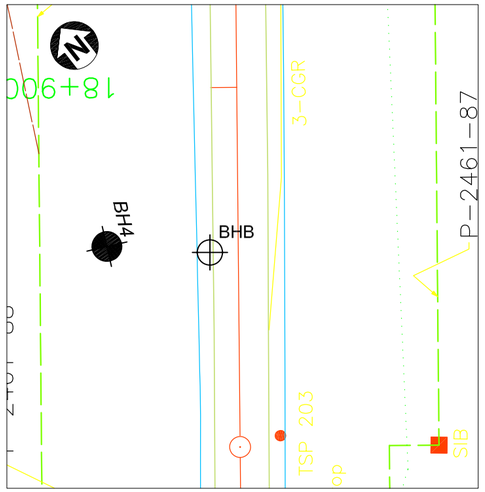
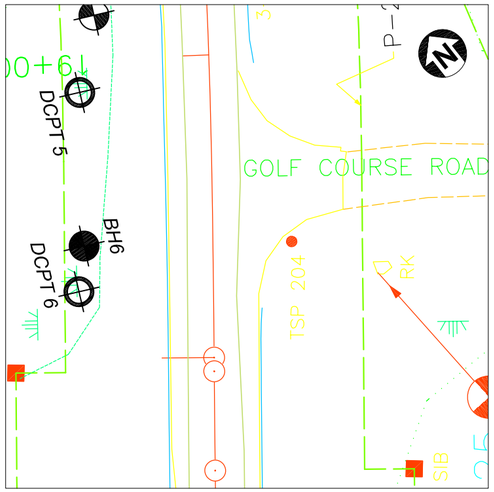
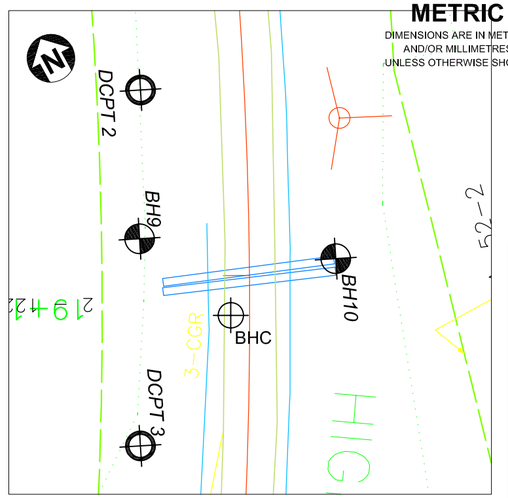
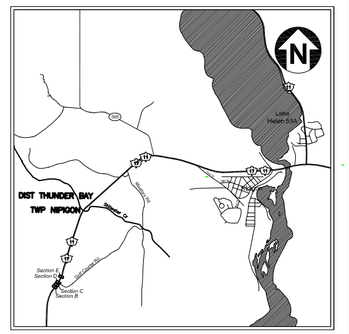








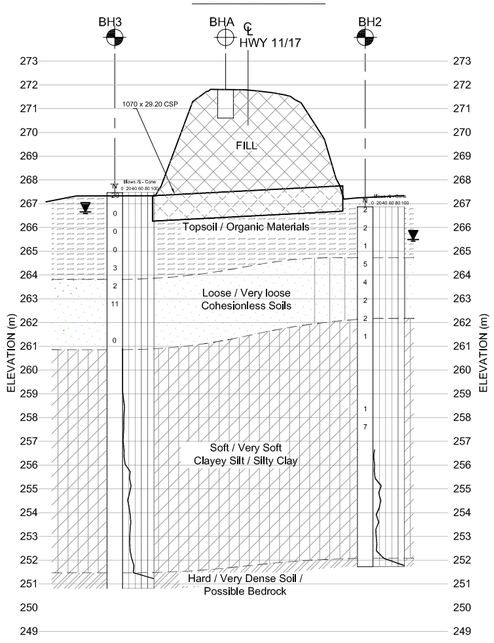


KEY PLAN

1000m 0 1000 2000 3000 4000m

Approximate Scale

SECTION B-B



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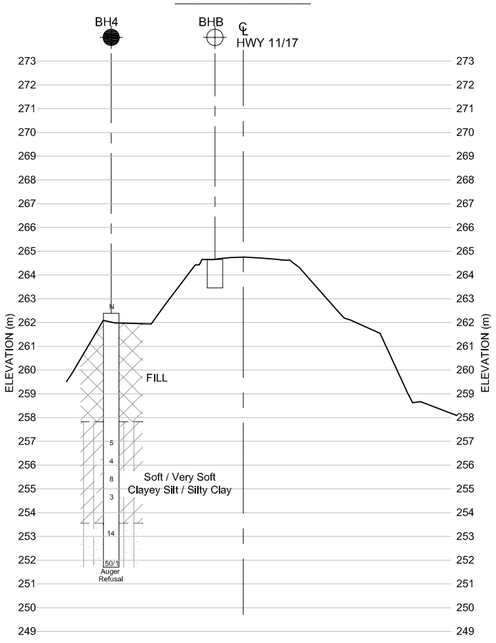

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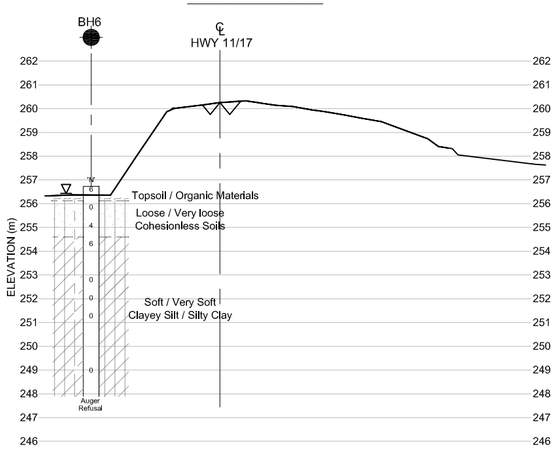

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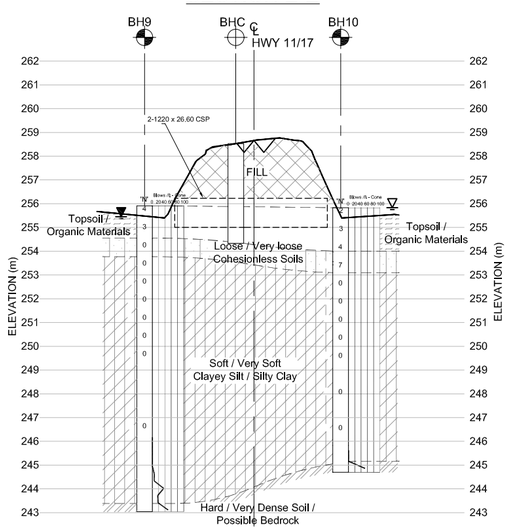

SOIL STRATIGRAPHY

- SAND
- SANDS & SILTS (Cohesionless Soils)
- CLAYS & SILTS (Clayey Silt / Silty Clay)
- SANDS & GRAVEL
- SILTS
- ORGANICS OR TOPSOIL
- FILL
- BEDROCK
- CLAYS

LEGEND

- BOREHOLE
- DYNAMIC CONE PENETRATION TEST (DCPT)
- BOREHOLE AND DCPT
- PREVIOUSLY DRILLED BOREHOLE BY MTO
- 'N' BLOWS / FT (STD PEN TEST 350ft lbs energy)
- GROUND WATER LEVEL AT TIME OF INVESTIGATION
- ARTESIAN WATER ENCOUNTERED

BOREHOLE	MTM COORDINATES NORTHING	MTM COORDINATES EASTING	ELEVATION (m)
BH3	5429005	207281	267.47
BH2	5428973	207305	266.85
BH4	5429115	207361	262.39
BH6	5429188	207422	256.73
BH9	5429297	207496	255.91
BH10	5429280	207521	255.83
PREVIOUSLY DRILLED BOREHOLES (MTO 2002)	STATION	OFFSET	ELEVATION (m)
BHA	18+729	4.2 Lt	-
BHB	18+870	4.2 Lt	-
BHC	19+096	4.6 Lt	-

NOTES:
The boundaries between soil strata have been established only at borehole locations. Between boreholes, the boundaries are assumed from geological evidence and may be subject to considerable error.

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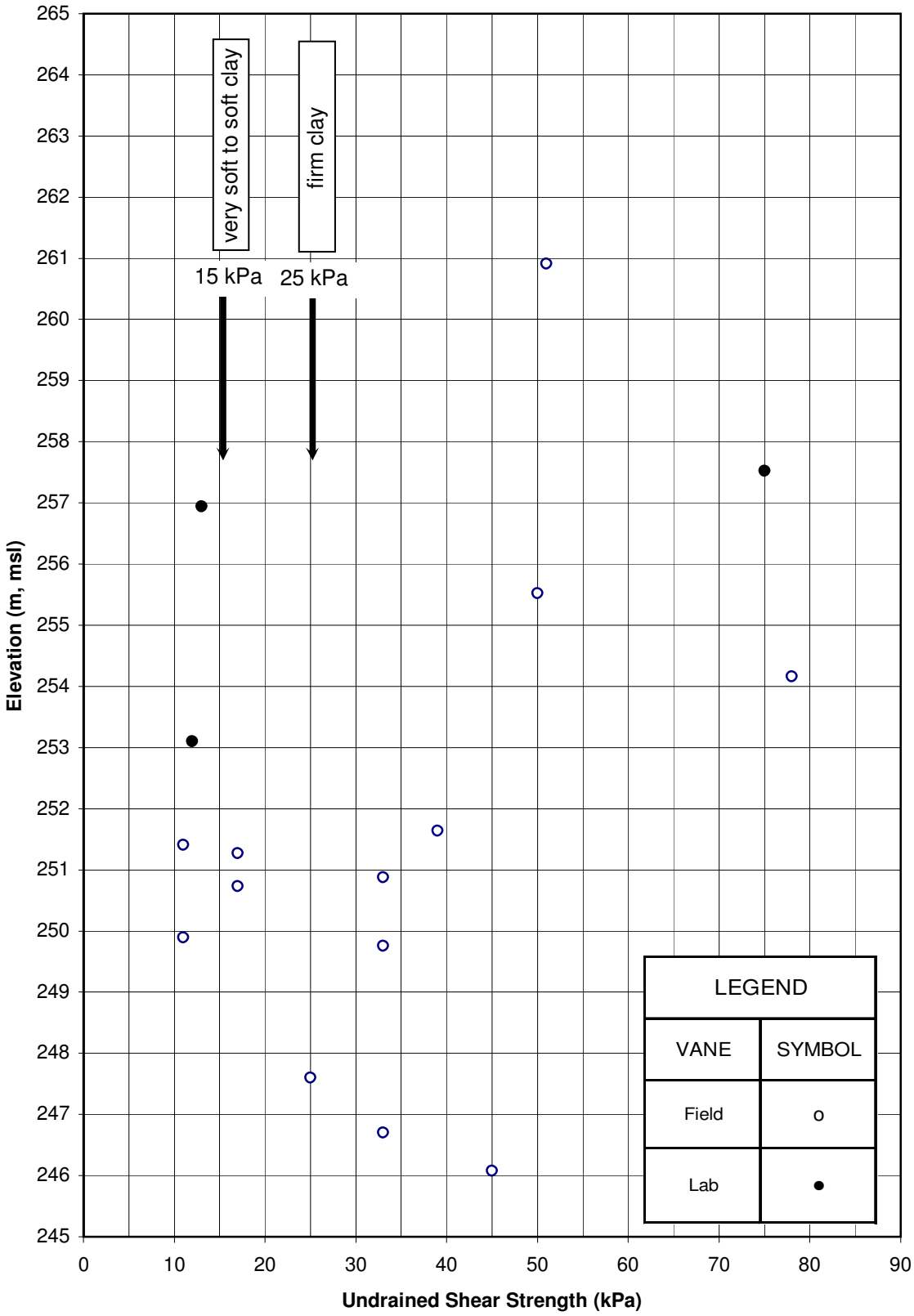
STRATIGRAPHIC PROFILE
0 9 18 27m
SCALE

REVISIONS

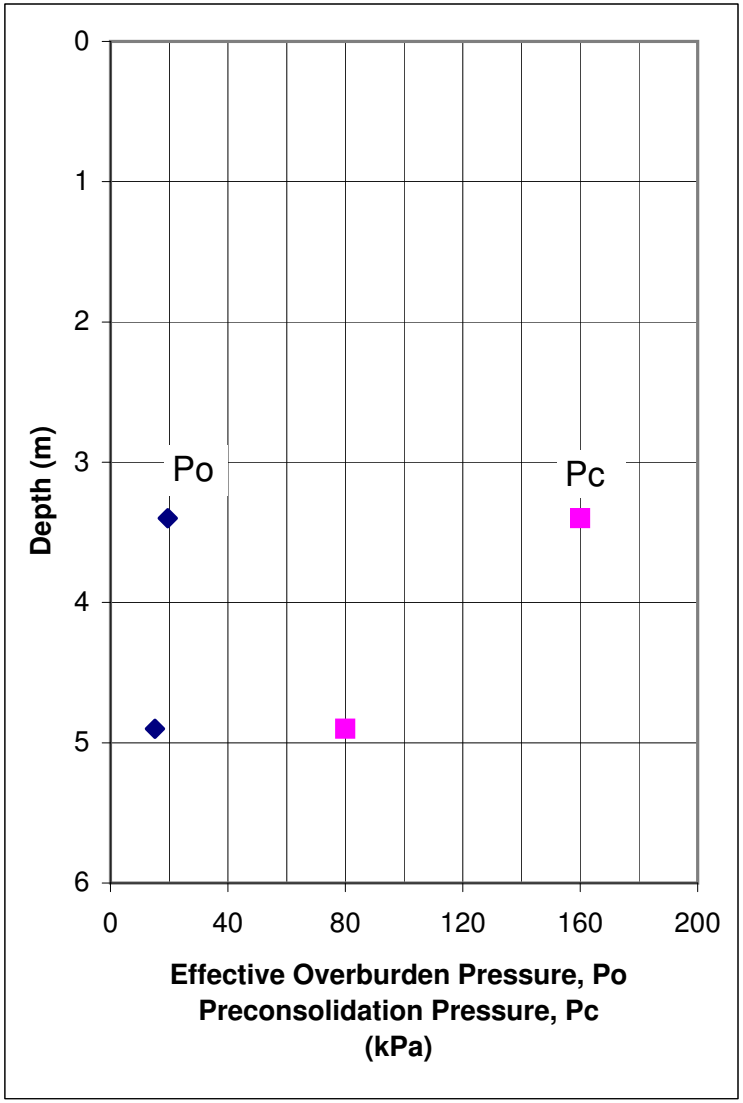
DESIGN PB	CHK PB			DATE MAR. 2009
DRAWN KW	CHK GC	SITE		DWG 4

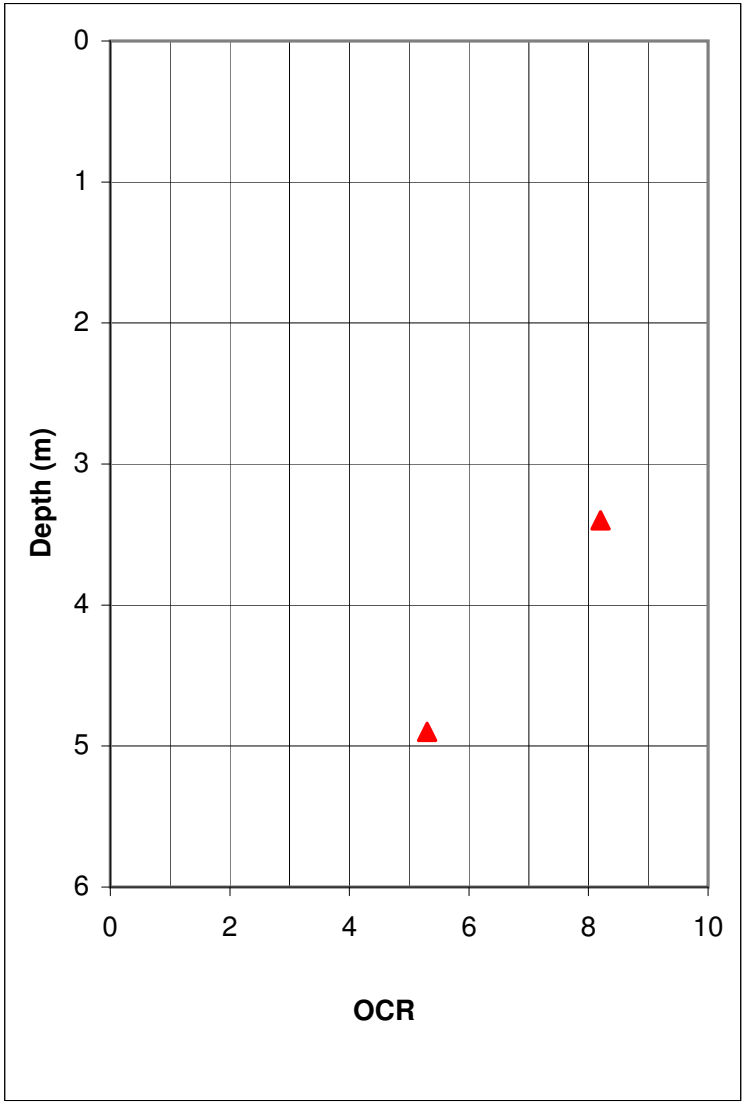
P:\GEO\Projects\2009\TT93000 - Foundation Engineering - MTO 6008-E-0027\Drawings AMEC\TT93000 - Nipigon_Profile_1_and_2.dwg

FIGURES



Ontario	**VARIATION OF VANE SHEAR STRENGTH WITH ELEVATION**	FIGURE 1 CA # 6008-E-0027 Hwy. 11/ 17 Twp.





Ontario

VARIATION OF Po, Pc AND OCR
WITH DEPTH BELOW GROUND SURFACE

FIGURE 2
CA # 6008-E-0027
Hwy. 11/ 17 Twp. Nipigon

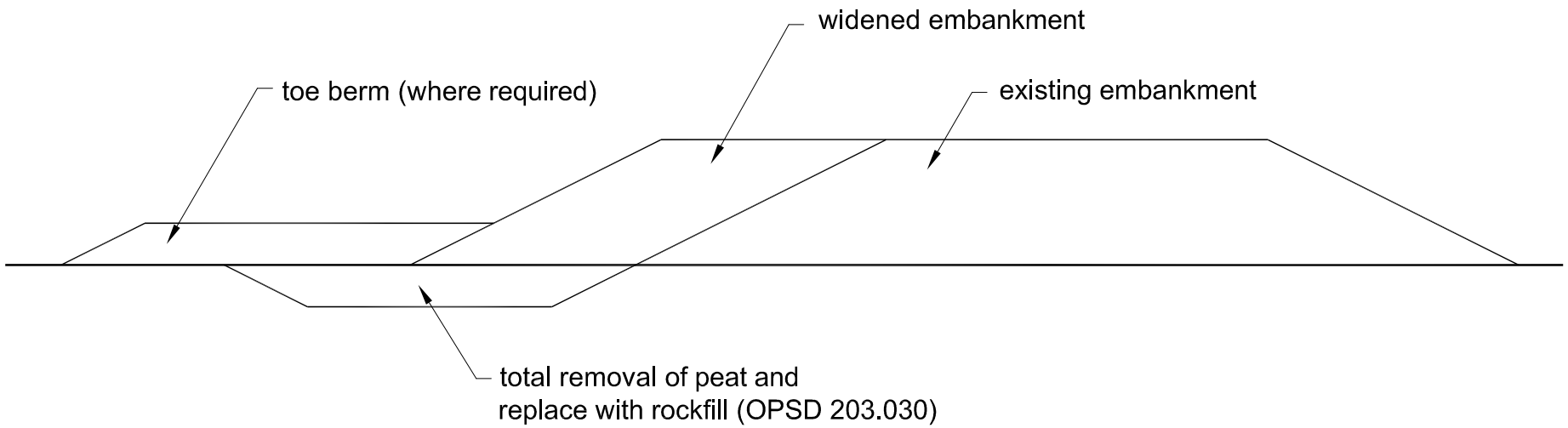


Figure 3a - Proposed Configuration of Widened Embankment

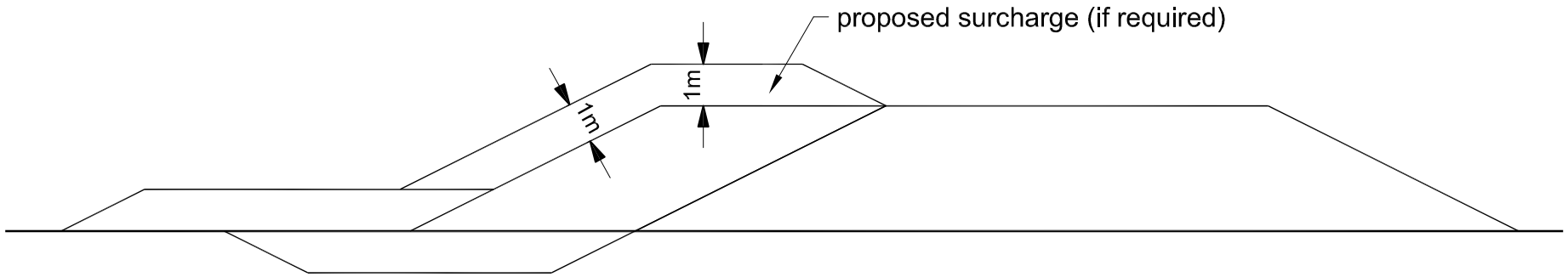


Figure 3b - Proposed Configuration of 1m Thick Surcharge (if required)

APPENDIX A

RECORD OF BOREHOLES / DCPT TEST HOLES

EXPLANATION OF BOREHOLE LOG

This form describes some of the information provided on the borehole logs, which is based primarily on examination of the recovered samples, and the results of the field and laboratory tests. Additional description of the soil/rock encountered is given in the accompanying geotechnical report.

GENERAL INFORMATION

Project details, borehole number, location coordinates and type of drilling equipment used are given at the top of the borehole log.

SOIL LITHOLOGY

Elevation and Depth

This column gives the elevation and depth of inferred geologic layers. The elevation is referred to the datum shown in the Description column.

Lithology Plot

This column presents a graphic depiction of the soil and rock stratigraphy encountered within the borehole.

Description

This column gives a description of the soil stratums, based on visual and tactile examination of the samples augmented with field and laboratory test results. Each stratum is described according to the *Modified Unified Soil Classification System*.

The compactness condition of cohesionless soils (SPT) and the consistency of cohesive soils (undrained shear strength) are defined as follows *(Ref. Canadian Foundation Engineering Manual)*:

Compactness of Cohesionless Soils	SPT N-Value*
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

Consistency of Cohesive Soils	Undrained Shear Strength kPa	Undrained Shear Strength psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	Over 200	Over 4000

* For penetration of less than 0.3 m, N-values are indicated as the number of blows for the penetration achieved (e.g. 50/25: 50 blows for 25 centimeter penetration).

Soil Sampling

Sample types are abbreviated as follows:

SS Split Spoon	TW Thin Wall Open (Pushed)	RC Rock Core	GS Grab Sample
AS Auger Sample	TP Thin Wall Piston (Pushed)	WS Washed Sample	AR Air Return Sample

Additional information provided in this section includes sample numbering, sample recovery and numerical testing results.

Field and Laboratory Testing

Results of field testing (e.g., SPT, pocket penetrometer, and vane testing) and laboratory testing (e.g., natural moisture content, and limits) executed on the recovered samples are plotted in this section.

Instrumentation Installation

Instrumentation installations (monitoring wells, piezometers, inclinometers, etc.) are plotted in this section. Water levels, if measured during fieldwork, are also plotted. These water levels may or may not be representative of the static groundwater level depending on the nature of soil stratum where the piezometer tips are located, the time elapsed from installation to reading and other applicable factors.

Comments

This column is used to describe non-standard situations or notes of interest.

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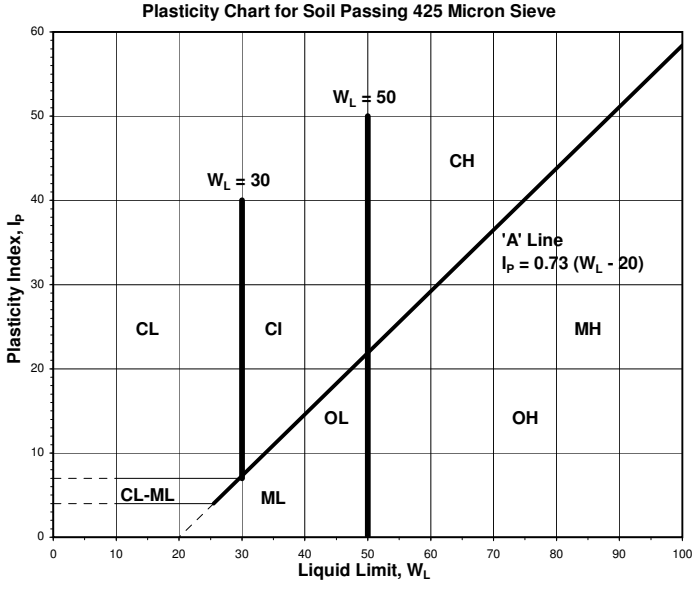
MODIFIED * UNIFIED CLASSIFICATION SYSTEM FOR SOILS

*The soil of each stratum is described using the Unified Soil Classification System (Technical Memorandum 36-357 prepared by Waterways Experiment Station, Vicksburg, Mississippi, Corps of Engineers, U.S Army. Vol. 1 March 1953.) modified slightly so that an inorganic clay of "medium plasticity" is recognized.

MAJOR DIVISION			GROUP SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75µm)	GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	CLEAN GRAVELS (TRACE OR NO FINES)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 4$; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTY GRAVELS (WITH SOME OR MORE FINES)	GM	SILTY GRAVELS, GRAVEL-SAND- SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 4
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 7
	SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm	CLEAN SANDS (TRACE OR NO FINES)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 6$; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3
			SP	POORLY GRADED GRAVELS, GRAVEL- SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTY SANDS (WITH SOME OR MORE FINES)	SM	SILTY SANDS, SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR P.I MORE THAN 7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75µm)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	W_L < 50%	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)
		W_L < 50%	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS	
	CLAYS ABOVE "A" LINE NEGLIGIBLE ORGANIC CONTENT	W_L < 30%	CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS	
		30% < W_L < 50%	CI	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS	
		W_L < 50%	CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
	ORGANIC SLITS & CLAYS BELOW "A" LINE	W_L < 50%	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINES CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G SF IS A MIXTURE OF SAND WITH SILT OR CLAY
		W_L < 50%	OH	ORGANIC CLAYS OF HIGH PLASTICITY	
HIGH ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE

SOIL COMPONENTS					
FRACTION	U.S STANDARD SIEVE SIZE		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS		
GRAVEL	COARSE	PASSING	RETAINED	PERCENT	DESCRIPTOR
		76 mm	19 mm	35-50	AND
				20-35	Y/EY
	FINE	19 mm	4.75 mm	10-20	SOME
				1-10	TRACE
SAND	COARSE	4.75 mm	2.00 mm		
	MEDIUM	2.00 mm	425 µm		
	FINE	425 µm	75 µm		
FINES (SILT OR CLAY BASED ON PLASTICITY)		75 µm			
OVERSIZED MATERIAL					
ROUNDED OR SUBROUNDED: COBBLES 76 mm TO 200 mm BOULDERS > 200 mm				NOT ROUNDED: ROCK FRAGMENTS > 76 mm ROCKS > 0.76 CUBIC METRE IN VOLUME	



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Note 1: Soils are classified and described according to their engineering properties and behaviour.

Note 2: The modifying adjectives used to define the actual or estimated percentage range by weight of minor components are consistent with the Canadian Foundation Engineering Manual (4th Edition, Canadian Geotechnical Society, 2006.)

Ontario

RECORD OF BOREHOLE No BH 1

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 18+722 Offset: 21.9 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 15 January 2009 - 15 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
267.8 / 0.0	brown to grey **SANDY SILT** very loose frozen soil		1	SS	2				25	Ice and snow at ground surface to depth of about 0.2 m
267.2 / 0.6	black **ORGANIC MATERIALS** fibrous peat some wood fragments		2	SS	1		1	267	30	
			3	SS	0	▽	2	266		No recovery
			4	SS	2				140	
			5	SS	0		3	265	25	
264.2 / 3.7	grey **SANDY SILT** loose wet		6	SS	10		4	264	15	
263.6 / 4.3	grey **CLAYEY SILT / SILTY CLAY** trace sand moist		7	TW			5	263		0 2 74 24
	boulder from 5.5 m to 5.9 m depth						6	262		
261.9 / 5.9	grey **SAND** trace gravel and silt compact wet		8	SS	28				0	
							7	261		Water flowing into augers
259.8 / 8.1	**End of Borehole** Sand flowing significantly into hollow stem augers preventing further augering. Changed to Dynamic Cone Penetration Test (DCPT) below 8.1 m depth.		9	SS	9		8	260	0	2 94 (4)
							9	259		
257.9								258		DCPT = 50 blows/0" at 9.9 m depth

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 1

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 18+722 Offset: 21.9 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 15 January 2009 - 15 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT / WATER CONTENT (%)	SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		m	m	○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE; 20 40 60 80 100	w_P w w_L; 10 20 30	PPM	GR SA SI CL
9.9	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 9.9 m depth Groundwater in open borehole on completion: 1.7 m Borehole was backfilled with bentonite at the completion of drilling.											

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 2

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 18+731 Offset: 18.0 Rt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Solid Stem Augering and Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 12 January 2009 - 12 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" VALUES	DEPTH (m)	ELEVATION (m)	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
266.9								
0.0	black **ORGANIC MATERIALS** fibrous peat some wood fragments	1	SS	2			80	Ice and snow at ground surface to depth of about 1.2 m
		2	SS	2	1	266	75	
		3	SS	1			80	
264.7					2	265		
2.1	grey **SILTY SAND** trace clay and gravel very loose to loose wet	4	SS	5			80	
		5	SS	4	3	264	60	
		6	SS	2	4	263	60	2 63 31 4
262.1								
4.7	grey **CLAYEY SILT / SILTY CLAY** very soft to firm moist	7	SS	2	5	262	75	
		8	SS	1			25	
		9	TW		6	261		No recovery
		10	TW		7	260		No recovery
		11	SS	1	8	259	10	
		12	SS	7	9	258	10	
		13	TW			257		

GROUND WATER CONDITIONS: ▽

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

Plotted values: 396, 126, 389, 15, 21, 32, 16, 31, 34, 5, 34, 26

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 2

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 18+731 Offset: 18.0 Rt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Solid Stem Augering and Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 12 January 2009 - 12 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	DEPTH (m)	ELEVATION SCALE (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.6												
10.2	**End of Borehole** Dynamic Cone Penetration Test (DCPT) was conducted below 10.2 m depth.						11 12 13 14 15	256 255 254 253 252				DCPT = 100 blows/2" at 15.1 m depth
251.7												
15.1	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 15.1 m depth Groundwater in open borehole on completion: 1.2 m Borehole was backfilled with bentonite at the completion of drilling.											

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: 20 40 60 80 100

WATER CONTENT (%): 10 20 30

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 3

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 18+740 Offset: 21.2 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 16 January 2009 - 16 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
267.5										
0.0	black **ORGANIC MATERIALS** fibrous peat some wood fragments		1	SS	20			267	50	Ice and snow at ground surface to depth of about 0.2 m
			2	SS	0	▽	1		45	
			3	SS	0		2	266	35	
			4	SS	0			265		No recovery
			5	SS	3		3		15	
263.8								264		
3.7	grey **SAND** very loose to compact wet 50 mm thick silty clay seam encountered at 4.0 m depth		6	SS	2		4		0	
	trace wood fragments		7	SS	11		5	263	0	
			8	SS	0		6	262		
260.9								261		
6.6	**End of Borehole** Sand flowing significantly into hollow stem augers preventing further augering. Changed to Dynamic Cone Penetration Test (DCPT) below 6.6 m depth.						7	260		
							8			
								259		
							9			
								258		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

489 ○ 35

23 ○ 0

20 ○ 0

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 3

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 18+740 Offset: 21.2 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 16 January 2009 - 16 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	DEPTH (m)	ELEVATION SCALE (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
250.8							11	257				DCPT = 100 blows/2" at 16.6 m depth
							12	256				
							13	255				
							14	254				
							15	253				
							16	252				
16.7	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 16.7 m depth Groundwater in open borehole on completion: 0.8 m Borehole was backfilled with bentonite at the completion of drilling.							251				

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: 20 40 60 80 100

WATER CONTENT (%): 10 20 30

$+^3$, $\times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 4

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 18+875 Offset: 20.2 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 18 January 2009 - 18 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
262.4										
0.0	**ROCK FILL** 150 mm to 1500 mm rock pieces mixed with sand and gravel									
257.8										
4.6	brown to grey **CLAYEY SILT / SILTY CLAY** trace sand and gravel soft to stiff moist		1	TW						1 5 51 41
			2	SS	5					
			3	SS	4					
			4	SS	8					
	some sand		5	SS	3					
253.6										
8.8	grey **SANDY SILT** some clay compact moist		6	SS	14					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

Plotted values: 18, 30, 38; 30; 36; 17, 27, 31; 4; 22

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 4

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 18+875 Offset: 20.2 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 18 January 2009 - 18 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%) 10 20 30	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SANDY SILT											DCPT = 50 blows/0" at 10.7 m depth
251.7								252				
10.7	**End of Borehole**		7	SS	50/1							
	No noticeable groundwater in open borehole on completion											
	Dynamic Cone Penetration Test (DCPT) was conducted below 10.7 m depth.											
	Refusal to Dynamic Cone Penetration Test at 10.7 m depth											
	Borehole was backfilled with bentonite at the completion of drilling.											

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 5

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 18+925 Offset: 22.7 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 17 January 2009 - 17 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
259.9 / 0.0	**ROCK FILL** 150 mm to 1500 mm rock pieces mixed with sand and gravel									
255.3 / 4.6	grey **CLAYEY SILT / SILTY CLAY** trace sand very soft to firm moist		1	SS	4	▽			25	
			2	SS	2				15	
			3	SS	2				25	
			4	SS	0				25	
			5	SS	0				15	0 2 73 25
			6	SS	0				0	

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)

SHEAR STRENGTH kPa (20 40 60 80 100)
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%) (10 20 30)

17 40 39

35

17 27 33

2

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 5

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 18+925 Offset: 22.7 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 17 January 2009 - 17 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT / SILTY CLAY											
248.7			7	SS	0		11	249		17 / 48	0	
11.1	**End of Borehole** Dynamic Cone Penetration Test (DCPT) was conducted below 11.1 m depth.											DCPT = 120 blows/9" at 11.7 m depth
247.9								248				
12.0	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 12.0 m depth Groundwater in open borehole on completion: 4.0 m Borehole was backfilled with bentonite at the completion of drilling.											

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 6

1 OF 1

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 18+970 Offset: 19.8 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Hollow Stem Augering | COMPILED BY SN

DATUM Geodetic | DATE 17 January 2009 - 17 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	DEPTH (m)	ELEVATION SCALE (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.7												
0.0	black **ORGANIC MATERIALS** fibrous peat some wood fragments		1	SS	6	▽					70	Ice and snow at ground surface to depth of about 0.6 m
256.1												
0.6	yellowish brown **SANDY SILT** trace to some clay with clayey zones, trace gravel very loose wet		2	SS	0		1	256			50	
	clayey sample at SS 3		3	SS	4		2	255		15 27 (23)	0	4 31 49 16
254.6												
2.1	grey **CLAYEY SILT / SILTY CLAY** very soft moist		4	SS	6		3	254		17 41 (28)	15	
			5	TW						17 44 (39)		0 0 38 62
			6	SS	0		4	253			0	
			7	SS	0		5	252		(46)	0	
			8	SS	0		6	251			0	
			9	SS	0		7	250				
							8	249		(49)	0	
247.9								248				
8.8	**End of Borehole due to Auger Refusal**											

Groundwater in open borehole on completion: 0.3 m

Borehole backfilled with bentonite at completion of drilling.

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 7

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 19+010 Offset: 17.4 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 17 January 2009 - 17 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.5										
0.0	**TOPSOIL**		1	SS	0	▽		256		Ice and snow at ground surface to depth of about 0.6 m
255.5							1			
0.9	brown **SANDY SILT** some clay very loose wet		2	SS	4			255	20	
			3	SS	5		2		20	
254.3								254		
2.1	grey **CLAYEY SILT / SILTY CLAY** trace sand very soft to firm moist		4	SS	2				50	
			5	TW			3	253		
			6	SS	0		4		0	
			7	SS	0			252	20	
							5			
			8	SS	0			251	10	
							6	250		
							7			
			9	SS	0			249	25	
							8			
								248		
			10	SS	0		9		20	0 3 36 61
246.5								247		

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)

SHEAR STRENGTH kPa (20 40 60 80 100)
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%) (10 20 30)

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 7

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 19+010 Offset: 17.4 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 17 January 2009 - 17 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	DEPTH (m)	ELEVATION SCALE (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
9.9	**End of Borehole** Dynamic Cone Penetration Test (DCPT) was conducted below 9.9 m depth.						11 12	246 245				DCPT = 100 blows/3" at 12.3 m depth
244.0												
12.4	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 12.4 m depth Groundwater in open borehole on completion: 0.6 m Borehole was backfilled with bentonite at the completion of drilling.											

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

Ontario

amec

RECORD OF BOREHOLE No BH 8

G.W.P. 6057-07-00	LOCATION HWY 11/17 - Station: 19+050 Offset: 18.3 Lt		ORIGINATED BY SAL
DIST Thunder Bay HWY 11/17	BOREHOLE TYPE Hollow Stem Augering		COMPILED BY SN
DATUM Geodetic	DATE 16 January 2009 - 16 January 2009		CHECKED BY PB
PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon			JOB NO. TT93000

SOIL PROFILE

ELEV / DEPTH (m)	DESCRIPTION
256.1 / 0.0	grey to brown **SANDY SILT** trace rootlets and wood fragments in SS2 very loose wet
253.9 / 2.1	grey **CLAYEY SILT / SILTY CLAY** very soft to firm moist
247.4 / 8.7	grey **POSSIBLE SAND AND GRAVEL** very dense
246.6 / 9.4	**End of Borehole** Sand flowing significantly into augers preventing further augering.

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	Shear strength (kPa)	w_P	w	w_L	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	5	▽					50	Ice and snow at ground surface to depth of about 0.5 m
2	SS	2				43		50	
3	SS	7						30	
4	SS	0				40		0	
5	SS	0			17	45	42	0	0 0 35 65
6	SS	0						25	
7	SS	0			16	35	33	0	
				+ 4 (field vane)					
8	SS	0						0	
9	SS	0				32		10	
10	SS	0			17	64	49	25	
11	SS	50/13							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

Depth scale (m): 1–9; Elevation scale (m): 256–247

+ 3, × 3: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 8

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 19+050 Offset: 18.3 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Hollow Stem Augering COMPILED BY SN

DATUM Geodetic DATE 16 January 2009 - 16 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT / WATER CONTENT (%)	SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		m	m	○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE; 20 40 60 80 100	w_P w w_L; 10 20 30	PPM	GR SA SI CL
	Groundwater in open borehole on completion: 0.3 m Borehole backfilled with bentonite at completion of drilling.											

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 9

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 19+102 Offset: 16.8 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 14 January 2009 - 14 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
255.9										
0.0	black **ORGANIC MATERIALS** fibrous peat some wood fragments		1	SS	4	▽			25	
			2	SS	3		1	255	40	
254.5										
1.4	grey **SILTY SAND** very loose wet		3	SS	0		2	254	60	
253.8										
2.1	grey **CLAYEY SILT / SILTY CLAY** trace sand very soft to soft moist		4	SS	0				70	
			5	SS	0		3	253	15	
			6	SS	0		4	252	0	
			7	SS	0		5	251	0	0 1 39 60
			8	SS	0		6	250	0	
			9	SS	0		7	249	0	
			10	TW			8	248		No recovery
			11	SS	0		9	247	10	
246.2										
9.8	**End of Borehole**							246		

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)

SHEAR STRENGTH kPa (20 40 60 80 100)
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%) (10 20 30)

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 9

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 19+102 Offset: 16.8 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 14 January 2009 - 14 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT / WATER CONTENT (%)	SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		m	m	○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE; 20 40 60 80 100	w_P w w_L; 10 20 30	PPM	GR SA SI CL
	Dynamic Cone Penetration Test (DCPT) was conducted below 9.8 m depth.						11	245				
							12	244				
243.0												DCPT = 50 blows/3" at 12.8 m depth
12.9	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 12.9 m depth Groundwater in open borehole on completion: 0.5 m Borehole was backfilled with bentonite at the completion of drilling.											

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 10

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 19+102 Offset: 13.3 Rt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 12 January 2009 - 13 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" VALUES	DEPTH (m)	ELEVATION (m)
255.8						
0.0	black **ORGANIC MATERIALS** fibrous peat some wood fragments, occasional sand/silty sand seam	1	SS	2		
		2	SS	3	1	255
254.0		3	SS	4		
1.8	grey **SANDY SILT / SILTY SAND** very loose wet	4	SS	7	2	254
253.1						
2.7	grey **CLAYEY SILT / SILTY CLAY** very soft to firm moist	5	SS	0	3	253
		6	SS	0	4	252
		7	SS	0	5	251
		8	SS	0	6	250
	trace sand	9	SS	0	7	249
		10	SS	0	8	248
		11	TW		9	247
		12	SS	0		
245.8						

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)

SHEAR STRENGTH kPa (20 40 60 80 100)
- ○ UNCONFINED
- ● QUICK TRIAXIAL
- + FIELD VANE
- × LAB VANE

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%) (10 20 30)

SOIL VAPOUR READING (PPM): 40, 100, 50, 15, 10, 0

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

No recovery

No recovery

Continued Next Page

$+^{3}, \times^{3}$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 10

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 19+102 Offset: 13.3 Rt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 12 January 2009 - 13 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT / WATER CONTENT (%)	SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		m	m	○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE; 20 40 60 80 100	w_P w w_L; 10 20 30	PPM	GR SA SI CL
10.0	**End of Borehole** Dynamic Cone Penetration Test (DCPT) was conducted below 10.1 m depth.											
244.7							11	245				
11.1	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 11.1 m depth Groundwater in open borehole on completion: 0.0 m Borehole was backfilled with bentonite at the completion of drilling. After backfilling, no groundwater seepage to the ground surface was visible.											

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 11

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 19+150 Offset: 14.7 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Hollow Stem Augering | COMPILED BY SN

DATUM Geodetic | DATE 14 January 2009 - 14 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.5										
0.0	yellowish brown **SANDY SILT / SILTY SAND** trace to some organics in SS1 very loose to compact wet		1	SS	3				50	
			2	SS	13		1	256	60	
			3	SS	8		2	255	25	
254.3										
2.1	grey **CLAYEY SILT / SILTY CLAY** trace sand very soft to firm moist		4	SS	0		3	254	20	
			5	SS	0			253	0	
			6	SS	0		4	252	0	
			7	SS	0		5	251	0	
			8	SS	0		6	250	0	0 5 71 25
			9	SS	0		7	249	0	
249.3										
7.2	grey **SAND AND GRAVEL** occasional cobbles dense wet		10	SS	43		8	248	0	
248.4										
8.1	**End of Borehole** Dynamic Cone Penetration Test (DCPT) was conducted below 8.1 m depth.						9			
247.4										
9.0	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 9.0 m depth Groundwater in hollow stem augers observed at 0.3 m above ground									DCPT = 100 blows/2" at 9.0 m depth

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)

SHEAR STRENGTH kPa (20 40 60 80 100)
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%) (10 20 30)

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 11

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 19+150 Offset: 14.7 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Hollow Stem Augering COMPILED BY SN

DATUM Geodetic DATE 14 January 2009 - 14 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT / WATER CONTENT (%)	SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		m	m	○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE; 20 40 60 80 100	w_P w w_L; 10 20 30	PPM	GR SA SI CL
	surface (artesian) during drilling. Borehole was backfilled with bentonite at the completion of drilling. After backfilling, no groundwater seepage to the ground surface was visible.											

$+^3$, $\times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 12

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 19+200 Offset: 17.2 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 14 January 2009 - 15 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.7										
0.0	brown **SILTY SAND** some organics frozen		1	SS	35	▽				
257.0								257		
0.6	grey **CLAYEY SILT / SILTY CLAY** very soft to firm moist		2	SS	4		1			No recovery
			3	SS	3			256		
							2			
	trace sand		4	SS	0			255		
			5	SS	0		3			
			6	SS	0			254		No recovery
			7	SS	0		4	253		
			8	SS	0		5	252		
	some sand		9	SS	0		6			
250.7								251		
7.0	grey becoming reddish brown **SAND** trace gravel and silt compact wet		10	SS	23		7	250		Hard augering
							8	249		
248.1			11	SS	26		9			9 88 3 0
9.6	**End of Borehole** Dynamic Cone Penetration Test							248		DCPT = 100

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)

SHEAR STRENGTH kPa (20 40 60 80 100)
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%) (10 20 30)

+ 5

17 44 39

16 27 35

17 29 33

24

14

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No BH 12

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 19+200 Offset: 17.2 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Hollow Stem Augering and Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 14 January 2009 - 15 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa / WATER CONTENT (%)	SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		m	m	20 40 60 80 100; ○ UNCONFINED + FIELD VANE; ● QUICK TRIAXIAL × LAB VANE; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; 10 20 30	PPM	GR SA SI CL
247.4	(DCPT) was conducted below 9.6 m depth.										blows/2" at 10.2 m depth
10.3	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 10.3 m depth Groundwater in open borehole on completion: 0.6 m Borehole was backfilled with bentonite at the completion of drilling.							247			

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No DCPT 1

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 19+176 Offset: 18.8 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 13 January 2009 - 13 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	DEPTH (m)	ELEVATION SCALE (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.4												
0.0	Dynamic Cone Penetration Test (DCPT) was conducted below ground surface.						0	257				
							1	256				
							2	255				
							3	254				
							4	253				
							5	252				
							6	251				
							7	250				
							8	249				
							9	248				

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No DCPT 1

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 19+176 Offset: 18.8 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 13 January 2009 - 13 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT / WATER CONTENT (%)	SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		m	m	○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE; 20 40 60 80 100	w_P w w_L; 10 20 30	PPM	GR SA SI CL
								247				DCPT = 100 blows/1" at 11.0 m depth
246.4												
11.0	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 11.0 m depth						11					

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No DCPT 2

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 19+129 Offset: 15.9 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 13 January 2009 - 13 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	DEPTH (m)	ELEVATION SCALE (m)	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.5 / 0.0	Dynamic Cone Penetration Test (DCPT) was conducted below ground surface.									DCPT = 50

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No DCPT 2

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 19+129 Offset: 15.9 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 13 January 2009 - 13 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

SOIL PROFILE			SAMPLES										
ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%) 10 20 30	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
246.3												blows/3" at 10.1 m depth
10.1	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 10.1 m depth							246				

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No DCPT 3

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 19+070 Offset: 16.4 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 16 January 2009 - 16 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	DEPTH (m)	ELEVATION SCALE (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.2									20 40 60 80 100	10 20 30		
0.0	Dynamic Cone Penetration Test (DCPT) was conducted below ground surface.						0	256				
							1	255				
							2	254				
							3	253				
							4	252				
							5	251				
							6	250				
							7	249				
							8	248				
							9	247				

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No DCPT 3

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 19+070 Offset: 16.4 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 16 January 2009 - 16 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT / WATER CONTENT (%)	SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		m	m	20 40 60 80 100; ○ UNCONFINED + FIELD VANE; ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100	w_P w w_L; 10 20 30	PPM	GR SA SI CL
								246				
245.1												DCPT = 100 blows/4" at 11.0 m depth
11.1	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 11.1 m depth						11					

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No DCPT 4

G.W.P. 6057-07-00 | LOCATION HWY 11/17 - Station: 19+033 Offset: 17.7 Lt | ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 | BOREHOLE TYPE Dynamic Cone Penetration | COMPILED BY SN

DATUM Geodetic | DATE 16 January 2009 - 16 January 2009 | CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon | JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	DEPTH (m)	ELEVATION SCALE (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.2												
0.0	Dynamic Cone Penetration Test (DCPT) was conducted below ground surface.						0	256				
							1	255				
							2	254				
							3	253				
							4	252				
							5	251				
							6	250				
							7	249				
248.6												DCPT = 100 blows/0" at 7.6 m depth
7.6	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 7.6 m depth											

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: 20 40 60 80 100

WATER CONTENT (%): 10 20 30

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No DCPT 5

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 18+999 Offset: 20.0 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 17 January 2009 - 17 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	DEPTH (m)	ELEVATION SCALE (m)	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.6 / 0.0	Dynamic Cone Penetration Test (DCPT) was conducted below ground surface.									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

ELEVATION SCALE (m): 256, 255, 254, 253, 252, 251, 250, 249, 248, 247

DEPTH (m): 1, 2, 3, 4, 5, 6, 7, 8, 9

DCPT = 100

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No DCPT 5

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 18+999 Offset: 20.0 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 17 January 2009 - 17 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	DEPTH (m)	ELEVATION SCALE (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
246.5												blows/4" at 10.1 m depth
10.2	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 10.2 m depth							246				

+ ^{3}, × ^{3}: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No DCPT 6

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 18+963 Offset: 20.6 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 17 January 2009 - 17 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	DEPTH (m)	ELEVATION SCALE (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	SOIL VAPOUR READING (PPM)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.8									20 40 60 80 100	10 20 30		
0.0	Dynamic Cone Penetration Test (DCPT) was conducted below ground surface.						0	256				
							1					
								255				
							2					
								254				
							3					
								253				
							4					
								252				
							5					
								251				
							6					
								250				
							7					
								249				
							8					
								248				
							9					
								247				
246.9												DCPT = 135 blows/8" at 9.8 m depth

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No DCPT 6

G.W.P. 6057-07-00 LOCATION HWY 11/17 - Station: 18+963 Offset: 20.6 Lt ORIGINATED BY SAL

DIST Thunder Bay HWY 11/17 BOREHOLE TYPE Dynamic Cone Penetration COMPILED BY SN

DATUM Geodetic DATE 17 January 2009 - 17 January 2009 CHECKED BY PB

PROJECT Highway 11/17, Proposed Westbound Passing Lane at Golf Course Road, Township of Nipigon JOB NO. TT93000

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT / WATER CONTENT (%)	SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		m	m	20 40 60 80 100; ○ UNCONFINED + FIELD VANE; ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100	w_P w w_L; 10 20 30	PPM	GR SA SI CL
10.0	**End of DCPT** Refusal to Dynamic Cone Penetration Test at 10.0 m depth											

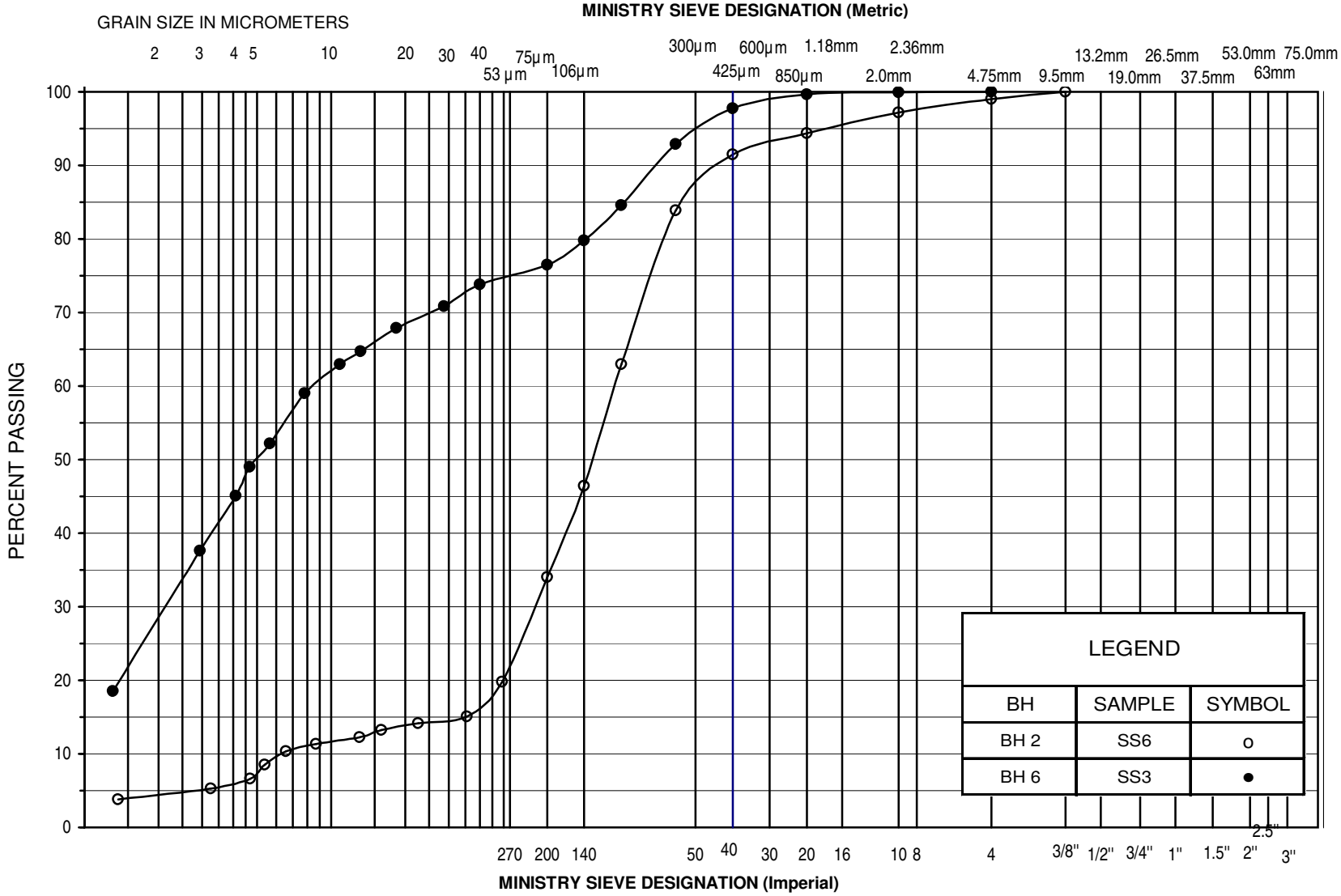
$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

APPENDIX B

LABORATORY TEST RESULTS

UNIFIED SOIL CLASSIFICATION SYSTEM

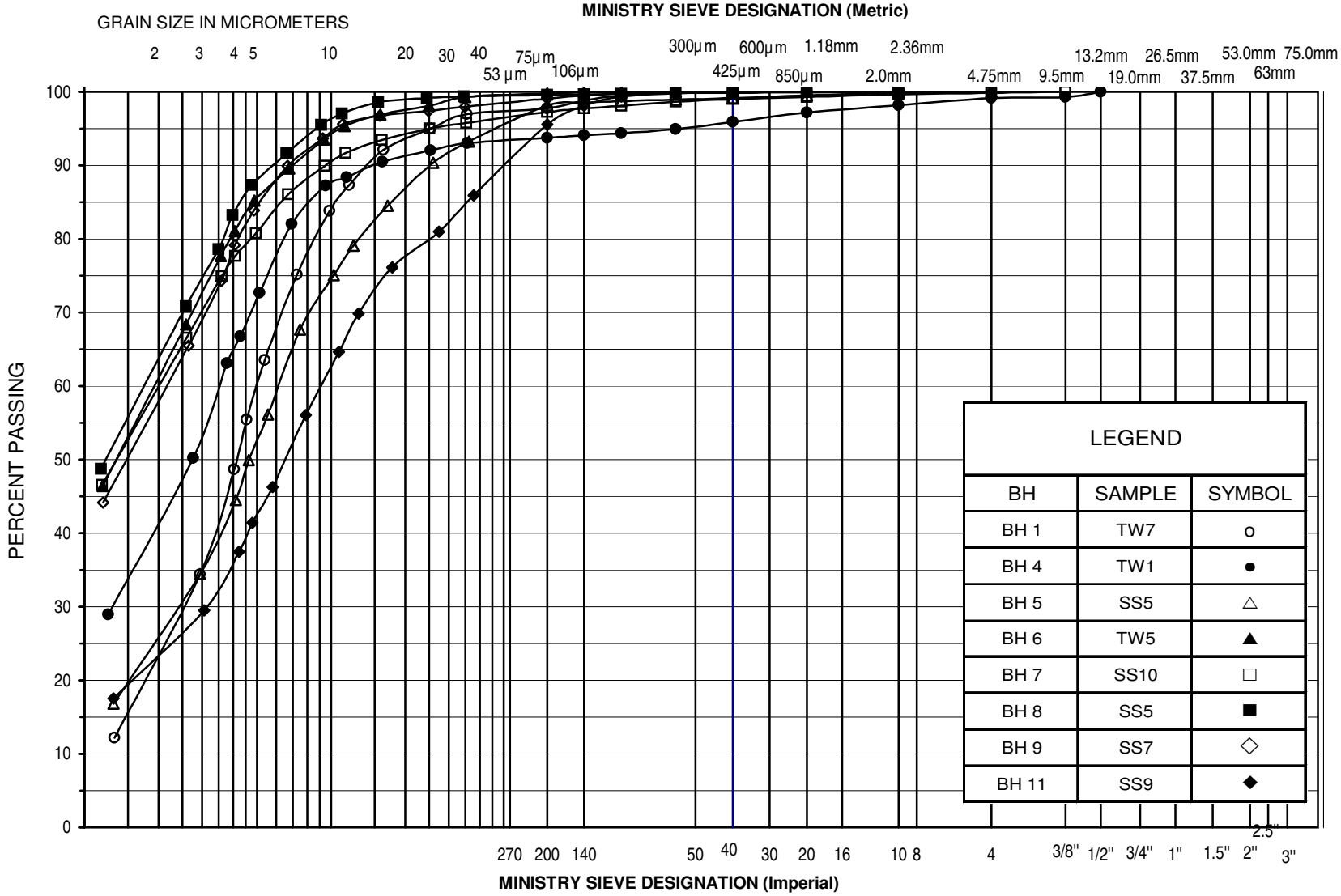
CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



Ontario	GRAIN SIZE DISTRIBUTION SANDY SILT / SILTY SAND trace to some clay, trace gravel	FIG. NO. B1 CA # 6008-E-0027 Hwy. 11/ 17 Twp. Nipigon

UNIFIED SOIL CLASSIFICATION SYSTEM

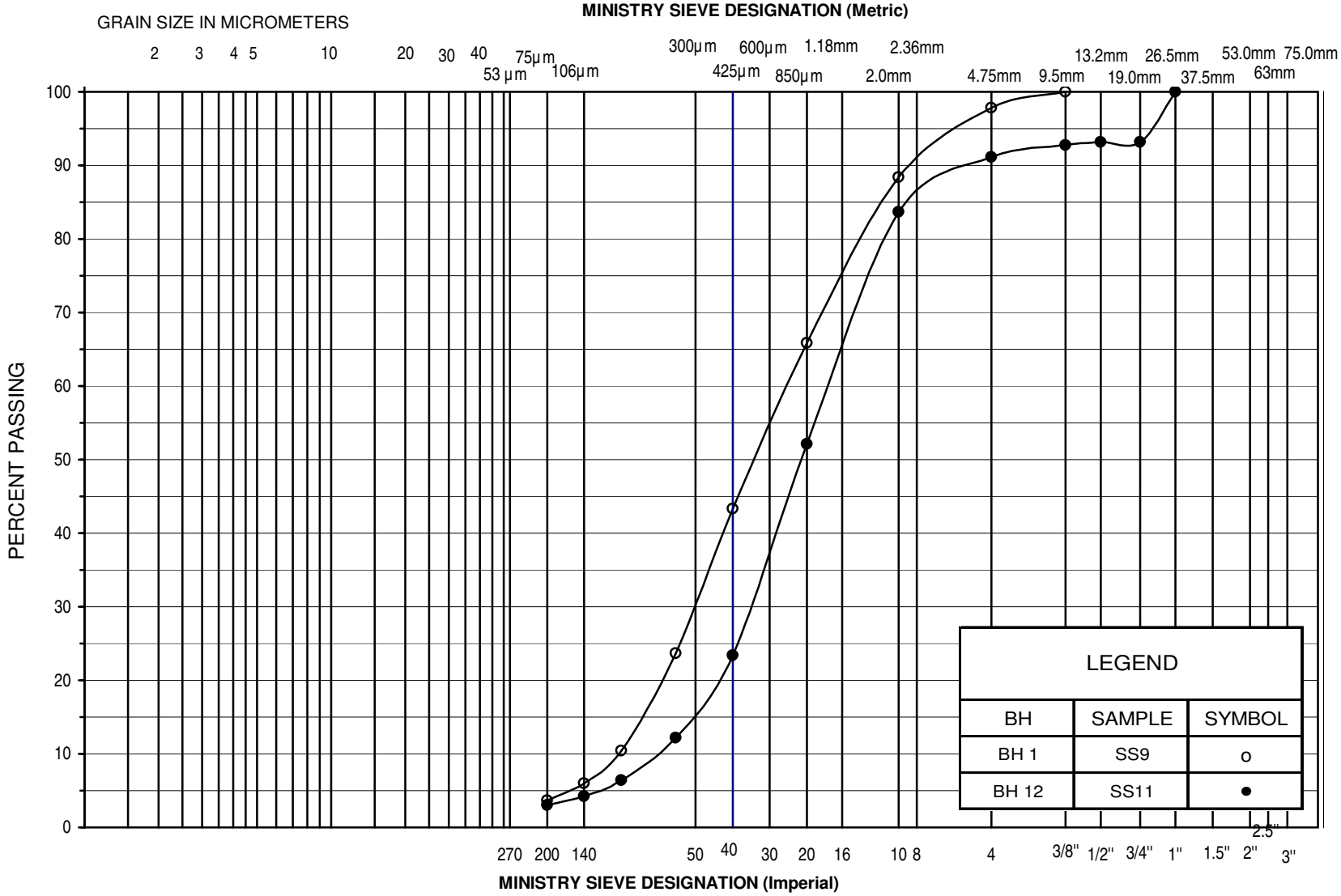
CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



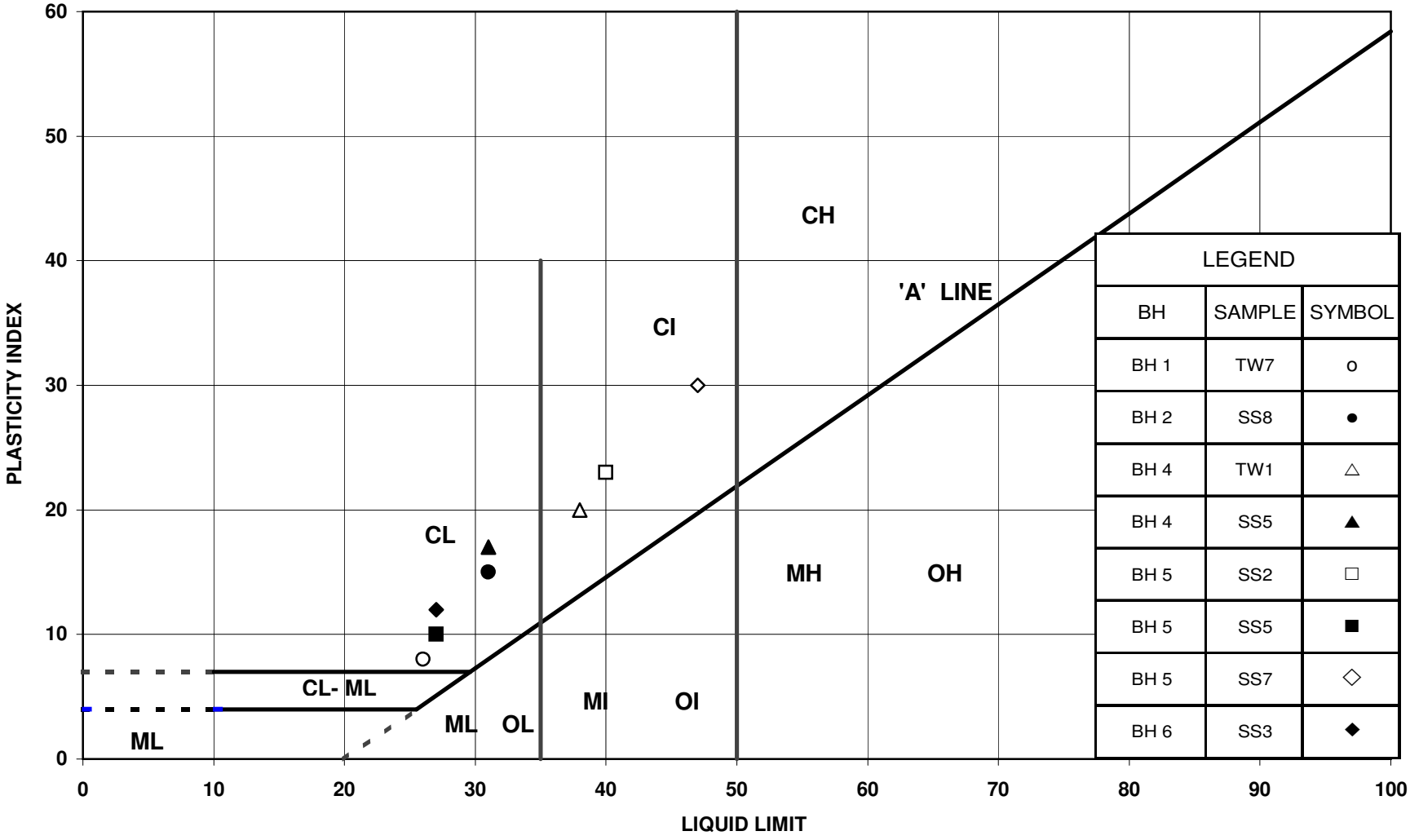
Ontario	GRAIN SIZE DISTRIBUTION CLAYEY SILT / SILTY CLAY trace sand and gravel	FIG. NO. B2 CA # 6008-E-0027 Hwy. 11/ 17 Twp. Nipigon

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



Ontario	GRAIN SIZE DISTRIBUTION SAND trace silt and gravel	FIG. NO. B3 CA # 6008-E-0027 Hwy. 11/ 17 Twp. Nipigon



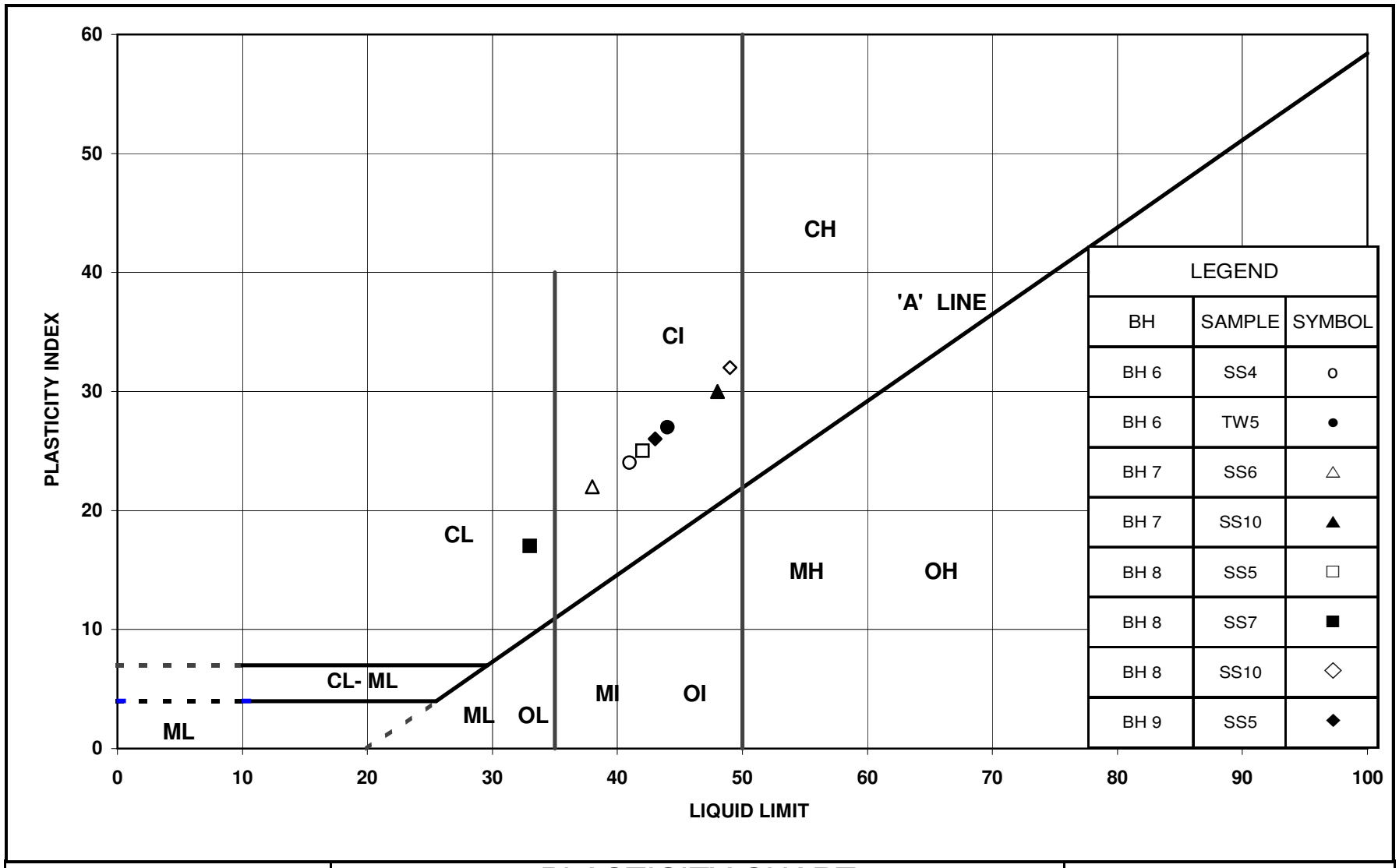
Ontario

PLASTICITY CHART

FIG No. B4

CA # 6008-E-0027

Hwy. 11/ 17 Twp. Nipigon



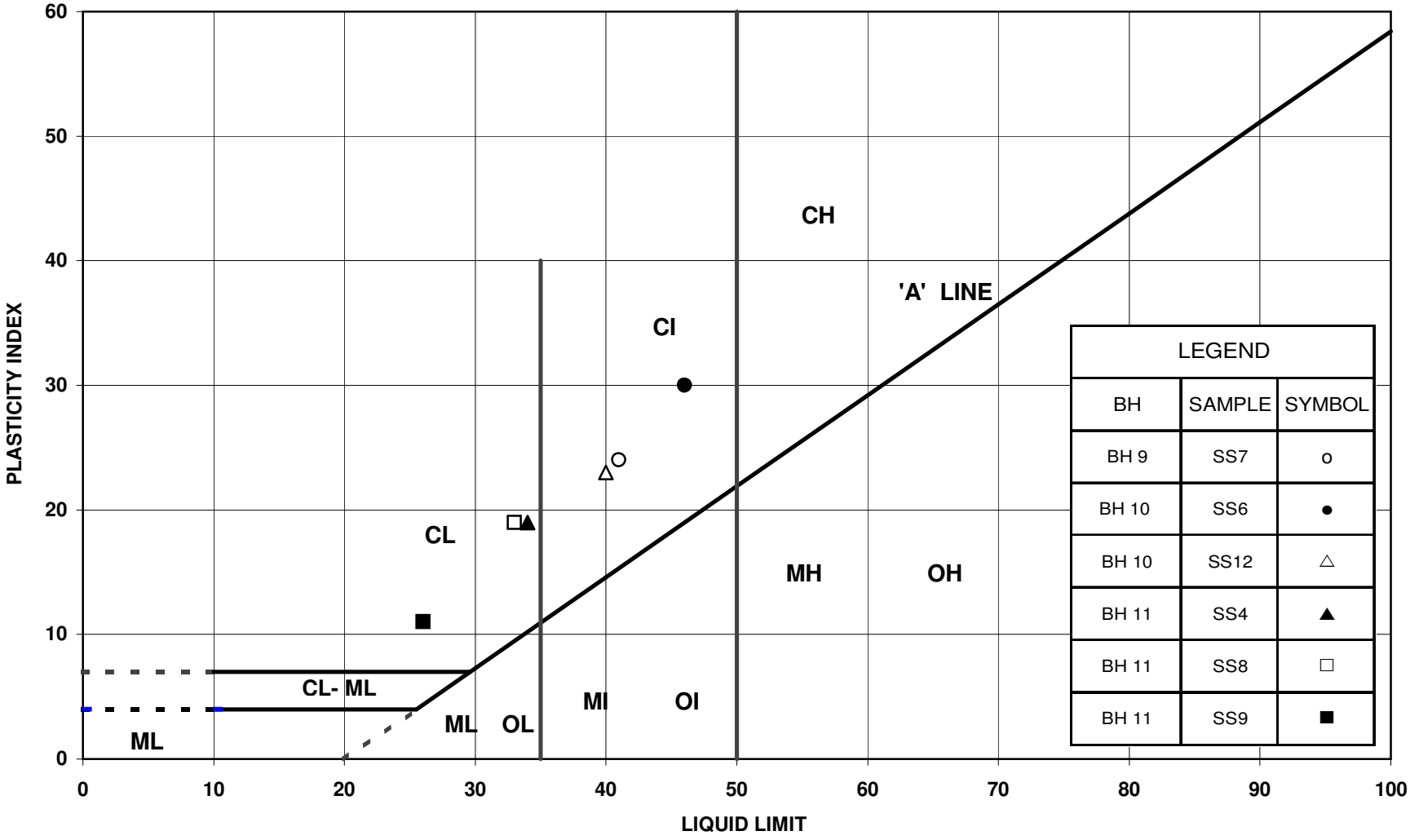
Ontario

PLASTICITY CHART

FIG No. B5

CA # 6008-E-0027

Hwy. 11/ 17 Twp. Nipigon



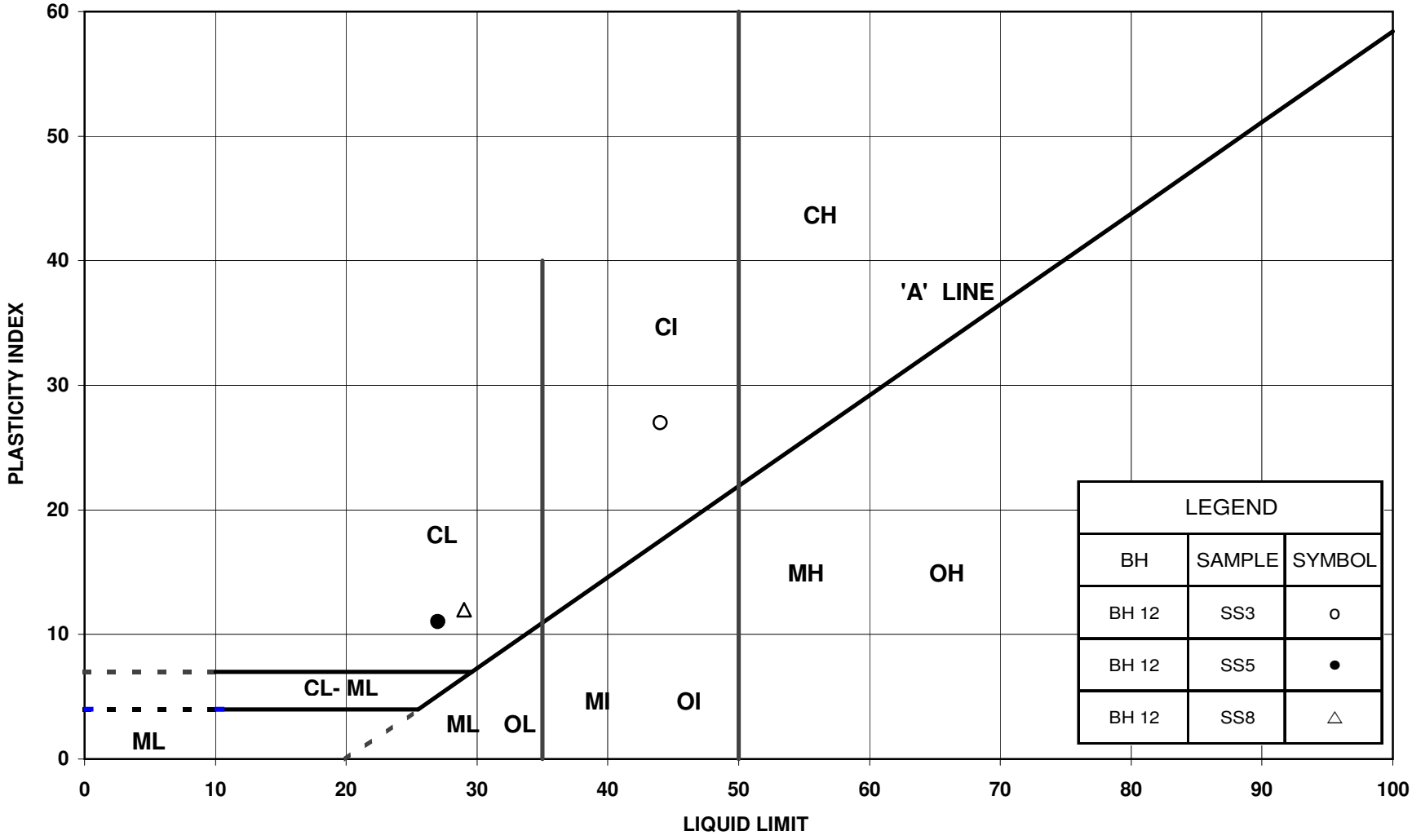
Ontario

PLASTICITY CHART

FIG No. B6

CA # 6008-E-0027

Hwy. 11/ 17 Twp. Nipigon



Ontario

PLASTICITY CHART

FIG No. B7

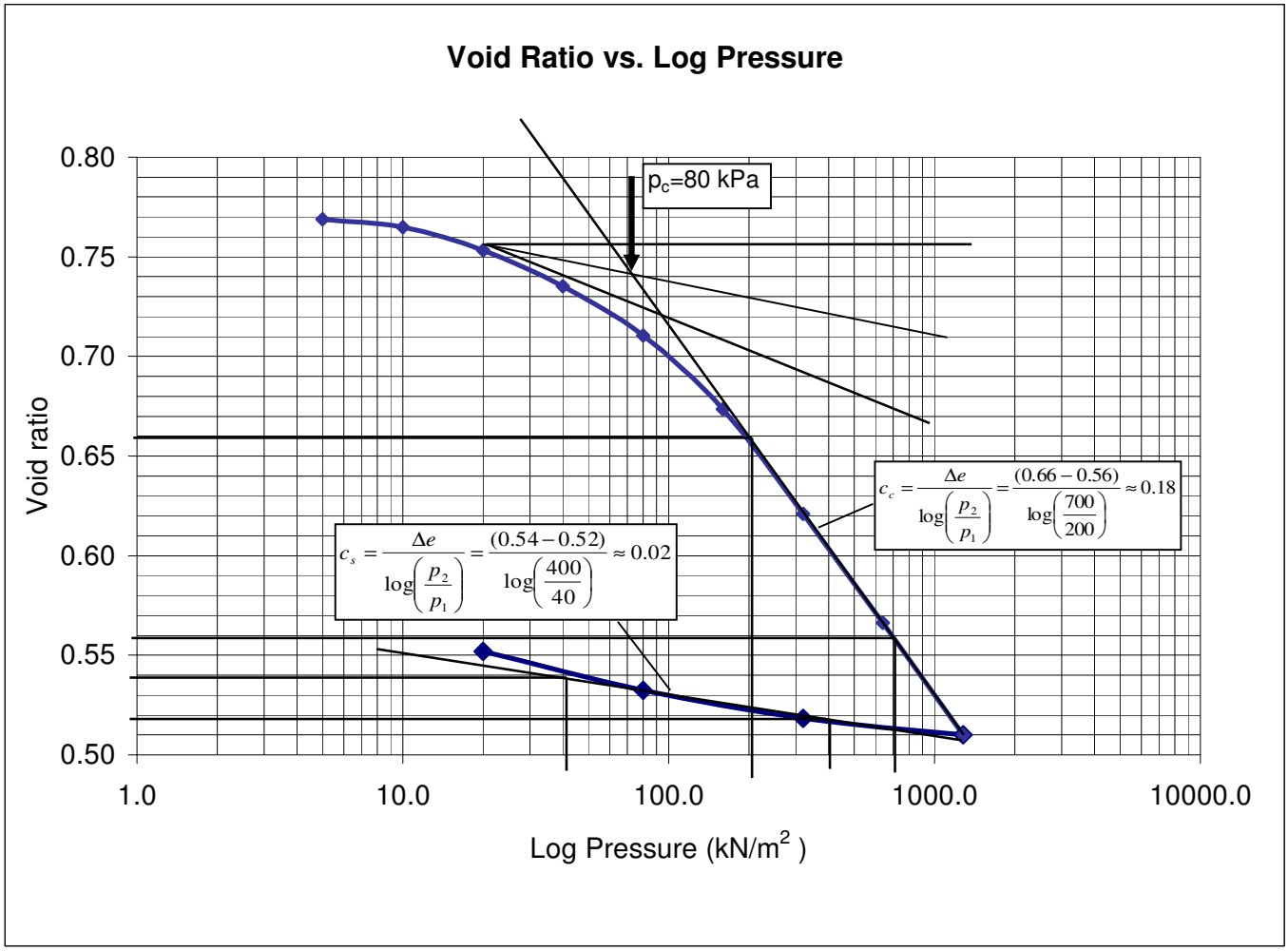
CA # 6008-E-0027

Hwy. 11/ 17 Twp. Nipigon

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: Proposed New Westbound Passing Lane Swamp Crossings **Job No.:** TT93000
Client: Ministry of Transportation
Date: 30-Jan-09 **Sample ID:** BH 1 - TW7 **Depth(m):** 4.6 - 5.2

σ'_v versus e and c_v



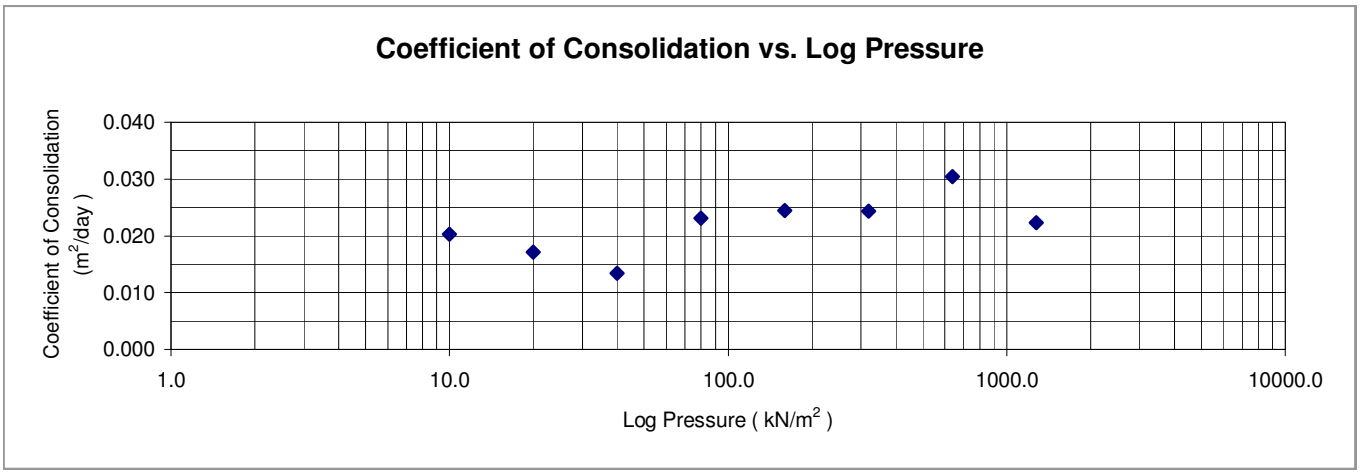
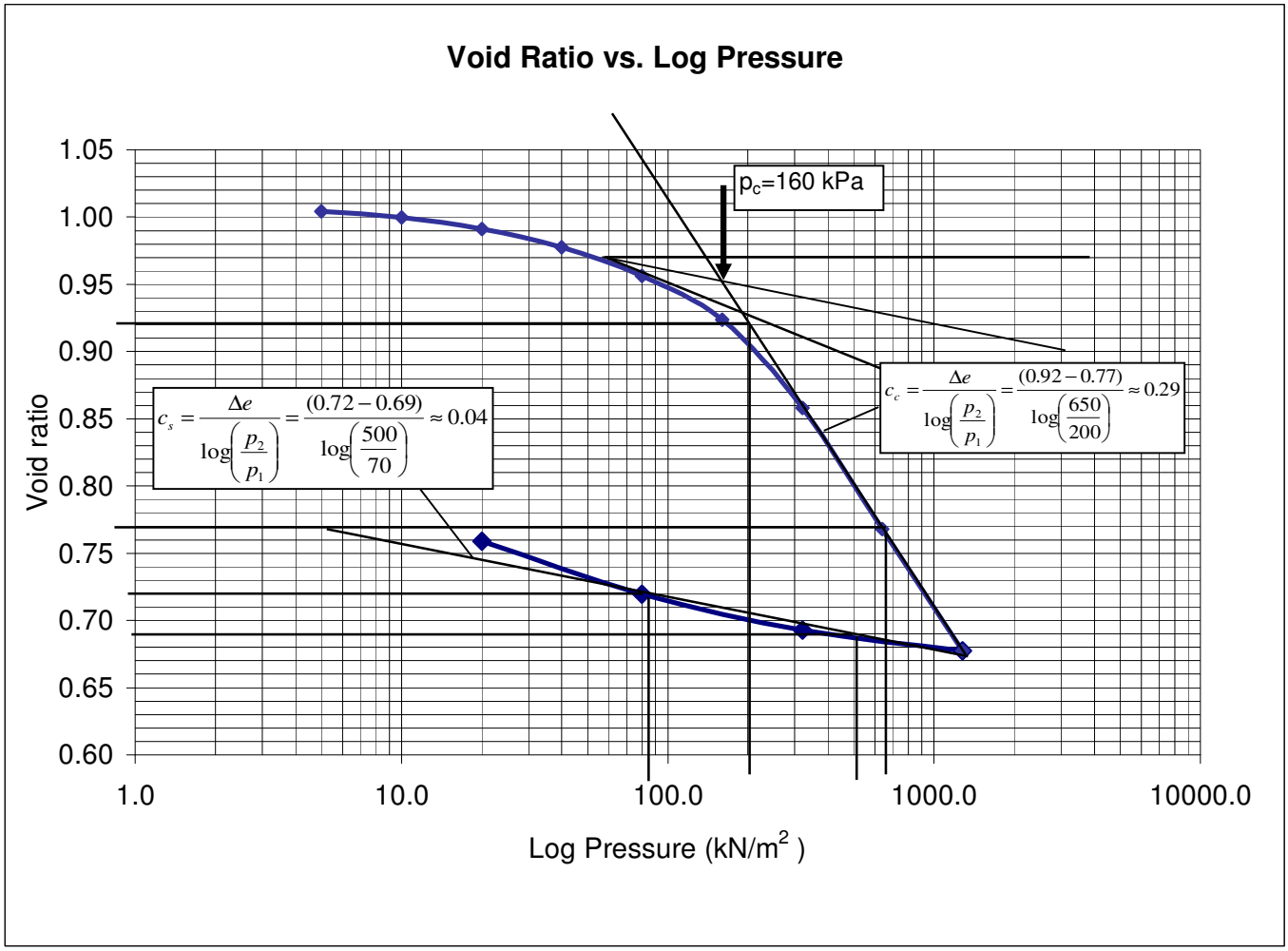


Figure B8

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: Proposed New Westbound Passing Lane Swamp Crossings **Job No.:** TT93000
Client: Ministry of Transportation
Date: 30-Jan-09 **Sample ID:** BH 6 - TW5 **Depth(m):** 3.1 - 3.7

σ'_v versus e and c_v



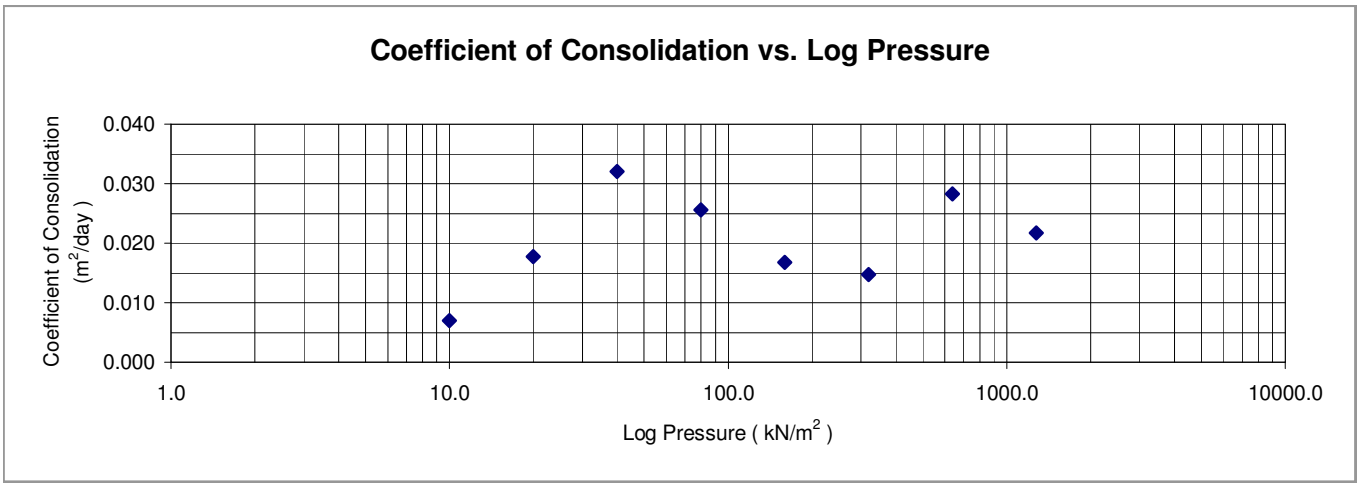
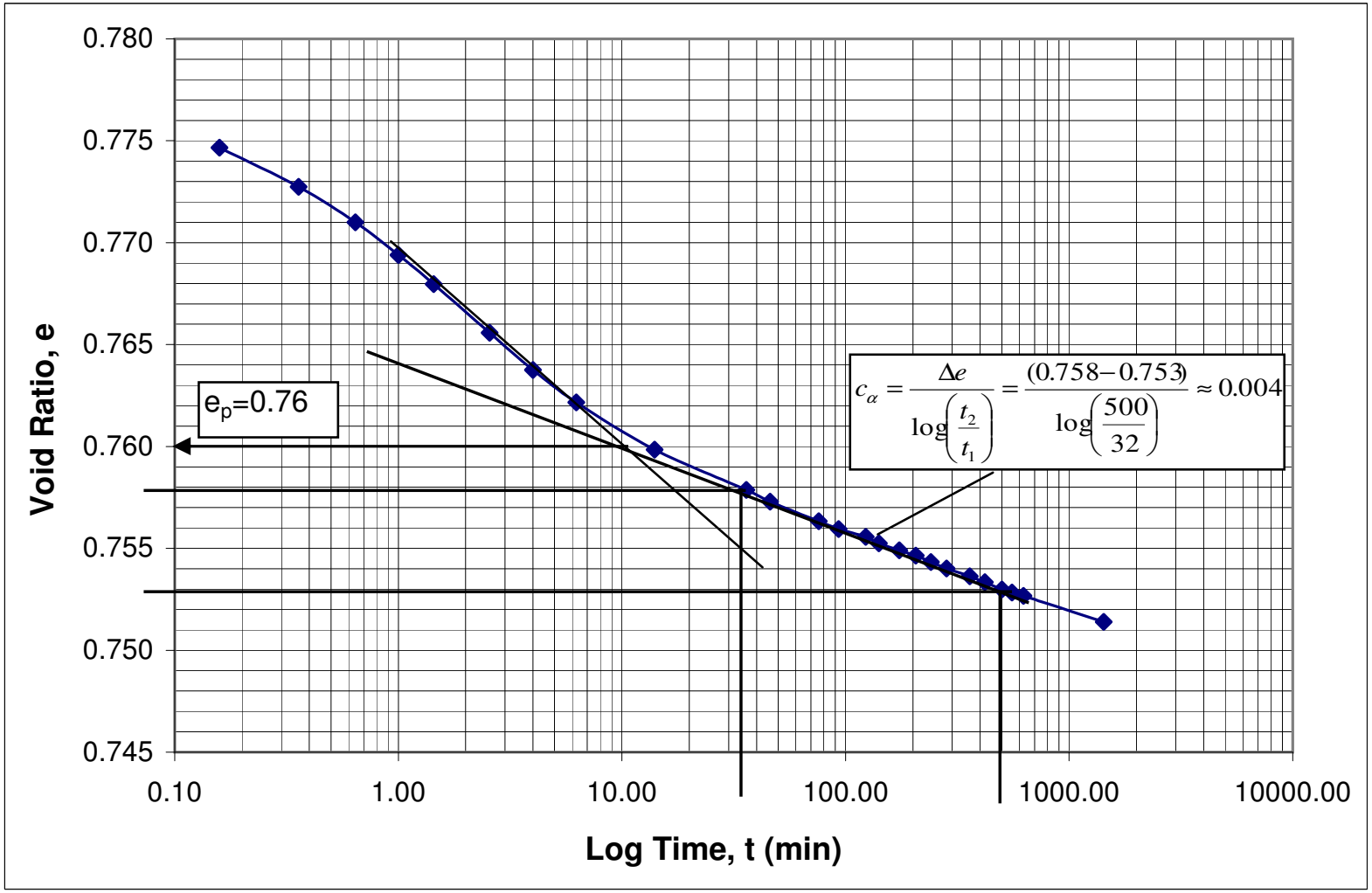
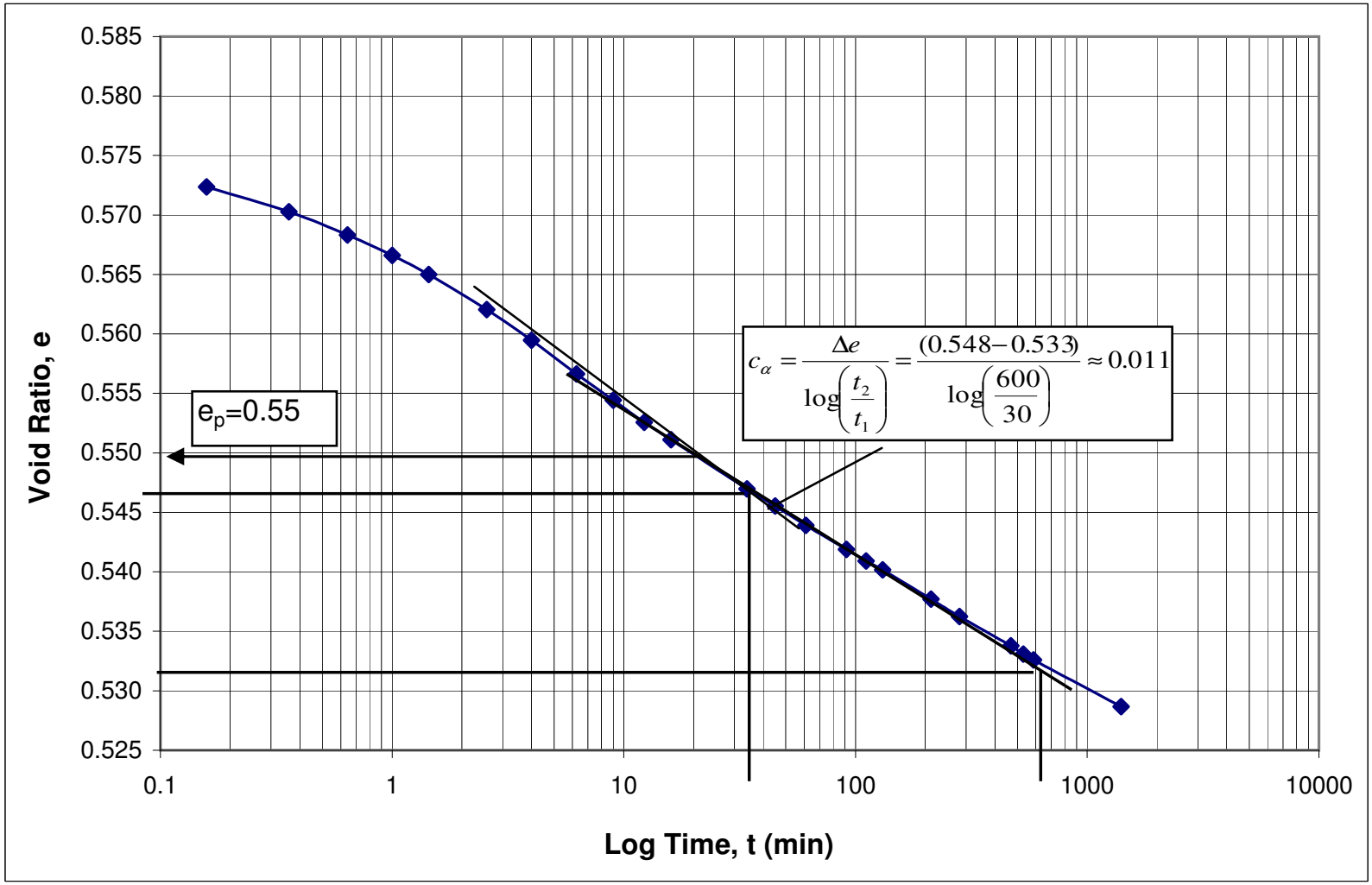


Figure B9



Ontario	**VOIDS RATIO vs LOG TIME** **Under 80 kPa Loading** **BH 6 - TW5**	FIG. NO. B10 CA # 6008-E-0027 Hwy. 11/ 17 Twp. Nipigon



Ontario	**VOIDS RATIO vs LOG TIME** **Under 160 kPa Loading** **BH 6 - TW5**	FIG. NO. B11 CA # 6008-E-0027 Hwy. 11/ 17 Twp. Nipigon

Client:	AMEC Earth and Environmental, a division of AMEC Americas Limited 104 Crockford Boulevard Scarborough, Ontario M1R 3C3	Report Date: Received Date:	February 12, 2009 February 05, 2009
		Page:	1 of 2
Project Name:	Foundation Engineering for HWY 11/17 TWP Nipigon	Sample Type:	Soil
Project Number:	TT 93000	Lab Ref.:	FN09-238
Contact:	Siva Nadarajah		Final

CERTIFICATE OF ANALYSIS

Corrosivity Package

Lab Number **Sample ID** **Date Collected**			**09-1692** **BH9 SS6** **NP**	**09-1692** **BH9 SS6** **NP**
Parameters	**Unit**	**MDL**		**(Replicate)**
Chloride	(µg/g)	1	6	6
pH	-	-	7.7	NR
Resistivity	(ohmscm)	-	4650	NR
Sulphate	(µg/g)	1	26	26

			Lab Blank (µg/g)	**Q. C. Standard Actual (mg/L)**	**Q. C. Standard Expected (mg/L)**	**Date of Analysis**
Parameters	**Unit**	**MDL**				
Chloride	(µg/g)	1	<1	3.9	4.2	09-Feb-09
pH	-	-	7.1	6.0	6.0	10-Feb-09
Resistivity	(ohmscm)	-	-	-	-	10-Feb-09
Sulphate	(µg/g)	1	<1	22.6	24.0	09-Feb-09

			Method References
Parameters	**Unit**	**MDL**	
Chloride	(µg/g)	1	MOE 3013, APHA 4110 C
pH	-	-	MOE 9045
Resistivity	(ohmscm)	-	MOE 3137
Sulphate	(µg/g)	1	MOE 3013, APHA 4110 C

Cynthia Ridge, C. Chem.
Q.A./Q.C. Officer

CHARTERED CHEMICAL PROFESSIONALS OF THE ASSOCIATION OF ONTARIO
Suman Punani
CHEMIST

Suman Punani, C. Chem.
Laboratory Manager

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Fax +1 (905) 890-1141
www.amec.com

/gz

~ GENERAL COMMENTS ~

MDL	Method Detection Limit
RDL	Reporting Detection Limit
ANR	Analysis not required
NA	Analysis not applicable
NP	Not Provided
NR	No Lab Replicate
	Result in (brackets) represents Lab Replicate.
	Results relate only to the items tested.

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APPENDIX C

SITE PHOTOGRAPHS

Photo No. 1 General view of Hwy 11/17 looking west from east end of project area (Sta.19+200).

Photo No. 2 General view of Hwy 11/17 looking east from west end of project area (Sta.18+700)

Photo No. 3 General View of embankment looking east from BH 9 (Sta. 19+100).

Photo No. 4 General View of embankment looking west from BH 8 (Sta. 19+050).

Photo No. 5 General View of north side of the culvert at Sta. 18+730, looking east. (Boreholes BH1 and BH 3). Arrows indicate direction of creek.

Photo No. 6 General View of south side of the culvert at Sta. 18+730, looking south. (Boreholes BH 2). Arrows indicate direction of creek.

Photo No. 7 General View of north side of the culvert at Sta. 19+102, looking north. (Boreholes BH 9).

Photo No. 8 General View of south side of the culvert at Sta. 19+102, looking south. (Boreholes BH 10).

Photo No. 9 General view of existing asphalt road surface, looking west from Sta. 19+200.

Photo No. 10 Typical view of existing asphalt road surface.

APPENDIX D

RESULTS OF PREVIOUS INVESTIGATIONS (1972, 2002 & 2008)

(Provided by Ministry of Transportation)

WP 6057-07-00 - Hwy 11/17, WBPL in vicinity of Golf Course Rd near Nipigon

Township Red Rock

Station 18+718 16.0 Rt D-2.4

0	-	100	Tps
100	-	900	Br F Sa with Si Tr Gr Occ Cob & Bld
		900	NFP Bld *

Station 18+720 18.0 Lt D-4.5

0	-	1.6	Blk Org
		1.6	NFP RF *

Station 18+727 18.0 Lt D-4.5

0	-	2.0	Wat
2.0	-	3.9	Blk Org (soft)
3.9	-	6.8	Br Sa(y) Si (firm with soft layers)
6.8	-	7.7	Gry Si(y) Cl (wet) (firm)
7.7	-	9.0	Gry Si(y) Cl Occ Cob (wet) (firm)
		9.0	NFP Bld Poss BR *

Station 18+732 17.0 Rt D-3.6

0	-	300	Wat
300	-	700	Br F Sa with Sa Gr & Tps mixed (loose)
700	-	2.1	Blk Org (wet) (soft)
2.1	-	3.1	Gry Si(y) Cl (wet) (soft)
3.1	-	3.6	Gry Sa(y) Si (wet) (comp)

Station 18+746 17.0 Rt D-2.5

0	-	100	Tps
100	-	700	RF
700	-	900	Br F Sa with Si Tr Gr Occ Cob & Bld
		900	NFP Bld *

Station 18+747 18.0 Lt D-4.6

0	-	100	Wat
100	-	3.0	Blk Org
3.0	-	3.9	Gry Si(y) Cl (moist) (stiff)
3.9	-	4.5	Gry Sa(y) Si (wet) (comp)
4.5	-	4.6	Gry Si(y) Sa with Gr (comp)
		4.6	NFP Bld Poss BR *

Station 18+765 15.0 Lt D-1.7

0			BR on Surf (bottom of vertical face)

Station 18+825 12.0 Rt

0	-	60	Tps
60	-	3.4	Br F - M Sa some Gr Tr Si Occ Bld (wet @ 2.0)

Station 18+825 12.0 Rt @ 1.0 m

% Passing 4.75 mm	73.6%
% Passing 75 µm	9.7%
FMC	6.2%
LSFH	
GS	SW-SM

Not Acceptable for Granular 'B'

Station 18+825 12.0 Rt @ 2.0 m

% Passing 4.75 mm	85.0%
% Passing 75 µm	15.2%
FMC	15.7%
LSFH	
GS	SM

Not Acceptable for Granular 'B'

Station 18+825 12.0 Rt @ 3.0 m

% Passing 4.75 mm	78.9%
% Passing 75 µm	13.2%
FMC	15.8%
LSFH	
GS	SM

Not Acceptable for Granular 'B'

Station 18+825 37.0 Lt D-6.0

0	-	300	Tps (wat @ surf) (soft)
300	-	1.5	Br Si(y) Cl (moist) (stiff)

Station 18+850 12.5 Lt D-3.0

0	-	600	Tps
600	-	300	Br F - M Sa some Gr Tr Si Occ Bld
		300	NFP RF *

Station 18+875 12.0 Lt

0	-	60	Tps
60	-	1.7	Br F - M Sa some Gr Tr Si Occ Bld
		1.7	NFP Prob RF *

Station 18+875 17.0 Lt D-3.0

0	-	60	Tps
60	-	500	Br F - M Sa some Gr Tr Si Occ Cob
		500	NFP RF *

WP 6057-07-00 - Hwy 11/17, WBPL in vicinity of Golf Course Rd near Nipigon

Township Red Rock

Station 18+875 19.0 Lt D-3.5
0 - 60 Tps
60 - 1.5 Br F - M Sa some Gr Tr Si Occ Bld
1.5 NFP RF *
Station 18+875 19.0 Lt @ 1.0 m
% Passing 4.75 mm 59.7%
% Passing 75 µm 5.6%
FMC 5.2%
LSFH
GS SP-SM
Acceptable for Granular 'B'

Station 18+875 33.0 Lt D-5.1
0 - 1.1 Blk Org (wat @ surf) (soft)
1.1 - 2.0 Br Si(y) Cl (moist) (stiff)

Station 18+900 14.0 Lt D-2.8
0 - 160 Tps
160 - 1.0 Br F - M Sa some Gr Tr Si Occ Bld
1.0 NFP RF *
Station 18+900 33.0 Lt @ 1.0 m
% Passing 4.75 mm 84.4%
% Passing 75 µm 5.9%
FMC 5.8%
LSFH
GS SW-SM
Acceptable for Granular 'B'

Station 18+925 11.0 Lt
0 - 60 Tps
60 - 750 Br F - M Sa some Gr Tr Si Occ Bld
750 NFP RF *

Station 18+935 20 Lt
East End of RF Berm

Station 18+950 11.0 Lt D-2.8
0 - 60 Tps
60 - 500 Br F - M Sa some Gr Tr Si Occ Bld
500 NFP RF *

Station 18+950 21.0 Lt D-4.5
0 - 1.5 Blk Org (wat @ surf) (soft)

Station 18+975 20.0 Lt D-3.7
0 - 1.1 Blk Org (standing wat on surf)
1.1 - 4.1 Gry Si(y) Cl (soft & wet) (wet @ firm @ 1.9) (moist @ stiff @ 3.1)

Station 19+000 16.0 Lt D-2.9
0 - 900 Blk Org (standing wat on surf)
900 - 2.5 Gry Si(y) Cl (soft) (wet) (firm @ 1.5) (stiff @ 2.3)
2.5 - 2.8 Gry Sa(y) Si (wet) (comp)

Station 19+025 16.0 Lt D-2.8
0 - 1.0 Blk Org (standing wat on surf)
1.0 - 2.1 Gry Si(y) Cl (wet) (soft) (moist @ stiff @ 1.3)

Station 19+050 19.0 Lt D-2.7
0 - 300 Wat
300 - 600 Blk Org (wet) (soft)
600 - 2.0 Gry Si(y) Cl (moist) (stiff)

Station 19+075 18.0 Lt D-2.7
0 - 700 Wat
700 - 1.3 Blk Org (soft) (wet)
1.3 - 2.3 Gry Si(y) Cl (wet) (soft) (firm @ 1.4)

Station 19+100 18.0 Lt D-2.6
0 - 50 Wat
50 - 200 Tps & Si(y) Sa
200 - 1.3 Br F Sa with Si & comp layers of Si (wet) (loose)
1.3 - 3.2 Gry Si(y) Cl (moist) (firm) (soft @ 2.0) (stiff @ 2.2)

Station 19+125 19.0 Lt D-2.3
0 - 150 Tps (wat @ surf)
150 - 1.3 Br F Sa with Si Occ Gr (wet) (comp)
1.3 - 1.7 Gry Si(y) Sa Tr Cl (wet) (firm)
1.7 - 2.0 Br Si(y) Cl (moist) (stiff)

Station 19+150 19.0 Lt D-2.1
0 - 50 Wat
50 - 150 Tps & Sa with Si
150 - 1.2 Br F Sa with Si Occ Gr (wet) (comp)
1.2 - 1.6 Br Si(y) Sa Occ Gr
1.6 NFP Cob *

WP 6057-07-00 - Hwy 11/17, WBPL in vicinity of Golf Course Rd near Nipigon

Township Red Rock

Station 19+175 15.0 Lt D-2.1

0	-	150	Tps & Sa
150	-	400	Br F Sa with Si Occ Gr (wet) (comp)
400	-	550	Br Si(y) Cl Tr Sa mixed (moist) (stiff)
550	-	1.5	Br F Sa with Si Occ Gr (wet) (comp)

Station 19+175 18 Lt D-2.1

0	-	50	Wat
50	-	250	Org Tps (soft) (wet)
250	-	2.0	Br F Sa with Si Occ Gr (wet) (comp)

Station 19+200 16.0 Lt D-2.7

0	-	150	Tps & Sa
150	-	900	Br F Sa with Si Tr Gr Cob & Bld (wet)
900	-	1.1	Blk Org (wet) (soft)
1.1	-	2.0	Br Si(y) Cl (moist) (stiff)

Station 19+225 18.0 Lt D-2.6

0	-	100	Tps
100	-	800	Br F Sa with Si Tr Gr Cob & Bld (wet) (wat @ 200)
800	-	1.5	Br Si(y) Cl (moist) (stiff)

* 3 Attempts made within 1 m, no further penetration possible.

HIGHWAY 11 & 17
TWP. NIPIGON

Station 18+700 12.0 Rt (D-1.6)
0 - 100 Tps
100 - 400 Br F-Co Sa (Moist)
400 NFP Sh Rk

Station 18+702 17.0 Lt (D-2.7)
0 - 75 Tps
75 - 700 Br F-Co Sa Occ Gr with Blds & Sh Rk
700 NFP Sh Rk

Station 18+729 4.2 Lt (Culv)
0 - 30 CM
30 - 300 Cr Gr
300 - 900 Br F-Co Sa Tr Si Occ Gr (Moist)
900 - 1.2 Rk Fill
1.2 NFP Rk Fill

Station 18+732 23.0 Lt (D-4.5) (Culv)
0 - 7.7 Blk Orgs (F-Co Fib) (Stiff from 2.0-2.5) (Soft from 2.5-6.0) (Firm from 6.0-7.7)
7.7 - 8.0 Gry Si(y) Cl (Stiff)

Station 18+732 5.7 Rt (Culv)
0 - 50 Asph
50 - 350 Cr Gr
350 - 800 Br F-Co Sa Tr Si Occ Gr (Moist)
800 - 1.2 Rk Fill
1.2 NFP Rk Fill

Station 18+732 15.0 Rt (D-4.1) (Culv)
0 - 500 Wat
500 - 1.9 Blk Orgs (F-Co Fib) (Wet)

Station 18+750 15.0 Lt (D-3.5)
0 - 1.5 Blk Orgs (Co Fib) (Wet & Soft)

Station 18+750 13.0 Rt (D-1.6)
0 - 300 Ob to Sh Rk

Station 18+760 17.0 Lt (D-3.3)
0 - 1.5 Br Orgs (Co Fib) (Wet & Soft)
1.5 NFP Prob Frag BR

Station 18+770 15.0 Lt (D+600)
0 - 100 Ob to BR

Station 18+774 10.0 Rt (D-1.0)
Frag BR on Surf

Station 18+780 13.0 Lt (D+1.7)
BR on Surf

Station 18+788 11.0 Lt (D-1.1)
0 - 200 Ob to Sh Rk

Station 18+790 15.0 Lt (D+1.3)
0 - 200 Ob to BR

Station 18+800 15.0 Lt (D-1.1)
0 - 100 Ob to BR

Station 18+810 10.0 Rt (D-800)
BR on Surf

Station 18+820 15.0 Lt (D-1.7)
0 - 100 Tps
100 - 400 Br F-Co Sa Occ Gr
400 NFP BR

Station 18+825 15.0 Lt (D-2.3)
0 - 200 Ob Tr Rk Fill (Old Rd Bed)

Station 18+825 14.0 Rt (D-1.4)
0 - 10 Tps
10 - 500 Br F-Co Sa Occ Bld (Moist)
500 NFP Bld

Station 18+855 12.0 Lt (D+- 0.0)
0 - 25 Tps
25 - 1.3 Br F-Co Sa Occ F Gr (Moist)
1.3 NFP Poss BR

Station 18+860 16.0 Lt (D-2.8)
0 - 300 Ob to Sh Rk Fill

Station 18+860 16.0 Rt (D-2.7)
0 - 100 Tps
100 - 600 Br F-Co Sa with Blds & Sh Rk
600 NFP Blds

Station 18+865 13.0 Lt (D-300)

0	-	900	Br F-Co Sa Tr Si & F Gr (Moist)
		900	NFP Poss BR

Station 18+870 4.2 Lt

0	-	50	Asph
50	-	310	Cr Gr
310	-	850	Br F-Co Sa Tr Si & Gr (Moist)
850	-	1.2	Rk Fill
		1.2	NFP Rk Fill

Station 18+875 12.0 Lt (D-300)

0	-	25	Tps
25	-	1.4	Br F-Co Sa Tr F Gr (Moist)
		1.4	NFP Poss BR

Station 18+890 16.0 Lt (D-2.6)

0	-	100	Tps
100	-	500	Br F-Co Sa
		500	NFP Sh Rk

Station 18+900 12.0 Lt (D-1.2)

0	-	75	Tps (Moist)
75	-	300	Br F-Co Sa Tr F Gr (Wet)
300	-	600	Br VF-F Sa Tr Si (Wet) (Fr Wat @ 600)
600	-	1.2	Br VF-F Sa with Si (Wet)

Station 18+915 4.6 Rt

0	-	120	Asph
120	-	350	Cr Gr
350	-	900	Br F-Co Sa Tr Gr & Si (Moist)
900	-	1.2	Rk Fill
		1.2	NFP Rk Fill

Station 18+920 17.0 Rt (D-2.5)

0	-	75	Tps
75	-	700	Br F-Co Sa with Blds & Sh Rk
		700	NFP Blds

Station 18+930 16.0 Lt (D-2.5)

0	-	100	Tps
100	-	600	Br F-Co Sa with Blds & Sh Rk
		600	NFP Sh Rk

Station 18+958 18.0 Rt (D-2.4)

0	-	300	Ob to Sh Rk Fill

Station 18+960 7.0 Lt

0	-	60	Asph
60	-	410	Cr Gr
410	-	900	Br F-Co Sa Tr Gr & Si (Moist)
900	-	1.5	Rk Fill & Sa Mixed (Moist)
		1.5	NFP Rk Fill

Station 18+960 18.0 Lt (D-4.1)

0	-	400	Wat
400	-	900	Br Orgs (Co Fib) (Wet & Soft)
900	-	1.3	Lt Br Si(y) Cl Tr Sa (Wet & Firm)

Station 18+990 17.0 Lt (D-3.3)

0	-	200	Wat
200	-	1.4	Br Orgs (Co Fib) (Wet & Soft)
1.4	-	1.6	Lt Br Si(y) Cl Tr Sa (Wet & Firm)

Station 19+000 18.0 Rt (D-4.4)

0	-	700	Br Orgs (Co Fib) (Soft & Wet) (Wat on Surf)
700	-	1.2	Br-Gry Si(y) Cl Tr Sa (Wet & Stiff)

Station 19+020 17.0 Lt (D-3.1)

0	-	200	Wat
200	-	1.3	Br Orgs (F-Co Fib) (Wet & Soft)
1.3	-	1.6	Lt Br Si(y) Cl Tr Sa (Wet & Firm)

Station 19+030 18.0 Rt (D-4.0)

0	-	500	Br Orgs (Co Fib) (Soft & Wet) (Wat on Surf)
500	-	1.2	Br-Gry Si(y) Cl Tr Sa (Wet & Stiff)

Station 19+050 17.0 Lt (D-3.0)

0	-	1.2	Br Orgs (F-Co Fib) (Wet & Firm) (Fr Wat on Surf)
1.2	-	1.5	Lt Br Si(y) Cl Tr Sa (Wet & Firm)

Station 19+060 18.0 Rt (D-3.4)

0	-	200	Br F Sa Tr Cl & Si (Wat on Surf)
200	-	800	Br F-Co Sa (Wet)
800	-	1.2	Br Si(y) Cl with Sa (Wet & Stiff)

Station 19+080 5.4 Lt

0	-	40	CM
40	-	410	Cr Gr
410	-	1.2	Br F-M Sa Tr Si Occ Gr & Cobs (Moist)
1.2	-	1.5	Rk Fill & Sa Mixed (Moist) (Si(y) Sa Seam @ 2.0)
1.5	-	3.2	Br F-Co Sa Tr Si Occ Gr, Cobs & Blds (Moist) (Wet from 2.8)
3.2	-	3.7	Gry F-Co Sa Tr Si (Wet)

Station 19+080 17.0 Lt (D-3.1)

0	-	300	Wat
300	-	1.1	Blk Orgs (F Fib) (Wet & Firm)
1.1	-	1.5	Gry Si(y) Cl Tr Sa (Wet & Stiff)

Station 19+096 4.6 Lt (Culv's)

0	-	30	CM
30	-	250	Cr Gr
250	-	3.5	Br F-M Sa Tr Gr & Si Occ Cobs & Blds (Moist) (Wet from 2.8)
3.5	-	4.2	Gry F-Co Sa Tr Si (Wet)

Station 19+099 21.0 Lt (D-3.3) (Culv)

0	-	300	Wat
300	-	800	Br F Sa with Orgs Tr Cl & Si (Wet)
800	-	1.3	Br-Gry Si(y) Cl Occ Sa (Wet & Stiff)

Station 19+099 15.0 Rt (D-3.5) (Culv)

0	-	500	Wat
500	-	700	Br F-Co Sa Occ F Gr Tr Cl & Si (Wet)
700	-	900	Lt Br-Gry Si(y) Cl Tr Sa (Wet & Stiff)

Station 19+101 21.0 Lt (D-3.3) (Culv)

0	-	500	Wat
500	-	800	Br F Sa with Orgs Tr Cl & Si (Wet)
800	-	1.4	Br-Gry Si(y) Cl Occ Sa (Wet & Stiff)

Station 19+101 15.0 Rt (D-3.6) (Culv)

0	-	400	Wat
400	-	600	Br F-Co Sa Occ F Gr Tr Cl & Si (Wet)
600	-	900	Lt Br-Gry Si(y) Cl Tr Sa (Wet & Stiff)

Station 19+104 4.1 Rt (Culv)

0	-	40	Asph
40	-	300	Cr Gr
300	-	900	Br F-M Sa Tr Si Occ Gr (Moist)
900	-	1.2	Rk Fill
		1.2	NFP Rk Fill

Station 19+110 16.0 Lt (D-2.8)

0	-	1.4	Blk Orgs (F Fib) (Wet & Firm) (Fr Wat from 400)
1.4	-	1.9	Gry Si(y) Cl Tr Sa (Wet & Stiff)

Station 19+130 21.0 Rt (D-3.6)

0	-	1.0	Blk Orgs (F Fib) (Wat on Surf) (Wet & Soft-Firm)
1.0	-	1.5	Br-Gry Si(y) Cl Tr Sa (Wet & Stiff)

Station 19+140 16.0 Lt (D-2.7)

0	-	100	Wat
100	-	700	Blk Orgs (F Fib) (Wet & Firm)
700	-	1.2	Lt-Br-Gry Si(y) Cl (Wet & Stiff)

Station 19+143 4.6 Rt

0	-	30	Asph
30	-	230	Cr Gr
230	-	2.0	Br F-Co Sa Tr Si Occ Gr (Moist)

Station 19+160 18.0 Rt (D-4.4)

0	-	300	Wat
300	-	1.0	Blk Orgs (F Fib) with Sa (Wet & Stiff)
1.0	-	1.5	Gry Si(y) Cl Tr Sa (Wet & Stiff)

Station 19+170 16.0 Lt (D-2.6)

0	-	150	Wat
150	-	900	Blk Orgs (F-Co Fib) (Wet & Firm)
900	-	1.5	Gry Si(y) Cl (Wet & Stiff)

Station 19+180 5.1 Lt

0	-	30	CM
30	-	300	Cr Gr
300	-	1.6	Br F-M Sa Tr Si Occ Gr & Cobs (Moist)
1.6	-	1.7	Rk Fill Poss Blds
		1.7	NFP Rk Fill Poss Blds

Station 19+190 17.0 Rt (D-4.9)

0	-	800	Blk Orgs (F Fib) Tr Sa (Wet & Soft) (Fr Wat on Surf)
800	-	1.2	Lt Br Cl with Si Tr Sa (Wet & Stiff)

Station 19+200 16.0 Lt (D-3.9)

0	-	600	Blk Orgs (Co Fib) (Wet & Soft) (Fr Wat on Surf)
600	-	1.2	Lt Br Si(y) Cl Occ Sa (Wet & Stiff)

Station 19+220 18.0 Rt (D-5.1)

0	-	200	Tps (Moist)
200	-	1.2	Lt Br Si(y) Cl (Wet & Firm)

Station 19+230 16.0 Lt (D-3.1)

0	-	75	Tps
75	-	400	Br F-Co Si(y) Sa (Wet)
400	-	1.0	Lt Br Si(y) Cl Tr Sa (Wet & Stiff)
		1.0	NFP Poss BR

Station 19+240 4.3 Rt

0	-	50	Asph
50	-	250	Cr Gr
250	-	1.0	Br F-Co Sa Tr Si Occ Gr (Moist)
1.0	-	1.2	Rk Fill
		1.2	NFP Rk Fill

Station 19+250 11.0 Lt (D-1.4)

0	-	100	Tps
100	-	500	Rd F-Co Si(y) Sa Tr Gr & Cl (Moist)
		700	NFP Sh Rk

Station 19+260 12.0 Lt (D-1.0)

0	-	50	Tps
50	-	400	Br F Sa
400	-	700	Lt Br Si(y) Sa with Cl (Wet)
		700	NFP Prob Frag BR

Station 19+270 15.0 Lt (D+500)

Frag BR on Surf

Station 19+280 15.0 Lt (D+1.6)

Frag BR on Surf

Station 19+280 14.0 Rt (D-2.2)

0	-	150	Tps
150	-	500	Lt Br Cl with Sa, Si & Cobs (Moist)
500	-	800	Br F-Co Sa with Cobs & Blds (Moist)
		800	NFP Blds

Station 19+290 12.0 Lt (D+2.3)

0	-	25	Ob to BR

Station 19+300 4.2 Lt

0	-	70	Asph
70	-	200	Cr Gr
200	-	1.4	Br F-Co Sa with VF Gr Tr Si Occ Cob (Moist)
1.4	-	1.5	Sh Rk
		1.5	NFP Sh Rk Poss Frag BR

Station 19+300 8.0 Lt (D-300)

0	-	200	Ob to Sh Rk

Station 19+300 15.0 Lt (D+1.8)

0	-	25	Ob to BR

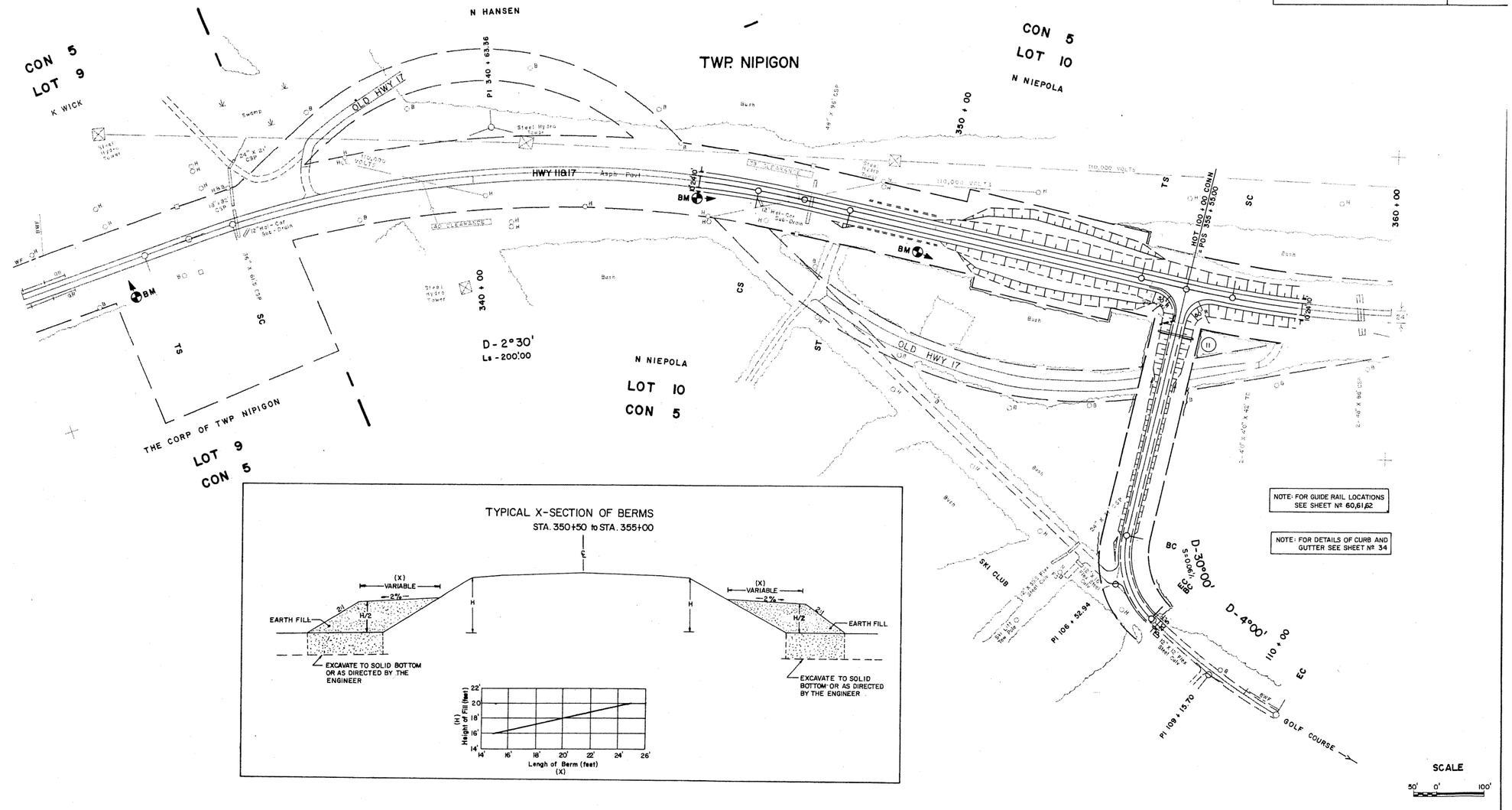
Station 19+300 14.0 Rt (D-2.3)

0	-	25	Tps
25	-	600	Br VF-Co Si(y) Sa Tr Cl Occ Gr (Wet)
		600	NFP Poss Frag BR

Station 19+305 15.0 Lt (D+500)

0	-	25	Tps
25	-	700	Br F Si(y) Sa
		700	NFP BR

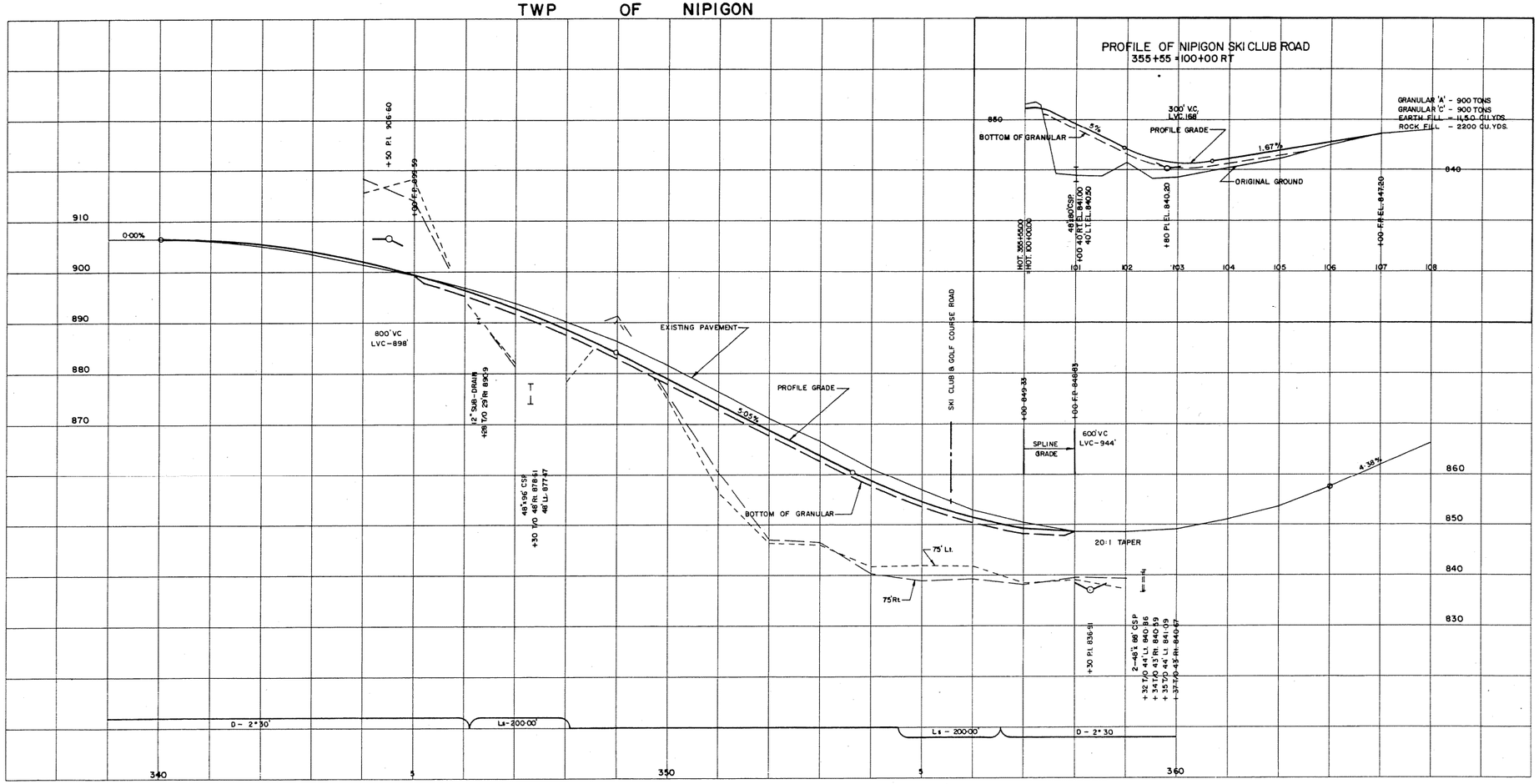
CONT. No. 74-46 W. P. No. 901-71-02	
NEW CONSTRUCTION STA. 330+00 TO STA. 360+00	SHEET 23



TYPICAL X-SECTION OF BERMS
STA. 350+50 to STA. 355+00

CONT. No. 74-46 W. P. No. 901-71-02	
NEW CONSTRUCTION STA. 330+00 TO STA. 360+00	SHEET 24

TWP OF NIPIGON



QUANTITIES

Sta.	– Sta.	
E.C.		7,350 C.Y.
St.		C.Y.
E.D.		C.Y.
M.E.		2,850 C.Y.
M.B.E.		3,300 C.Y.
E.F.		6,550 C.Y.
R.C.		660 C.Y.
Sh.		300 C.Y.
R.D.		C.Y.
M.B.R.		C.Y.
R.F.		C.Y.

Sta.	– Sta.	
E.C.		C.Y.
St.		C.Y.
E.D.		C.Y.
M.E.		C.Y.
M.B.E.		C.Y.
E.F.		C.Y.
R.C.		C.Y.
Sh.		C.Y.
R.D.		C.Y.
M.B.R.		C.Y.
R.F.		C.Y.

SCALES

50' 0 100'
Horizontal

5' 0 10'
Vertical

SOILS SURVEY DATA		
DATE OF SURVEY	TYPE OF SURVEY	LOCATION
JUNE 1973	POWER AUGER	SUICIDE HILL
JULY 1973	HAND AUGER	" "
AUGUST 1972	FOUNDATION	" "

CONT. No. 74-46
W. P. No. 907-71-01

STA. TO STA.

SHEET 24'A'

Station 345+00 13' Rt ℄
0" - 5" Gran "A" (AGM)
5" - 36" Br F-M Sa (Occ F Grav) (Mod Moist)
36" N F P Sh Rk Poss B/R

Station 345+00 13' Lt ℄
0" - 4" Pav't
4" - 10" Gran "A" (AGM)
10" - 24" Br F-M Sa (Occ F Grav) (Moist)
24" N F P Sh Rk Poss B/R

Station 345+50 13' Rt ℄
0" - 8" Gran "A" (AGM)
8" - 36" Br F-M Sa (Sl Si) (Occ F Grav)(Moist)
36" N F P B/R

Station 345+50 13' Lt ℄
0" - 4" Pav't
4" - 8" Gran "A" (AGM)
8" - 22" Br F-M Sa (Occ F Grav) (Moist)
22" N F P Sh Rk Poss B/R

Station 346+00 12' Rt ℄
0" - 4" Pav't
4" - 14" Gran "A" (AGM)
14" - 38" Br F-M Sa (Sl Si) (Occ F Grav)(Moist)
38" N F P Sh Rk

Station 346+00 13' Lt ℄
0" - 3" Pav't
3" - 8" Gran "A" (AGM)
8" - 28" Br F-M Sa (Occ F Grav) (Moist)
28" N F P Sh Rk Fill

Station 346+50 15' Rt ℄
0" - 2" Mulch
2" - 12" Gran "A"
12" - 40" Br F Sa (Sl Si) (Occ F Grav)(Moist)
40" N F P Sh Rk Fill

Station 346+50 13' Lt ℄
0" - 4" Pav't
4" - 8" Gran "A" (AGM)
8" - 30" Br F-M Sa (Occ F Grav) (Moist)
30" N F P Sh Rk Fill

Station 347+00 15' Rt ℄
0" - 2" Mulch
2" - 7" Gran "A" (AGM)
7" - 12" Br F Gravy Sa (Moist (Sl Si)
12" - 38" Br F Sa (Moist)
38" N F P Sh Rk Fill

Station 347+00 13' Lt ℄
0" - 4" Pav't
4" - 10" Gran "A" (AGM)
10" - 32" Br F-M Sa (Occ F Grav) (Moist)
32" N F P Sh Rk Fill

Station 347+50 15' Rt ℄
0" - 1" Mulch
1" - 12" Gran "A" (AGM)
12" - 38" Br F Sa (Occ F Grav) (Moist)
38" N F P Sh Rk Fill

Station 347+50 13' Lt ℄
0" - 4" Pav't
4" - 8" Gran "A" (AGM)
8" - 10" Br F-M Sa (Occ F Grav) (Moist)
10" - 13" Br F Grav (Moist)
13" - 18" Br F-M Sa (Occ F Grav) (Moist)
18" - 26" Br F Gravy Sa (Moist)
26" - 36" Br F-M Sa (Occ F Grav) (Moist)
36" N F P Sh Rk Fill

Station 348+00 13' Lt ℄
0" - 4" Pav't
4" - 12" Gran "A" (AGM)
12" - 15" Br F-M Sa (Occ F Grav) (Moist)
15" - 24" Br F Grav (Moist)
24" - 36" Br F-M Sa (Occ F Grav) (Moist)
36" N F P Sh Rk Fill

Station 348+50 15' Rt ℄
0" - 2" Mulch
2" - 9" Gran "A" (AGM)
9" - 24" Br F Sa (Moist)
24" - 36" Br F-M Sa & Sh Rk Mix
36" N F P Sh Rk Poss B/R

Station 348+50 13' Lt ℄
0" - 3" Pav't
3" - 16" Gran "A" (AGM)
16" - 30" Br F-M Sa (Occ F Grav) (Moist)
30" N F P Sh Rk Poss B/R

Station 349+00 15' Rt ℄
0" - 10" Gran "A" (AGM)
10" - 28" Br F-M Sa (Occ F Grav) (Moist)
28" - 36" Sh Rk Fill
36" N F P Sh Rk Poss B/R

Station 349+00 13' Lt ℄
0" - 4" Pav't
4" - 14" Gran "A" (AGM)
14" - 26" Br F-M Sa (Occ F Grav) (Moist)
26" N F P Sh Rk Poss B/R

Station 349+50 13' Rt ℄
0" - 3" Pav't
3" - 14" Gran "A" (AGM)
14" - 28" Br F-M Sa (Occ F Grav) (Moist)
28" - 36" Sh Rk Fill
36" N F P Sh Rk Fill

Station 349+50 13' Lt ℄
0" - 4" Pav't
4" - 12" Gran "A" (AGM)
12" - 22" Br F-M Sa (Occ F Grav) (Moist)
22" N F P Sh Rk Poss B/R

Station 350+00 13' Rt ℄
0" - 3" Pav't
3" - 14" Gran "A" (AGM)
14" - 30" Br F-M Sa (Occ F Grav) (Moist)
30" N F P Sh Rk Fill

Station 350+00 13' Lt ℄
0" - 3" Pav't
3" - 14" Gran "A" (AGM)
14" - 28" Br F-M Sa (Occ F Grav) (Moist)
28" N F P Sh Rk Fill

Station 350+50 13' Rt ℄
0" - 4" Pav't
4" - 12" Gran "A" (AGM)
12" - 32" Br F-M Sa (Occ F Grav) (Moist)
32" N F P Sh Rk Fill

Station 350+50 13' Lt ℄
0" - 4" Org
4" - 16" Gran "A" (AGM)
16" - 30" Br F-M Sa (Occ F Grav) (Moist)
30" N F P Sh Rk Fill

Station 351+00 13' Rt ℄
0" - 4" Pav't
4" - 18" Gran "A" (AGM)
18" - 44" Br F-M Sa (Occ F Grav) (Moist)
44" N F P Sh Rk Fill

Station 351+00 13' Lt ℄
0" - 6" Pav't
6" - 18" Gran "A" (AGM)
18" - 40" Br F-M Sa (Occ F Grav) (Moist)
40" N F P Sh Rk Fill

Station 351+00 55' Lt ℄
0" - 8" Org
8" - 48" Si Cl Sa Mix
48" N F P Poss B/R

Station 351+00 54' Lt ℄
0" - 8" Org
8" - 48" Si Cl (Wet @ 44")

Station 351+50 13' Rt ℄
0" - 5" Pav't
6" - 14" Gran "A" (AGM)
14" - 36" Br F-M Sa (Occ F Grav) (Moist)
36" - 40" Sh Rk Fill
40" N F P Sh Rk Fill

Station 351+50 13' Lt ℄
0" - 7" Pav't
7" - 16" Gran "A" (AGM)
16" - 32" Br F-M Sa (Occ F Grav) (Moist)
32" N F P Sh Rk Fill

Station 352+00 13' Rt ℄
0" - 6" Pav't
6" - 14" Gran "A" (AGM)
14" - 36" Br F-M Sa (Occ F Grav) (Moist)
36" - 42" Sh Rk Fill
42" N F P Sh Rk Fill

Station 352+00 13' Lt ℄
0" - 6" Pav't
6" - 10" Gran "A" (AGM)
10" - 30" Br F-M Sa (Occ F Grav) (Moist)
30" N F P Sh Rk Fill

Station 352+00 60' Rt ℄
0" - 6" Org
6" - 60" Firm Si Cl

Station 352+00 63' Lt ℄
0" - 12" Org
12" - 48" Firm Si Cl

Station 352+00 70' Lt ℄
0" - 24" Org
24" - 60" Firm Si Cl

Station 352+00 70' Rt ℄
0" - 24" Org
24" - 84" Firm Si Cl

Station 352+50 13' Rt ℄
0" - 8" Pav't
8" - 10" Gran "A" (AGM)
10" - 12" Pav't
12" - 18" Gran "A" (AGM)
18" - 32" Br F-M Sa (Occ F Grav) (Moist)
32" N F P Sh Rk Fill

Station 352+50 13' Lt ℄
0" - 10" Gran "A" (AGM)
10" - 22" Br F-M Sa (Occ F Grav) (Moist)
22" N F P Sh Rk Fill

Station 353+00 56' Lt ℄
0" - 24" Org
24" - 84" Firm Si Cl

Station 353+00 62' Rt ℄
0" - 18" Org
18" - 60" Firm Si Cl

Station 353+15 13' Lt ℄
0" - 8" Pav't
8" - 14" Gran "A" (AGM)
14" - 28" Br F-M Sa (Occ F Grav) (Moist)
28" N F P Sh Rk Fill

Station 353+17 14' Rt ℄
0" - 12" Gran "A" (AGM)
12" - 36" Br F-M Sa (Occ F Grav) (Moist)
36" N F P Sh Rk Fill

Station 353+50 13' Lt ℄
0" - 8" Pav't
8" - 14" Gran "A" (AGM)
14" - 24" Br F-M Sa (Occ F Grav) (Moist)
24" N F P Sh Rk Fill

Station 353+60 13' Rt ℄
0" - 5" Pav't
5" - 12" Gran "A" (AGM)
12" - 32" Br F-M Sa (Moist) (Occ F Grav)
32" N F P Sh Rk Fill

Station 353+75 60' Rt ℄
0" - 40" Org
40" - 90" Firm Si Cl
90" - 144" Soft Si Cl
144" - 180" Soft Si Cl (Varved)
180" - 300" Soft Gr Cl & Si
300" - 324" Co Sa & Rk Fragments

Station 353+75 60' Lt ℄
0" - 48" Org
48" - 140" Firm Si Cl (Varved)
140" - 160" Soft Si Cl
160" - 240" Soft Si Cl (Varved) Fractured B/R
240"

Station 354+00 50' Lt ℄
0" - 24" Org
24" - 84" Firm Si Cl

Station 354+00 70' Rt ℄
0" - 30" Org
30" - 96" Firm Si Cl

Station 354+00 13' Rt ℄
0" - 4" Pav't
4" - 15" Gran "A" (AGM)
15" - 40" Br F-M Sa (Occ F Grav) (Moist)
40" N F P Sh Rk Fill

Station 354+00 13' Lt ℄
0" - 5" Pav't
5" - 14" Gran "A" (AGM)
14" - 28" Br F-M Sa (Occ F Grav) (Moist)
28" N F P Sh Rk Fill

Station 354+50 13' Rt ℄
0" - 14" Gran "A" (AGM)
14" - 42" Br F-M Sa (Occ F Grav) (Moist)
42" N F P Sh Rk Fill

Station 354+50 13' Lt ℄
0" - 3" Pav't
3" - 13" Gran "A" (AGM)
13" - 26" Br F-M Sa (Occ F Grav) (Moist)
26" N F P Sh Rk Fill

Station 355+00 13' Rt ℄
0" - 4" Pav't
4" - 12" Gran "A" (AGM)
12" - 36" Br F-M Sa (Occ F Grav) (Moist)
38" N F P Sh Rk Fill

Station 355+00 13' Lt ℄
0" - 3" Pav't
3" - 14" Gran "A" (AGM)
14" - 26" Br F-M Sa (Occ F Grav) (Moist)
26" N F P Sh Rk Fill

Station 355+50 13' Lt ℄
0" - 4" Pav't
4" - 14" Gran "A" (AGM)
14" - 38" Br F-M Sa (Occ F Grav) (Moist)
36" N F P Sh Rk Fill

Station 356+00 13' Rt ℄
0" - 4" Pav't
4" - 10" Gran "A" (AGM)
10" - 42" Br F-M Sa (Occ F Grav) (Moist)
42" N F P Sh Rk Fill

Station 356+00 13' Lt ℄
0" - 3" Pav't
3" - 14" Gran "A" (AGM)
14" - 36" Br F-M Sa (Occ F Grav) (Moist)
36" N F P Sh Rk Fill

Station 356+50 13' Rt ℄
0" - 4" Pav't
4" - 12" Gran "A" (AGM)
12" - 44" Br F-M Sa (Occ F Grav) (Moist)
44" N F P Sh Rk Fill

Station 356+50 14' Lt ℄
0" - 14" Gran "A" (AGM)
14" - 28" Br F-M Sa (Occ F Grav) (Moist)
28" N F P Sh Rk Fill

Station 357+00 13' Rt ℄
0" - 3" Pav't
3" - 12" Gran "A" (AGM)
12" - 46" Br F-M Sa (Occ F Grav) (Moist)
46" N F P Sh Rk Fill

Station 357+00 13' Lt ℄
0" - 2" Pav't
2" - 12" Gran "A" (AGM)
12" - 38" Br F-M Sa (Occ F Grav) (Moist)
38" N F P Sh Rk Fill

Station 357+30 52' Lt ℄
0" - 24" Org
24" - 60" Firm Si Cl

Station 357+00 64' Rt ℄
0" - 30" Org
30" - 60" Firm Si Cl
60" - 84" Fi Sa

Station 357+45 13' Rt ℄
0" - 14" Gran "A" (AGM)
14" - 44" Br F-M Sa (Occ F Grav) (Moist)
44" N F P Sh Rk Fill

Station 357+50 13' Lt ℄
0" - 3" Pav't
3" - 12" Gran "A" (AGM)
12" - 42" Br F-M Sa (Occ F Grav) (Moist)
42" N F P Sh Rk Fill

Station 358+00 13' Rt ℄
0" - 2" Pav't
2" - 14" Gran "A" (AGM)
14" - 96" Br F-M Sa (Occ F Grav) (Moist)

Station 358+00 13' Lt ℄
0" - 4" Pav't
4" - 16" Gran "A" (AGM)
16" - 90" Br F-M Sa (Occ F Grav) (Moist)

SCALE

APPENDIX E

RESULTS OF SLOPE STABILITY ANALYSIS

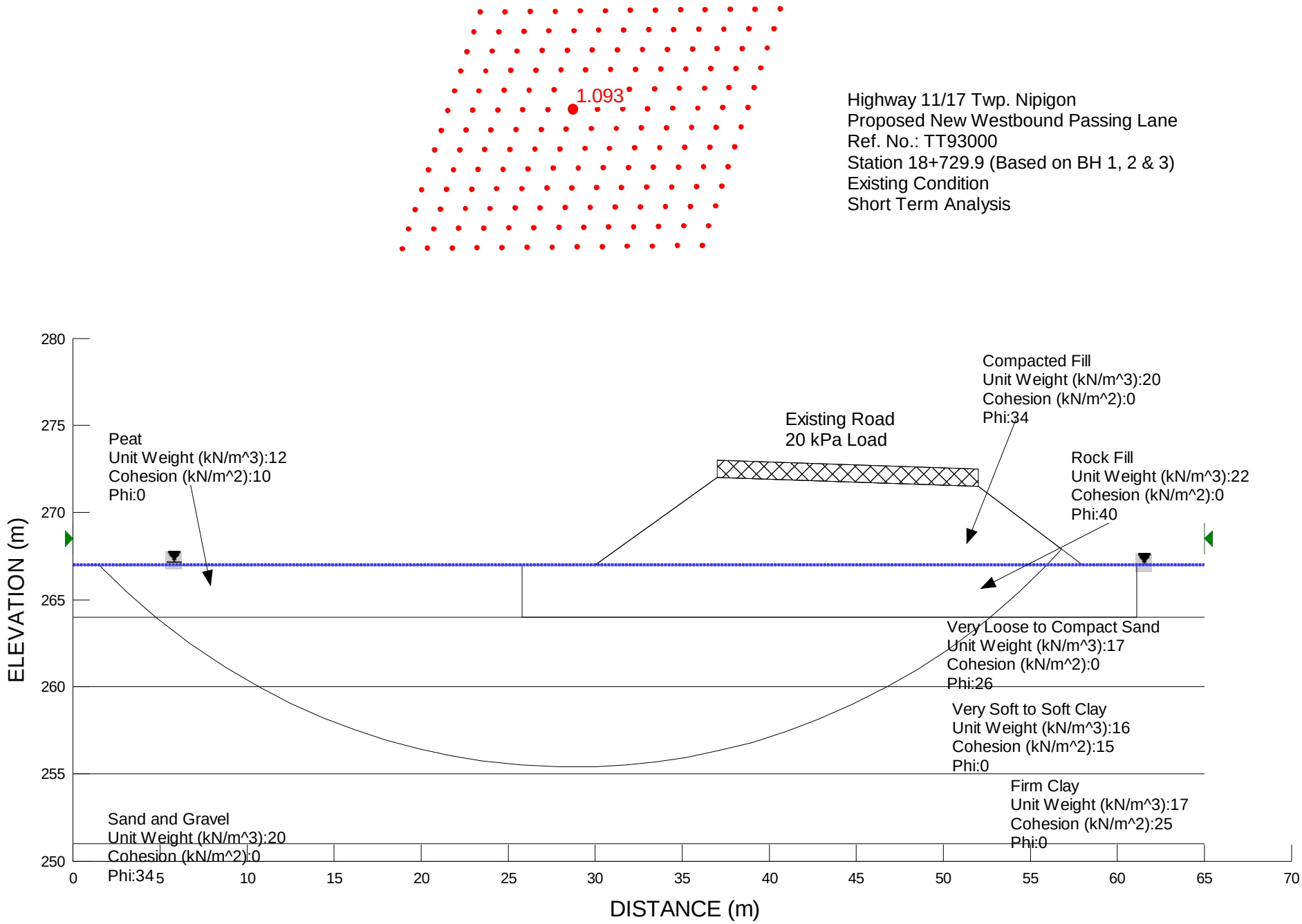


Figure E1.1 Slope Stability Analysis of Section at Station 18+730 (Existing Condition)

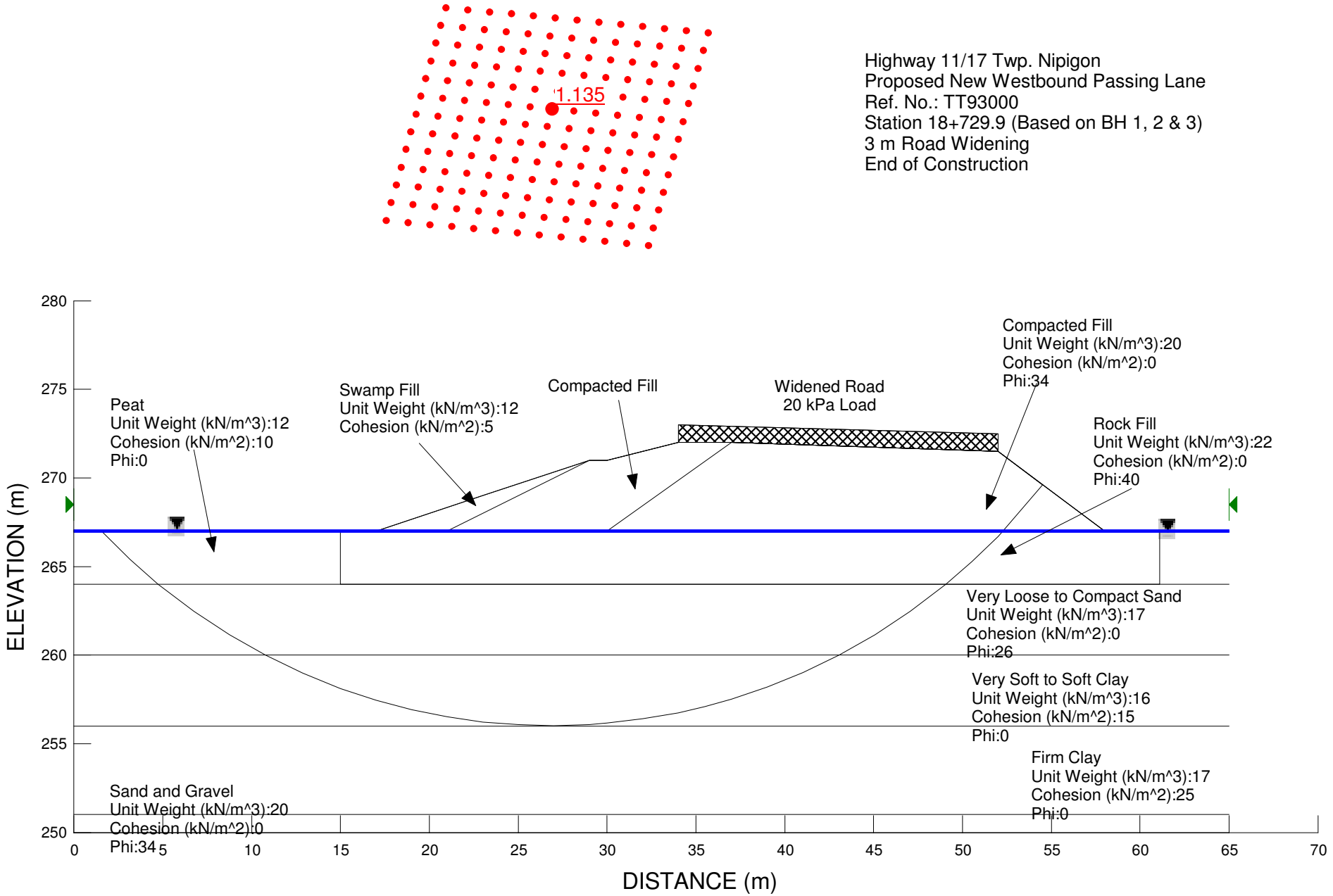


Figure E1.2a Slope Stability Analysis of Section at Station 18+730 with 3 m Road Widening (End of Construction)

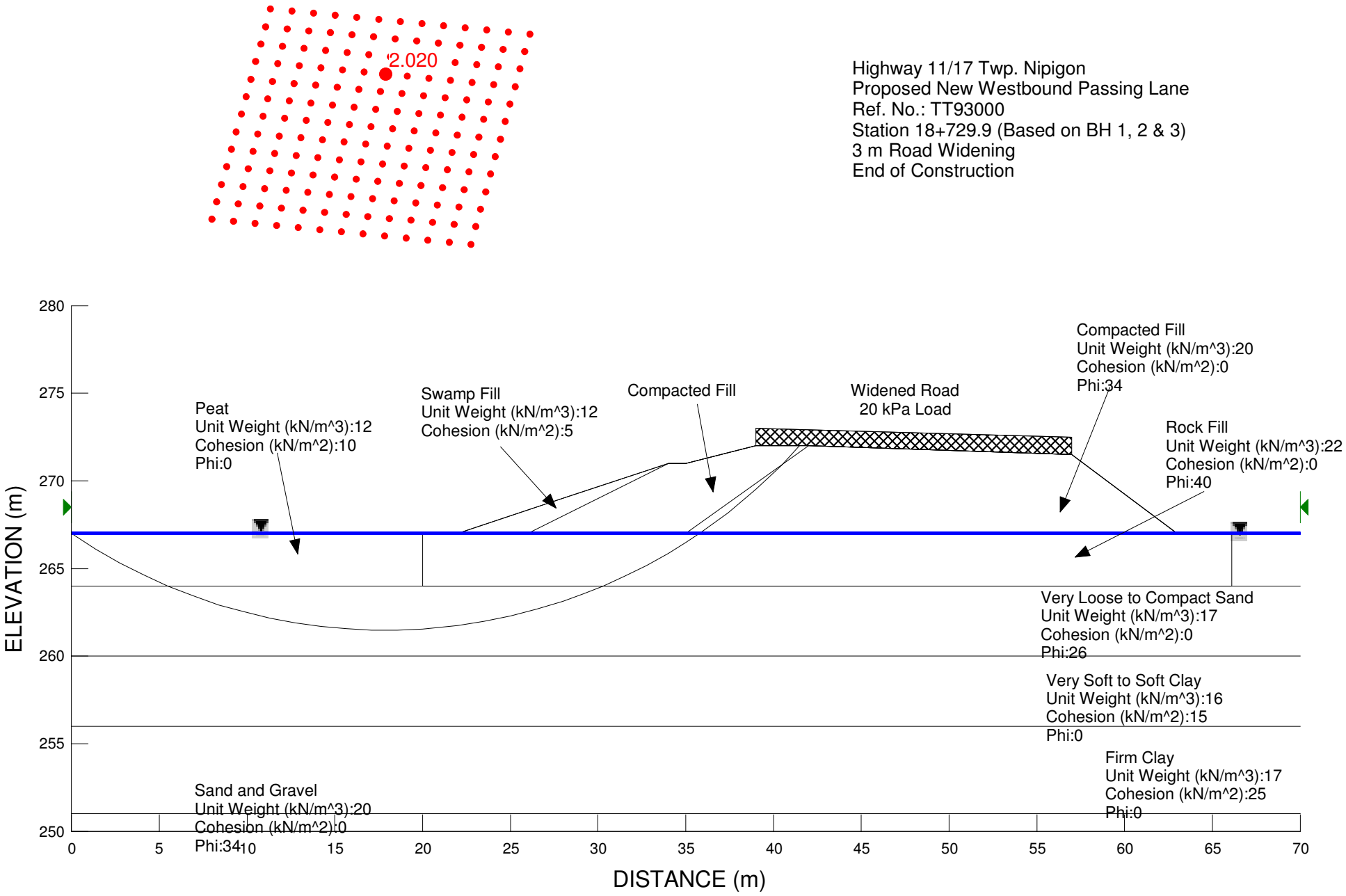


Figure E1.2b Slope Stability Analysis of Section at Station 18+730 with 3 m Road Widening (End of Construction - Shallow Slip Surface)

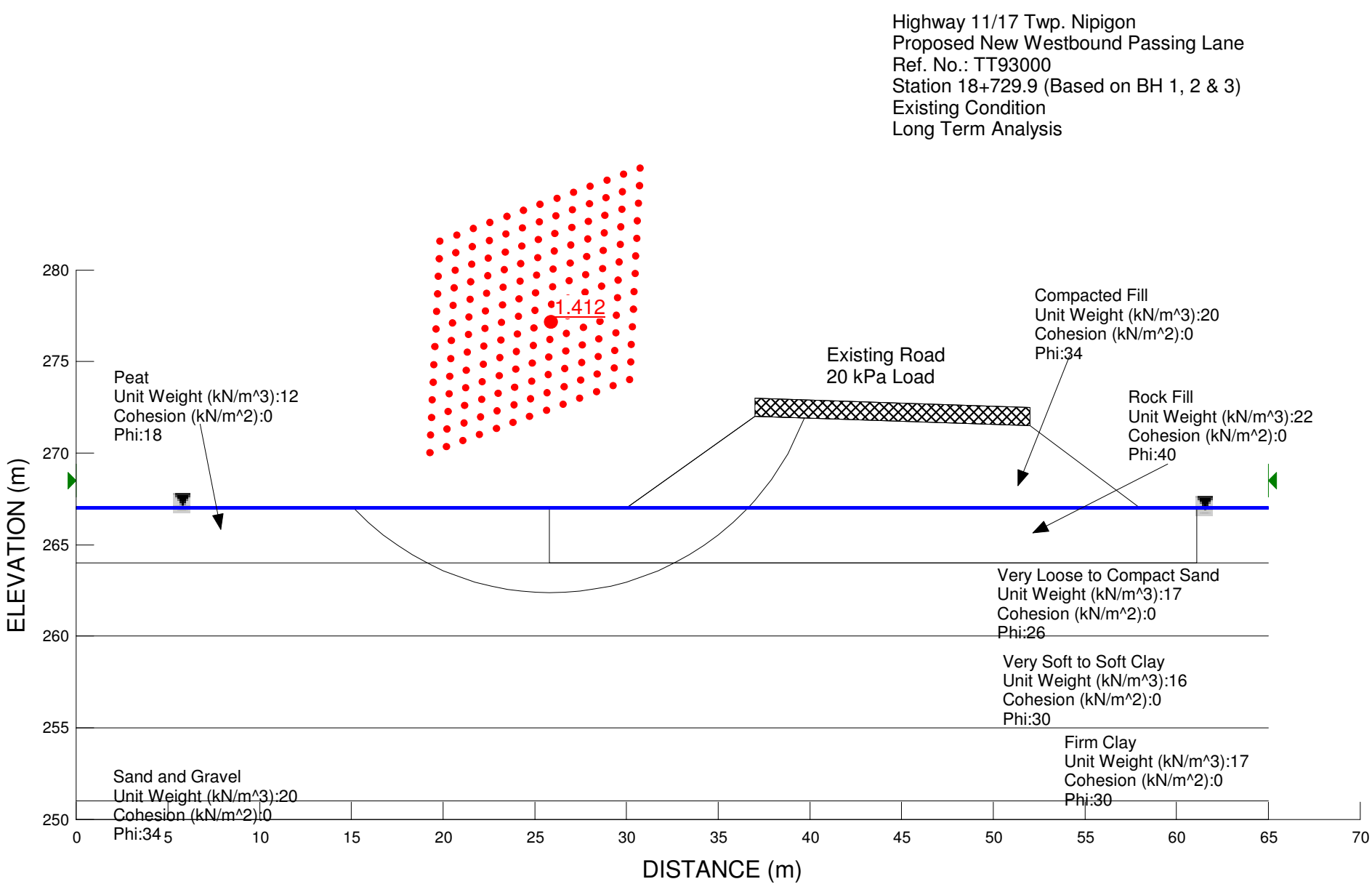


Figure E1.3 Slope Stability Analysis of Section at Station 18+730 (Existing Condition - Long Term Analysis)

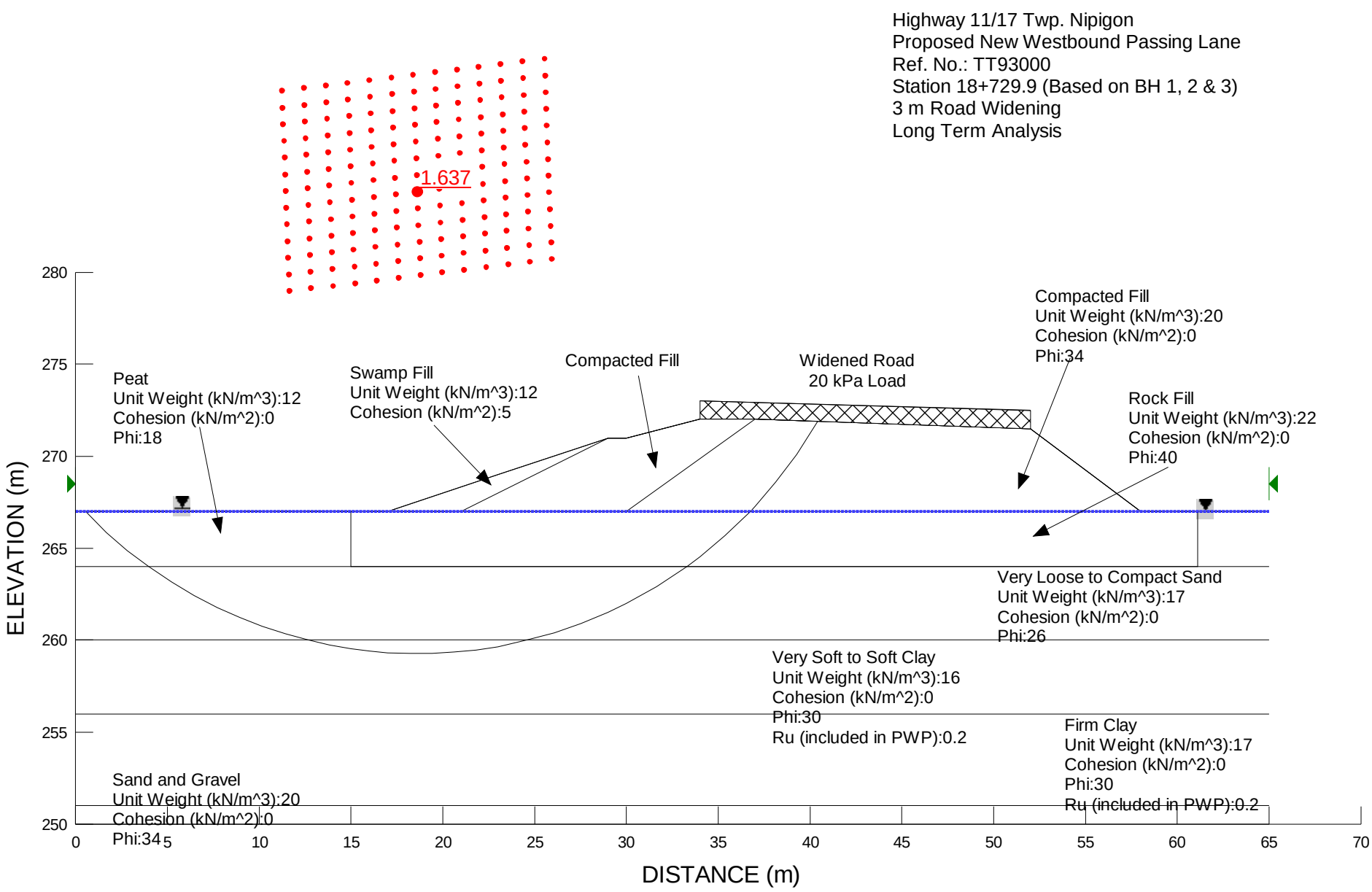


Figure E1.4 Slope Stability Analysis of Section at Station 18+730 with 3 m Road Widening (Long Term Analysis)

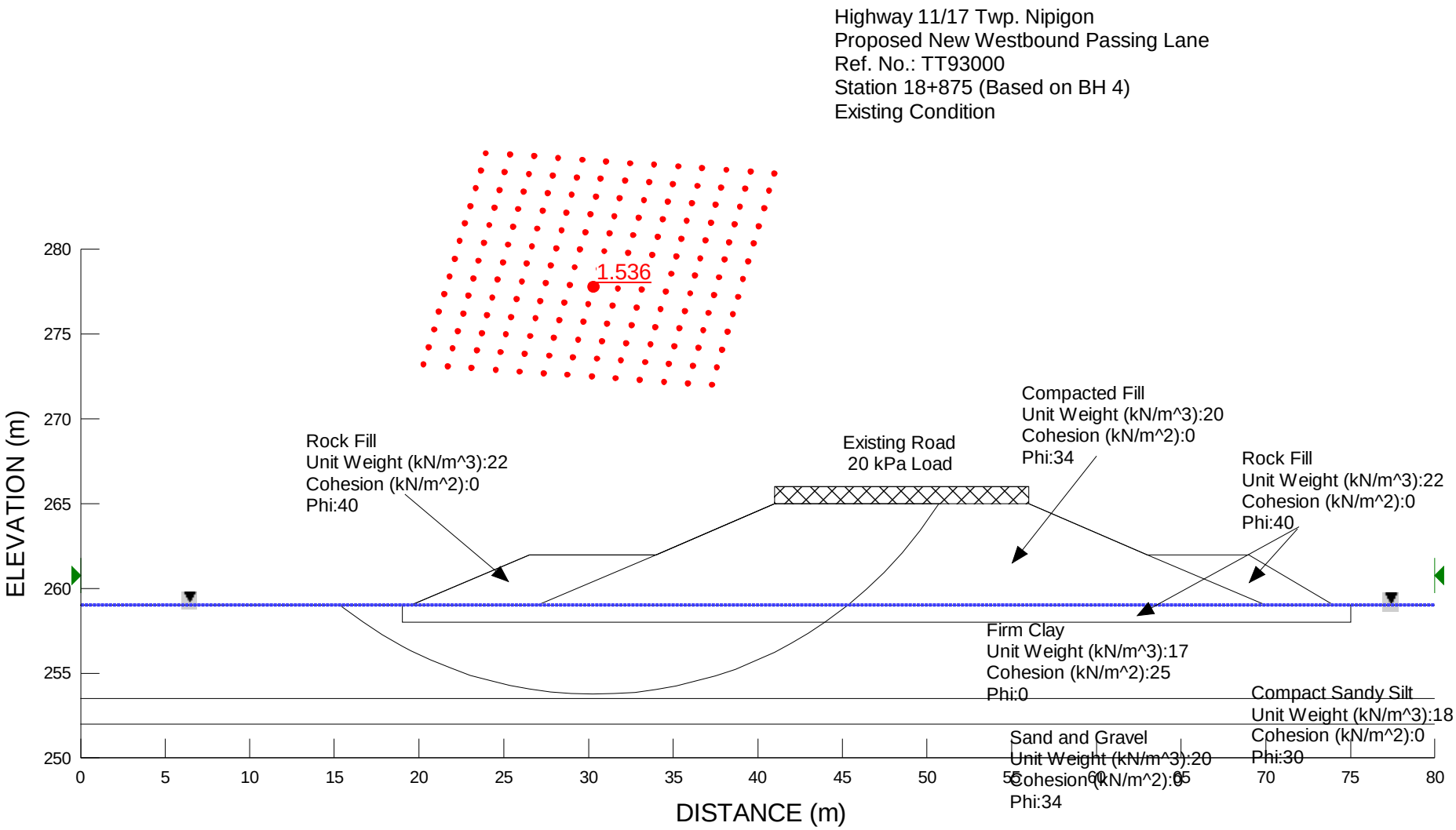


Figure E2.1 Slope Stability Analysis of Section at Station 18+875 (Existing Condition)

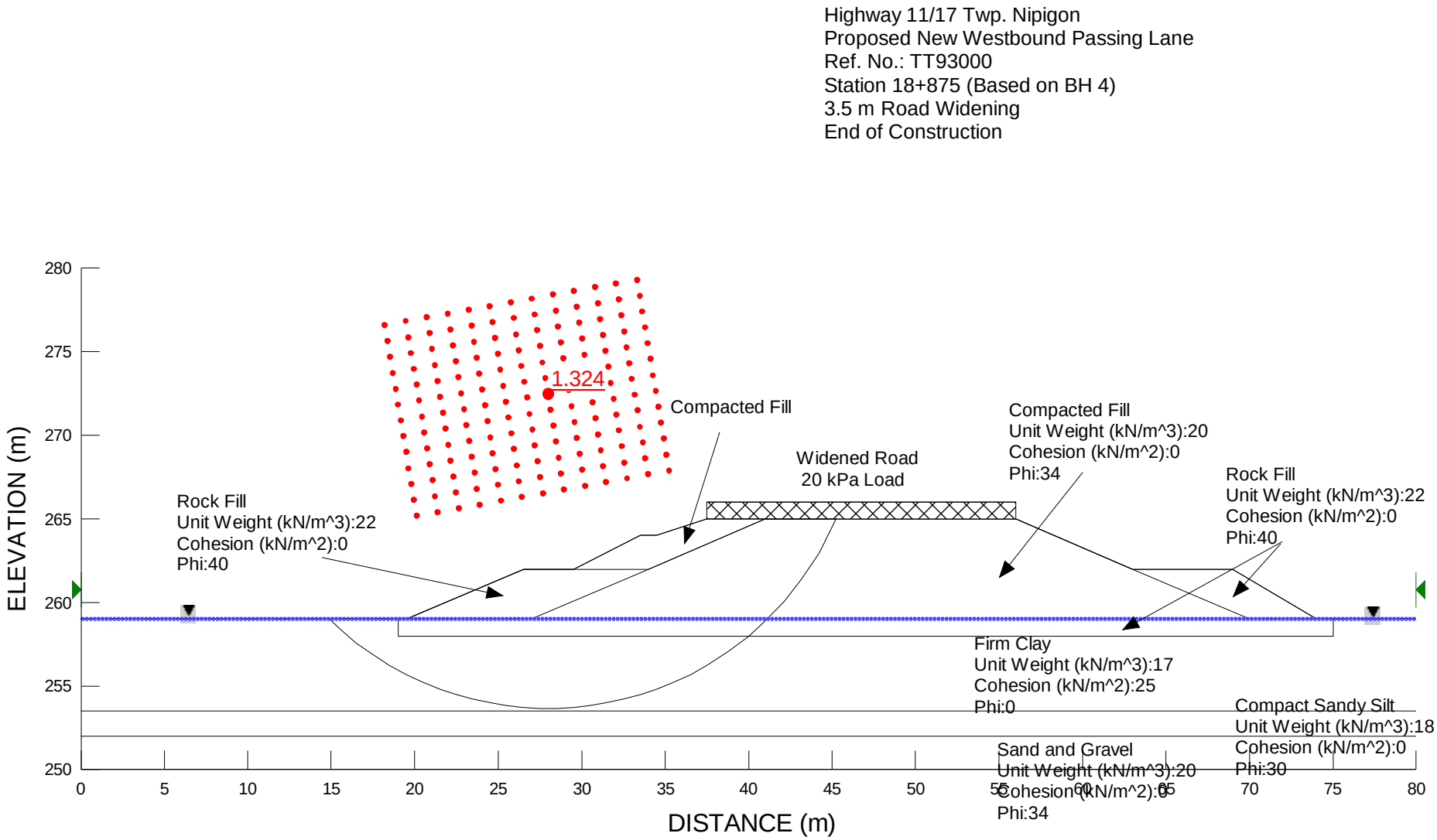


Figure E2.2 Slope Stability Analysis of Section at Station 18+875 with 3.5 m Road Widening (End of Construction)

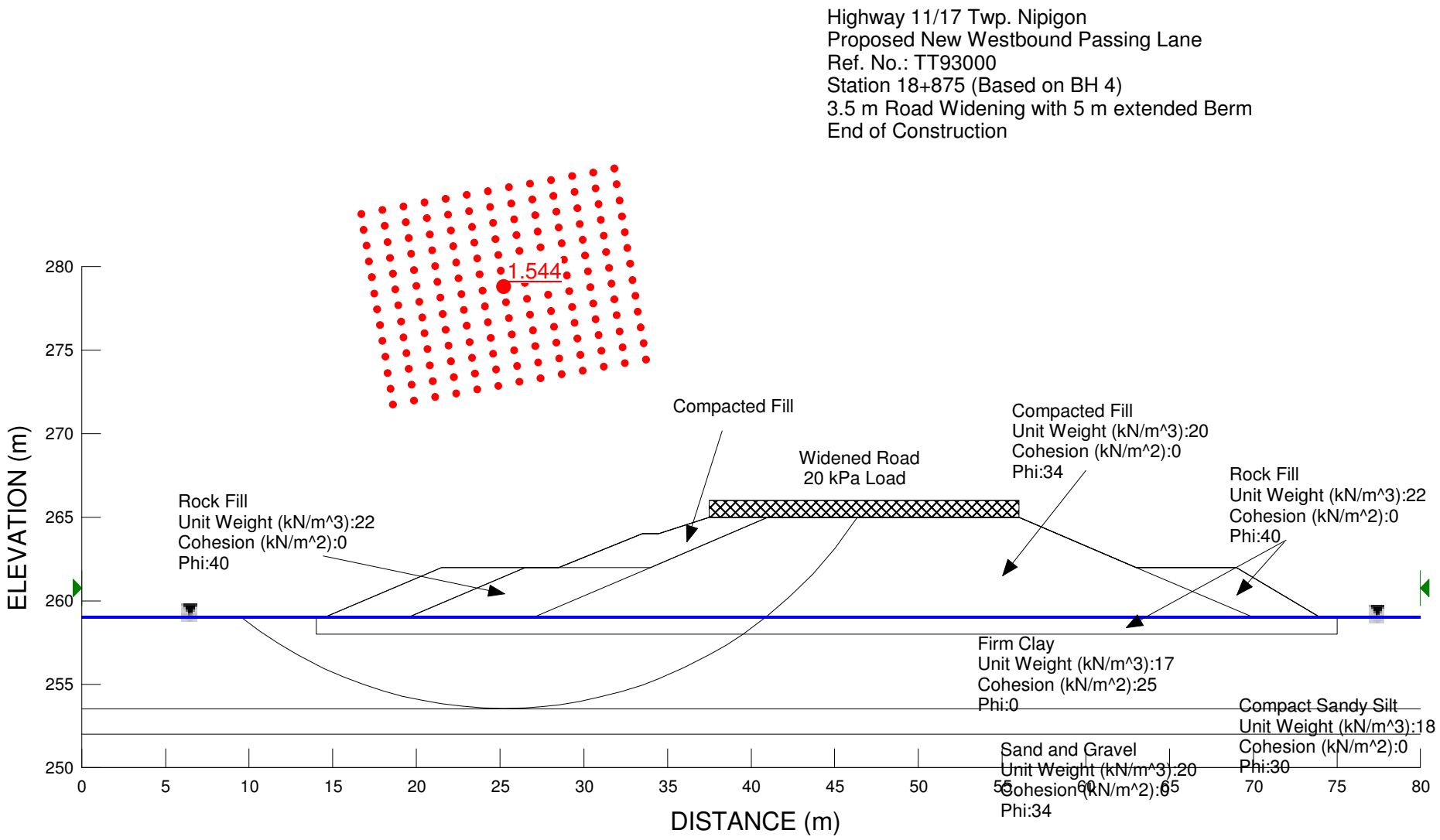


Figure E2.3a Slope Stability Analysis of Section at Station 18+875 - 3.5 m Road Widening with Extended Berm (End of Construction)

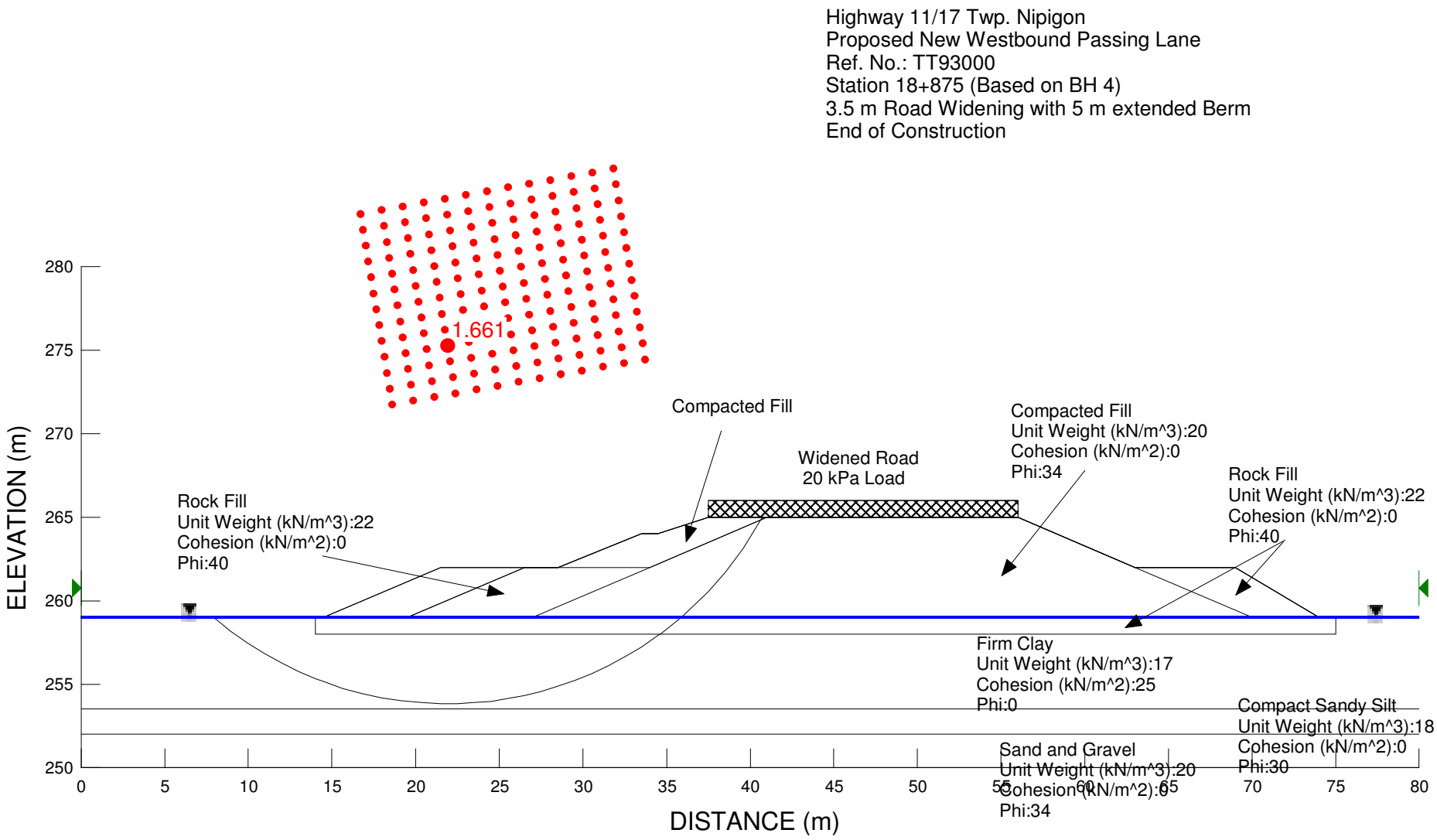


Figure E2.3b Slope Stability Analysis of Section at Station 18+875 - 3.5 m Road Widening with Extended Berm (End of Construction - Shallow Slip Surface)

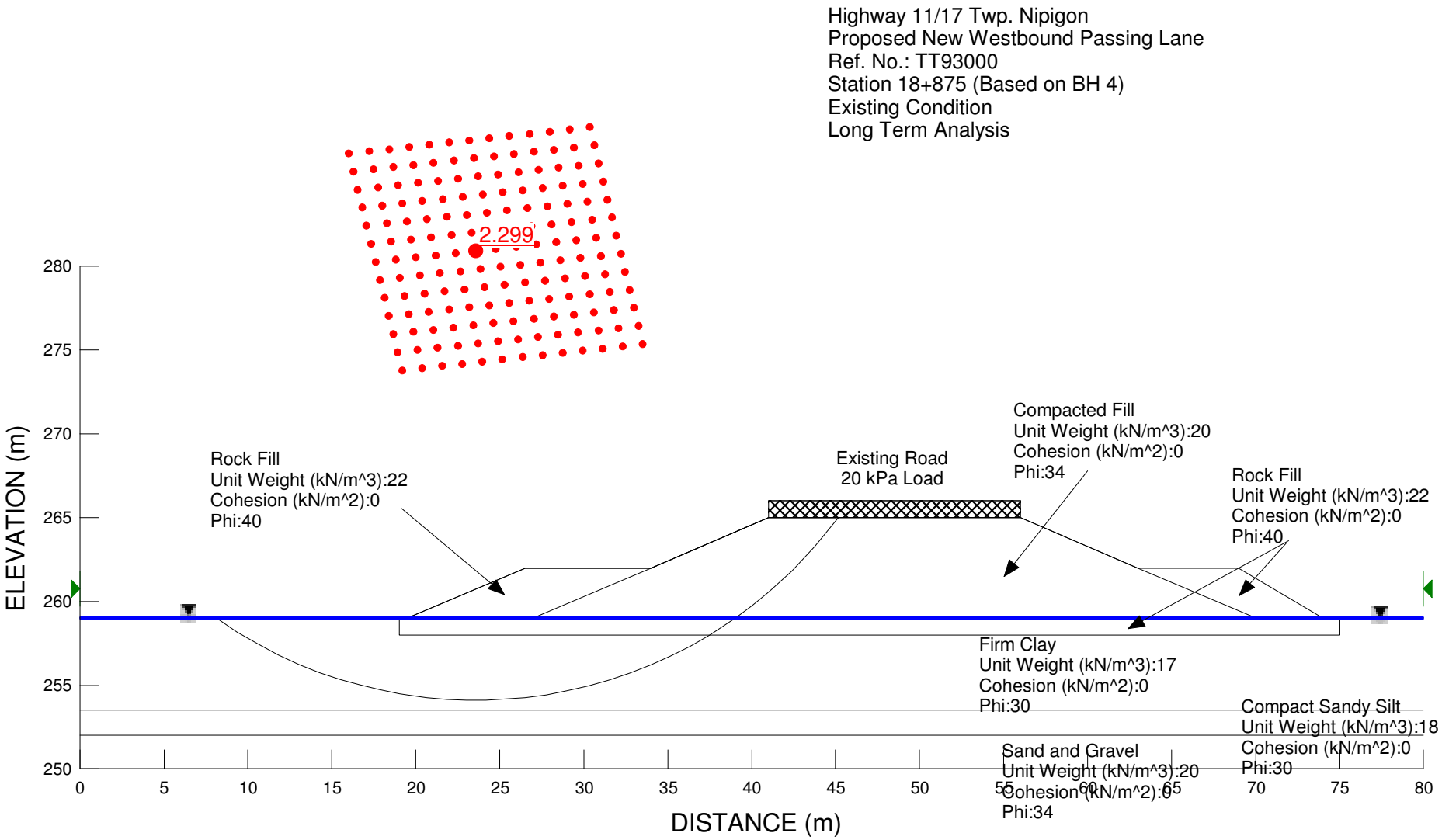


Figure E2.4 Slope Stability Analysis of Section at Station 18+875 (Existing Condition - Long Term Analysis)

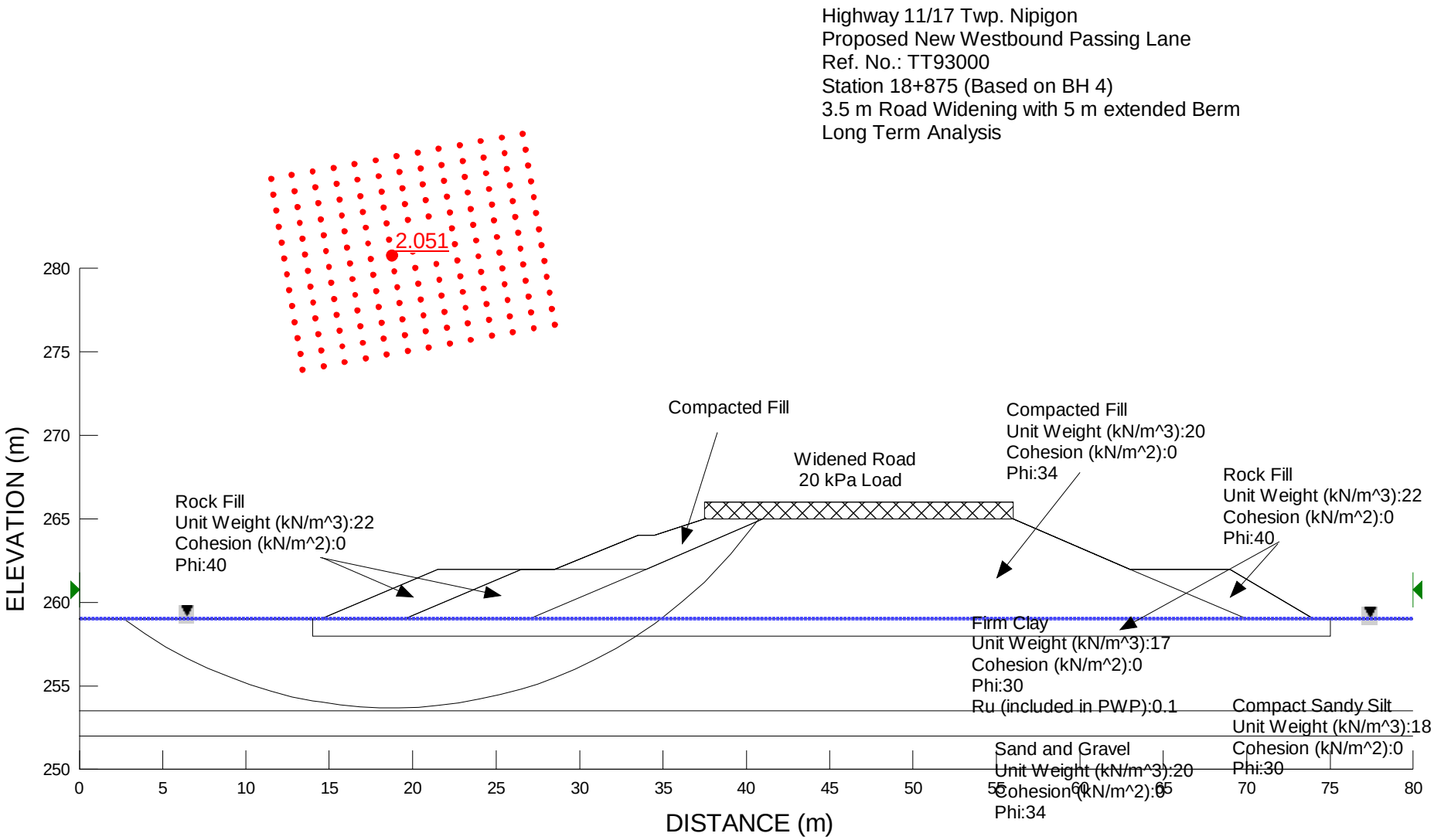


Figure E2.5 Slope Stability Analysis of Section at Station 18+875 - 3.5 m Road Widening with Extended Berm (Long Term Analysis)

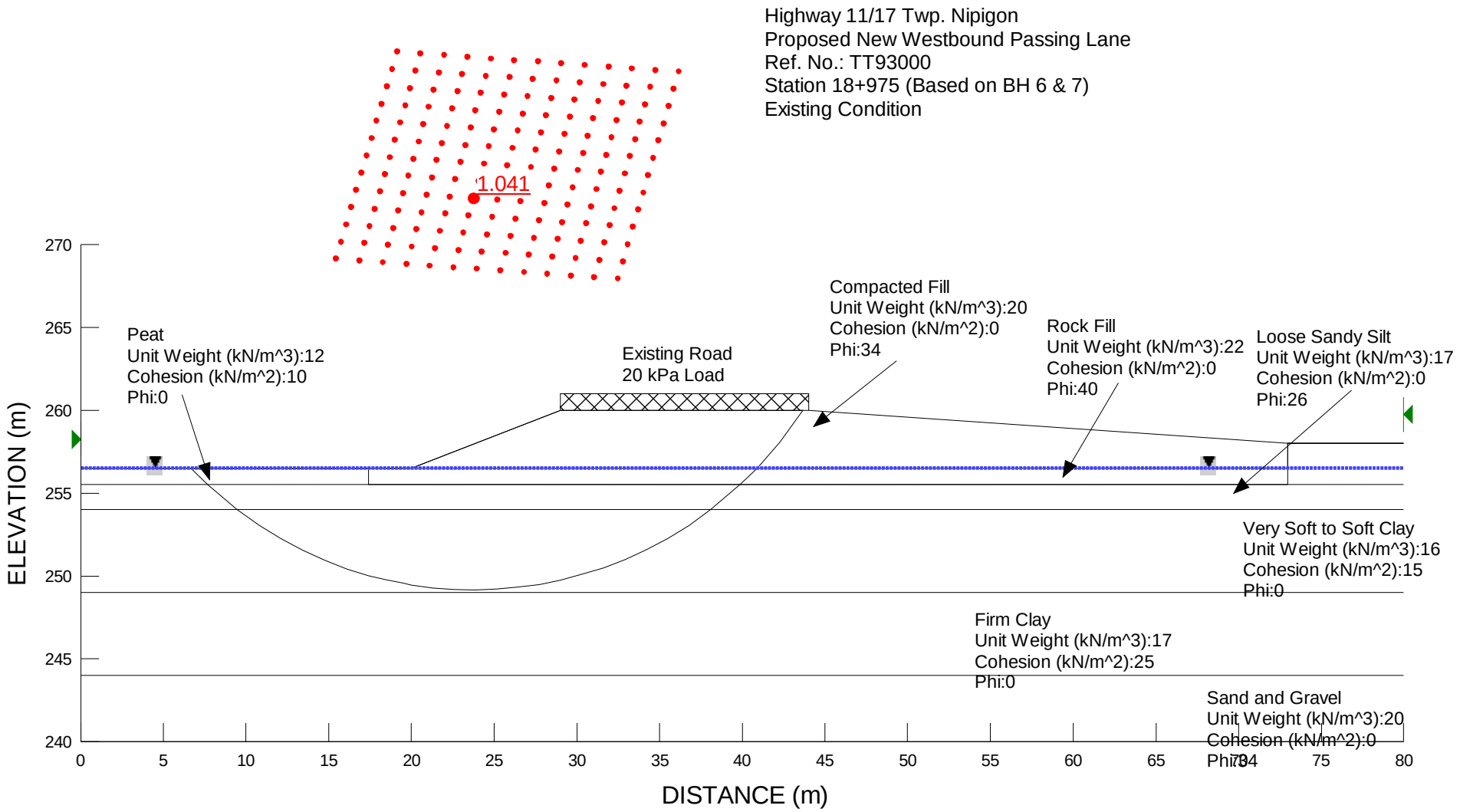


Figure E3.1 Slope Stability Analysis of Section at Station 18+975 (Existing Condition)

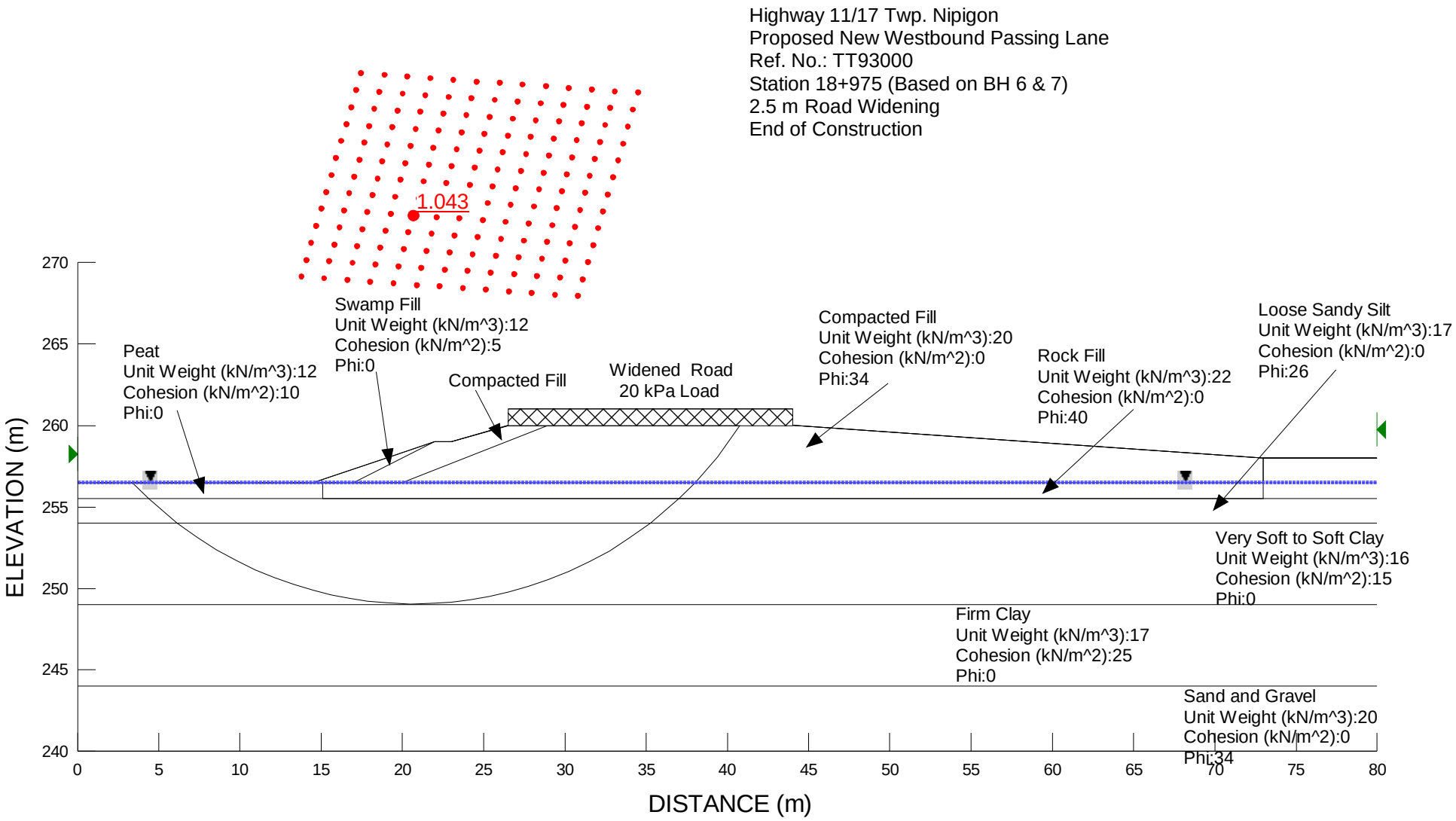


Figure E3.2 Slope Stability Analysis of Section at Station 18+975 with 2.5 m Road Widening (End of Construction)

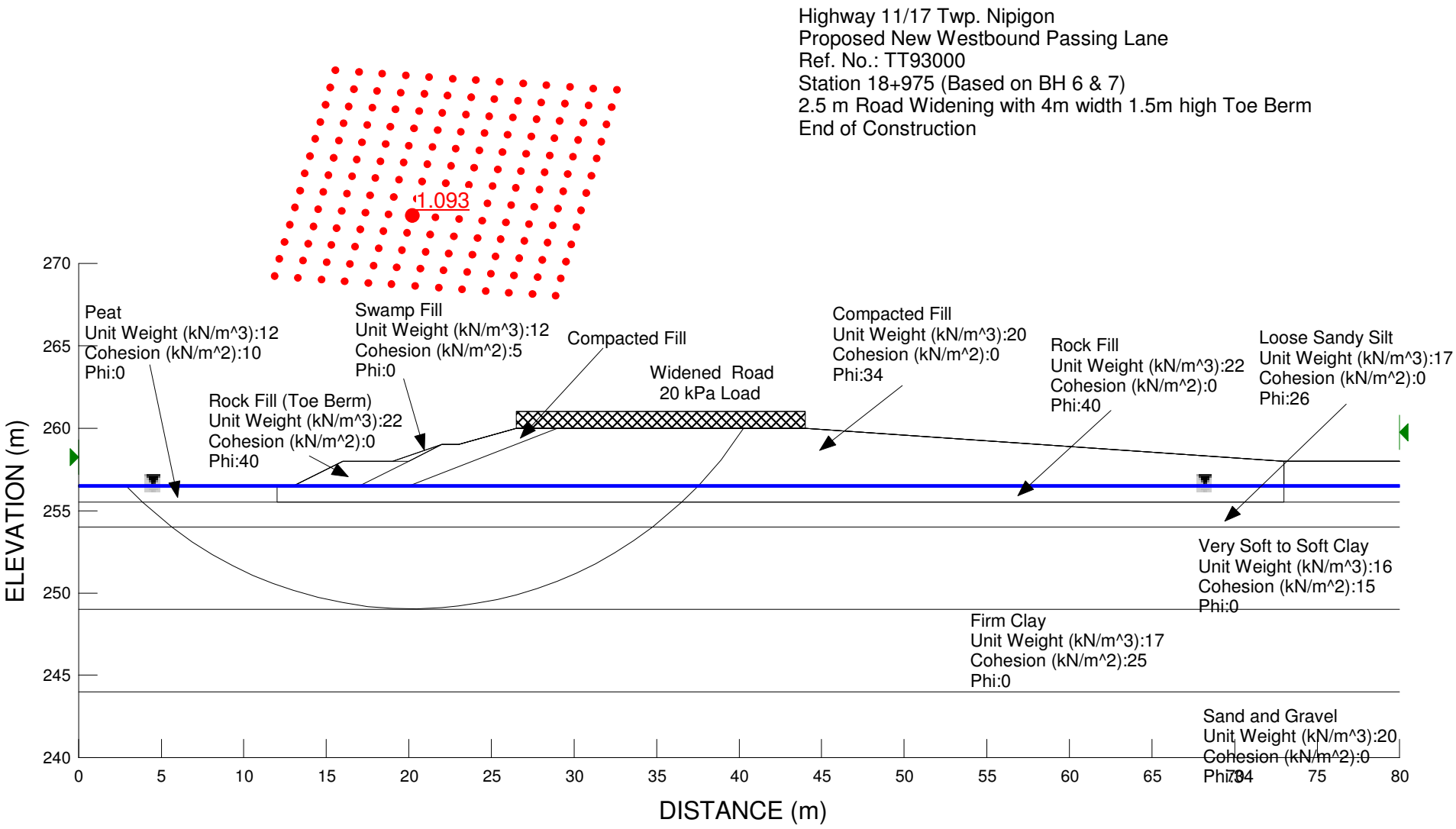


Figure E3.3a Slope Stability Analysis of Section at Station 18+975 - 2.5 m Road Widening with Toe Berm (End of Construction)

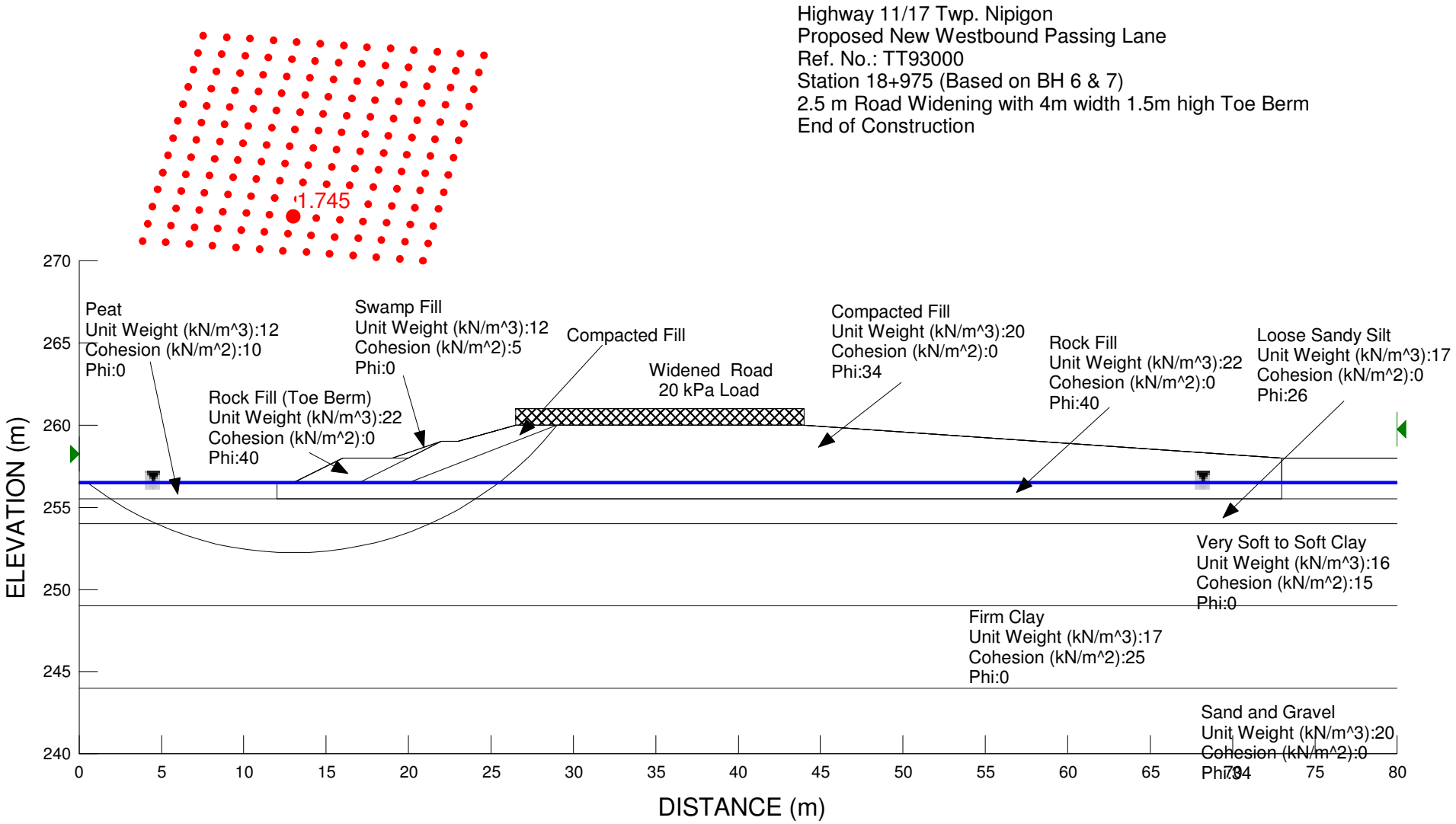


Figure E3.3b Slope Stability Analysis of Section at Station 18+975 - 2.5 m Road Widening with Toe Berm (End of Construction - Shallow Slip Surface)

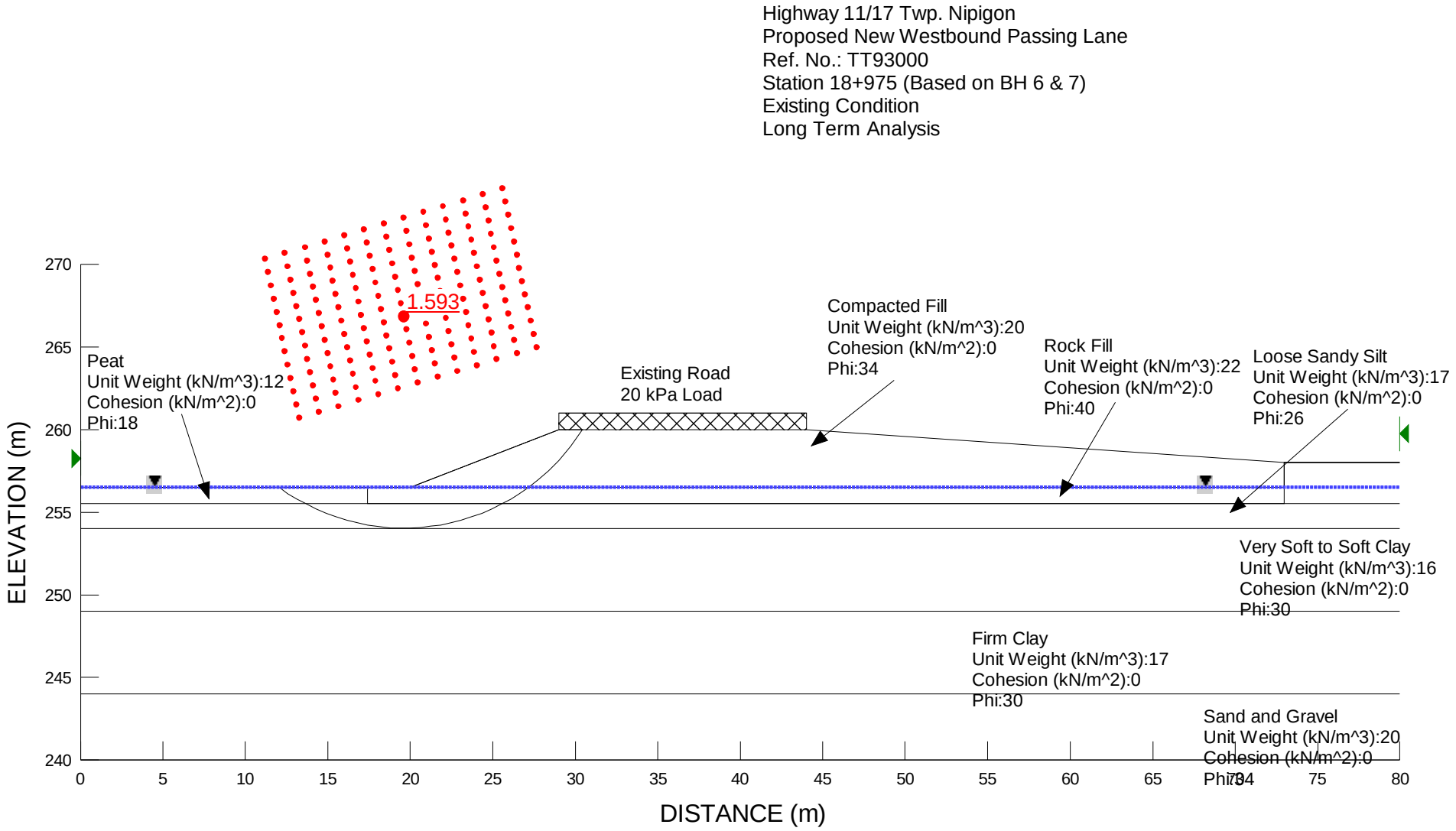


Figure E3.4 Slope Stability Analysis of Section at Station 18+975 (Existing Condition - Long Term Analysis)

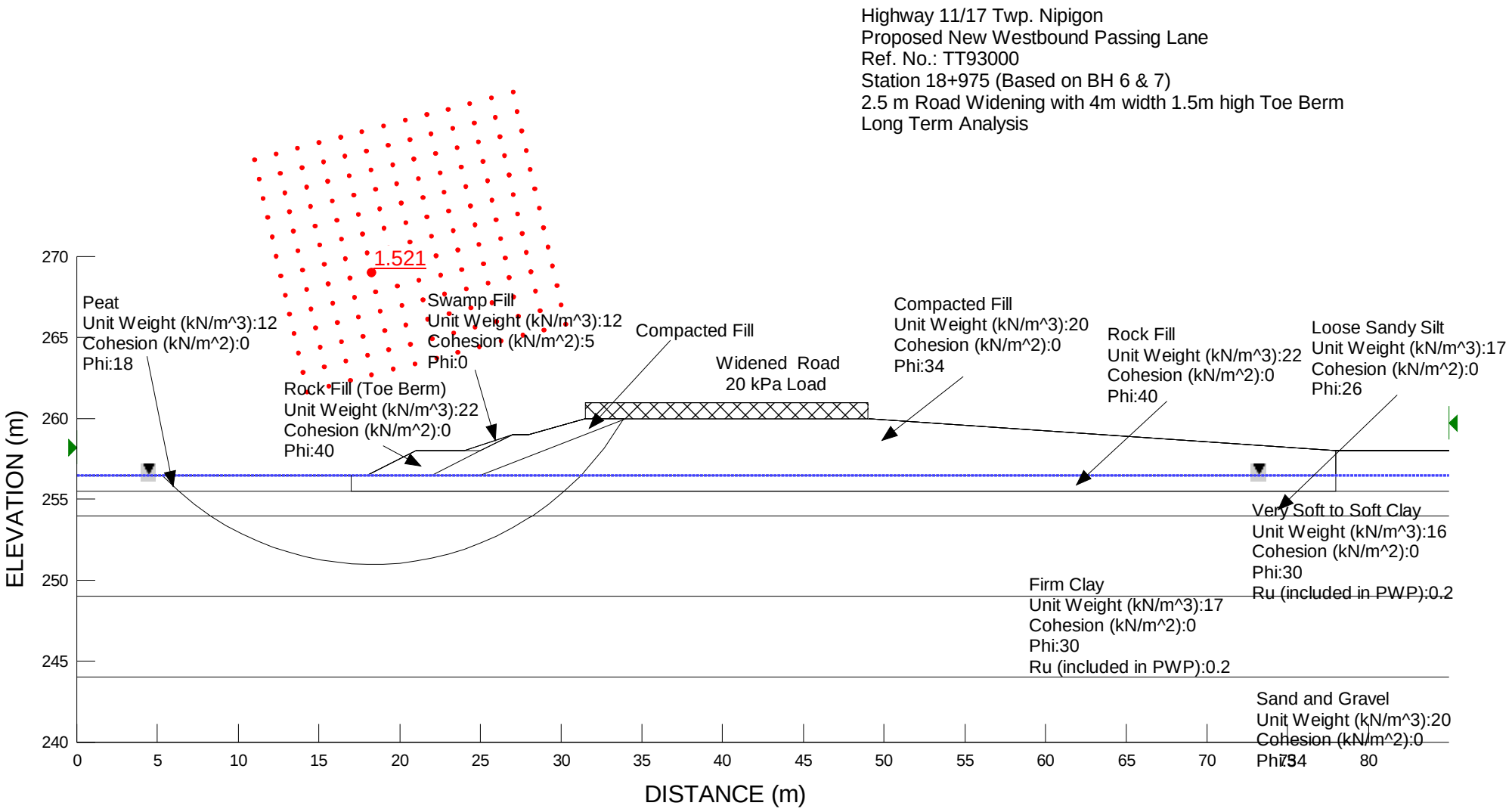


Figure E3.5 Slope Stability Analysis of Section at Station 18+975 - 2.5 m Road Widening with Toe Berm (Long Term Analysis)

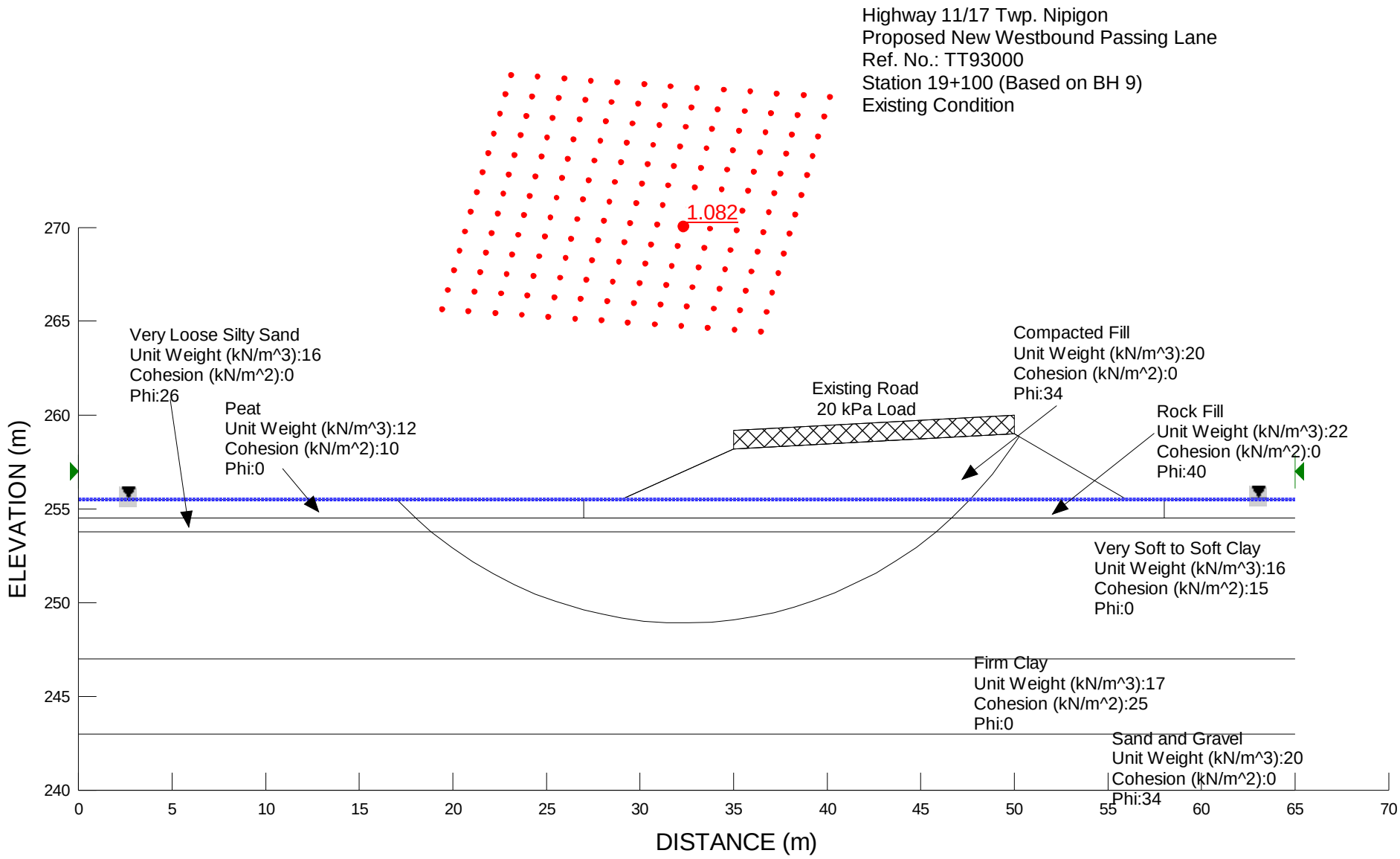


Figure E4.1 Slope Stability Analysis of Section at Station 19+100 (Existing Condition)

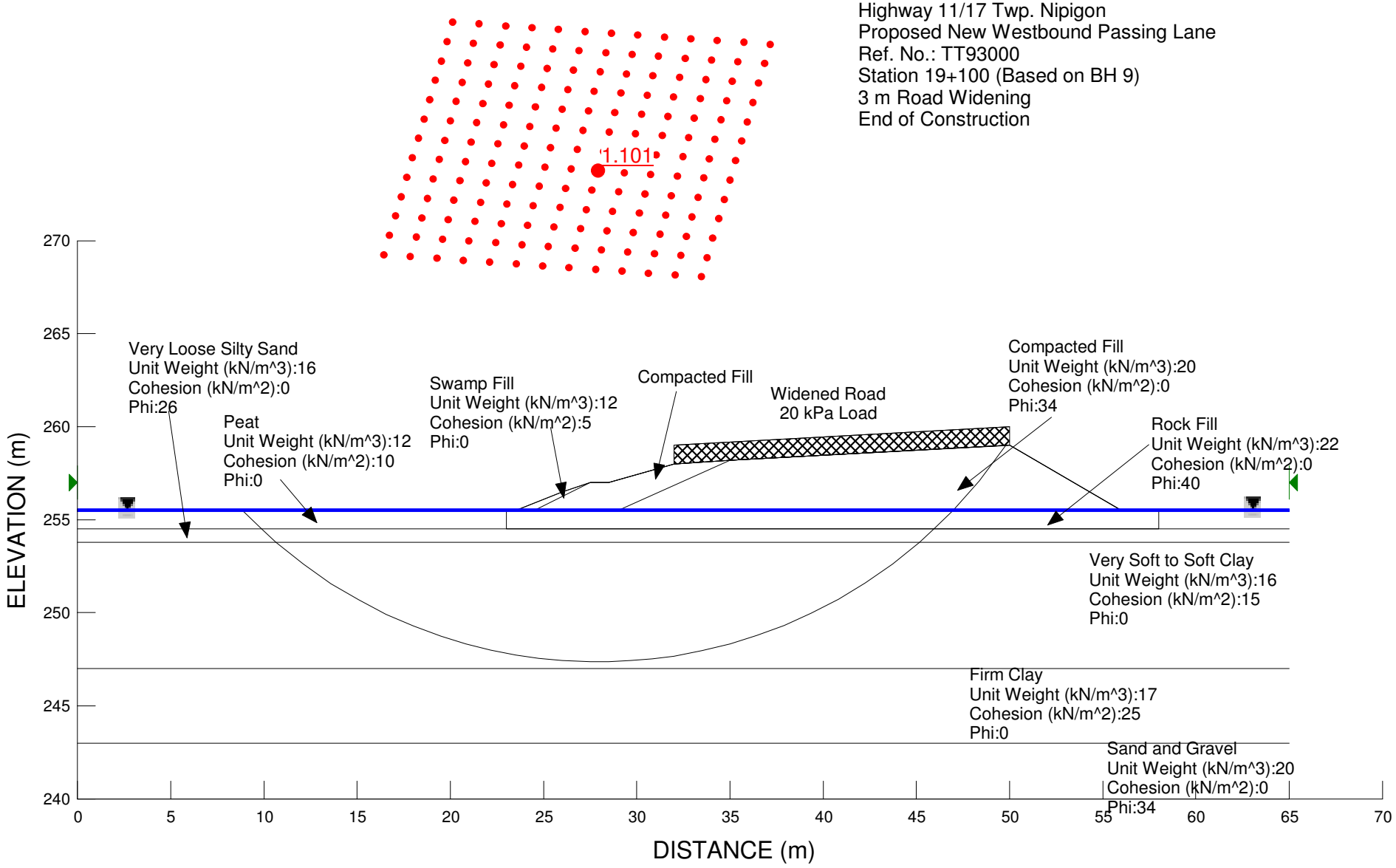


Figure E4.2a Slope Stability Analysis of Section at Station 19+100 with 3.0 m Road Widening (End of Construction)

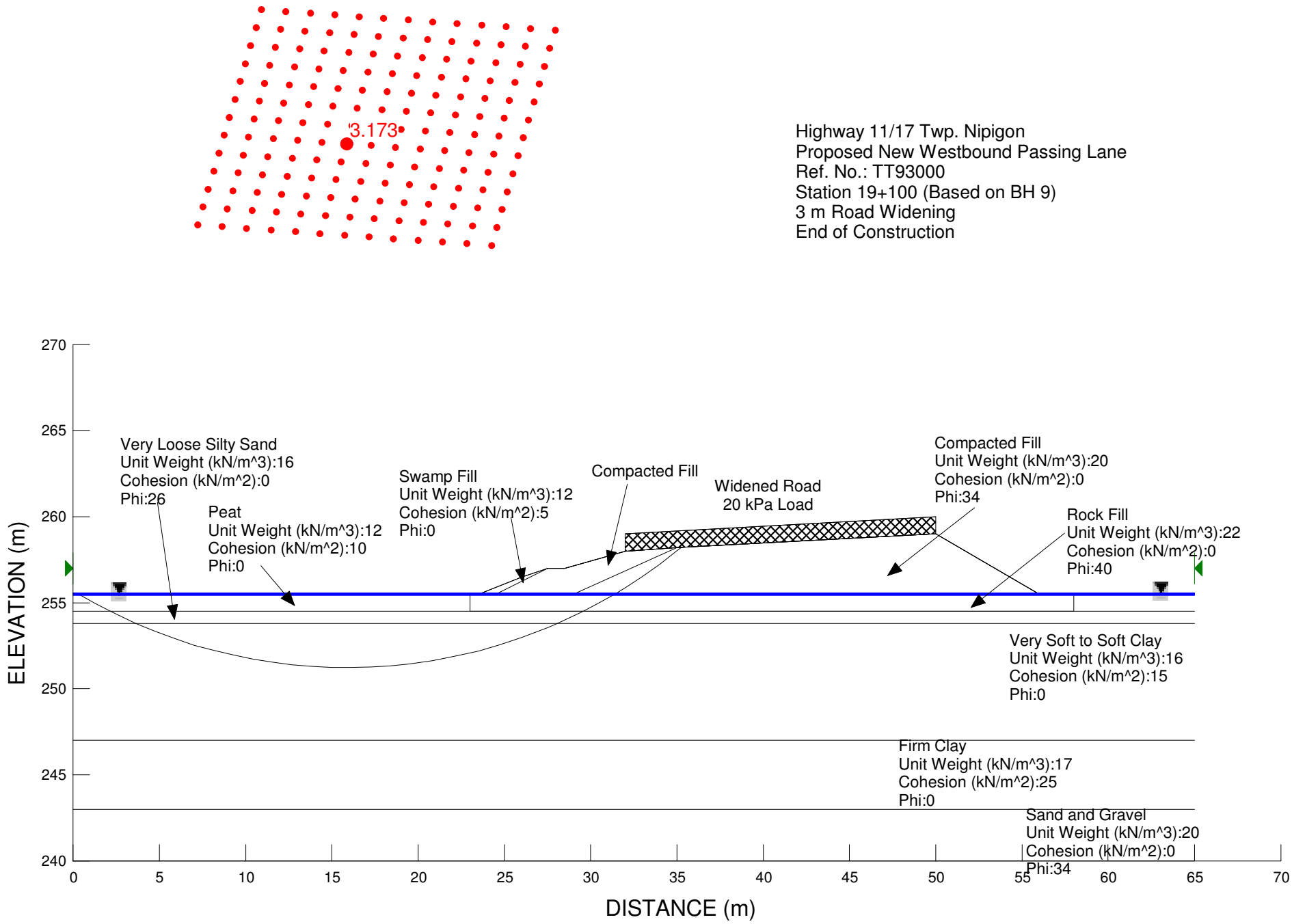


Figure E4.2b Slope Stability Analysis of Section at Station 19+100 with 3.0 m Road Widening (End of Construction - Shallow Slip Surface)

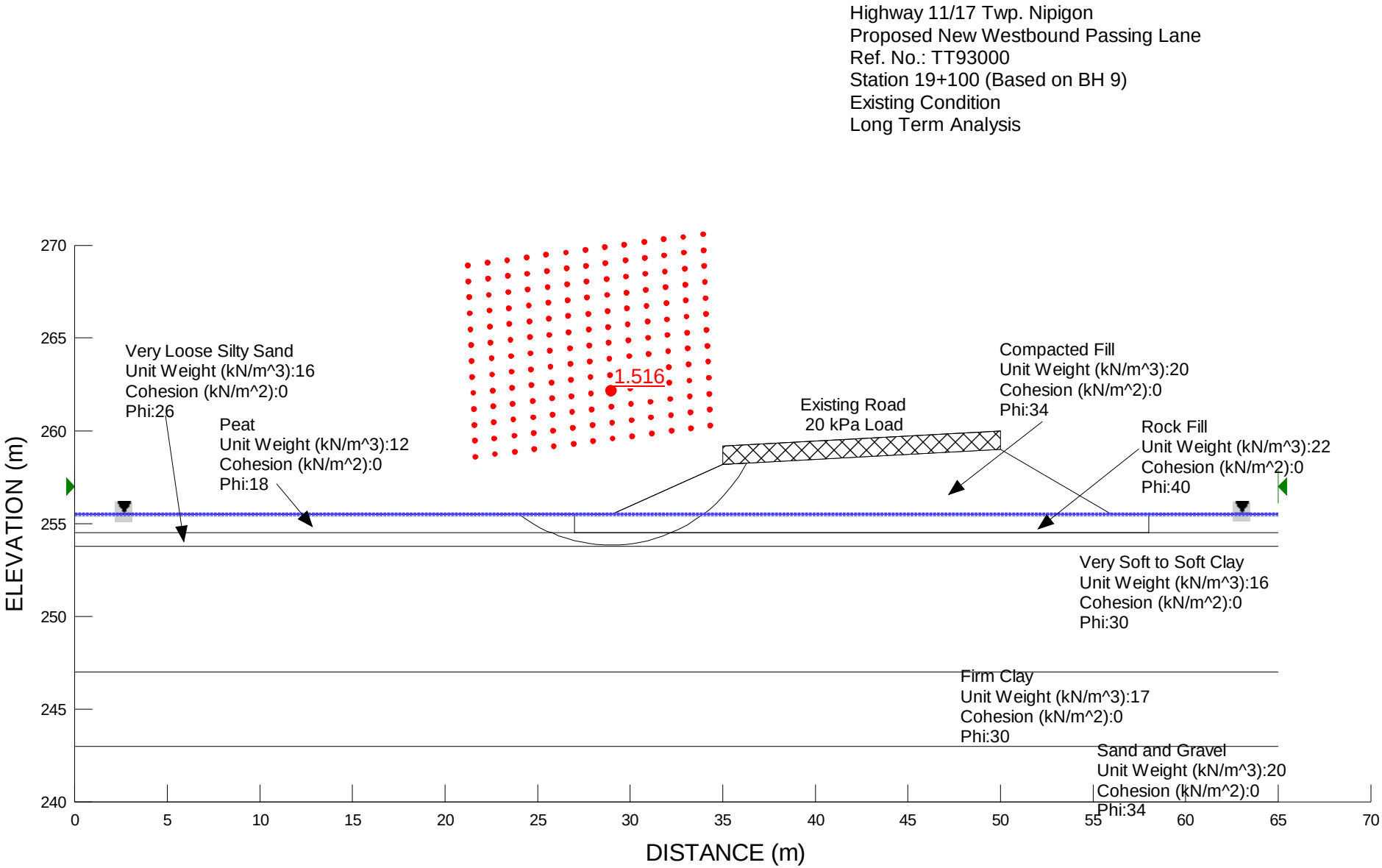


Figure E4.3 Slope Stability Analysis of Section at Station 19+100 (Existing Condition - Long Term Analysis)

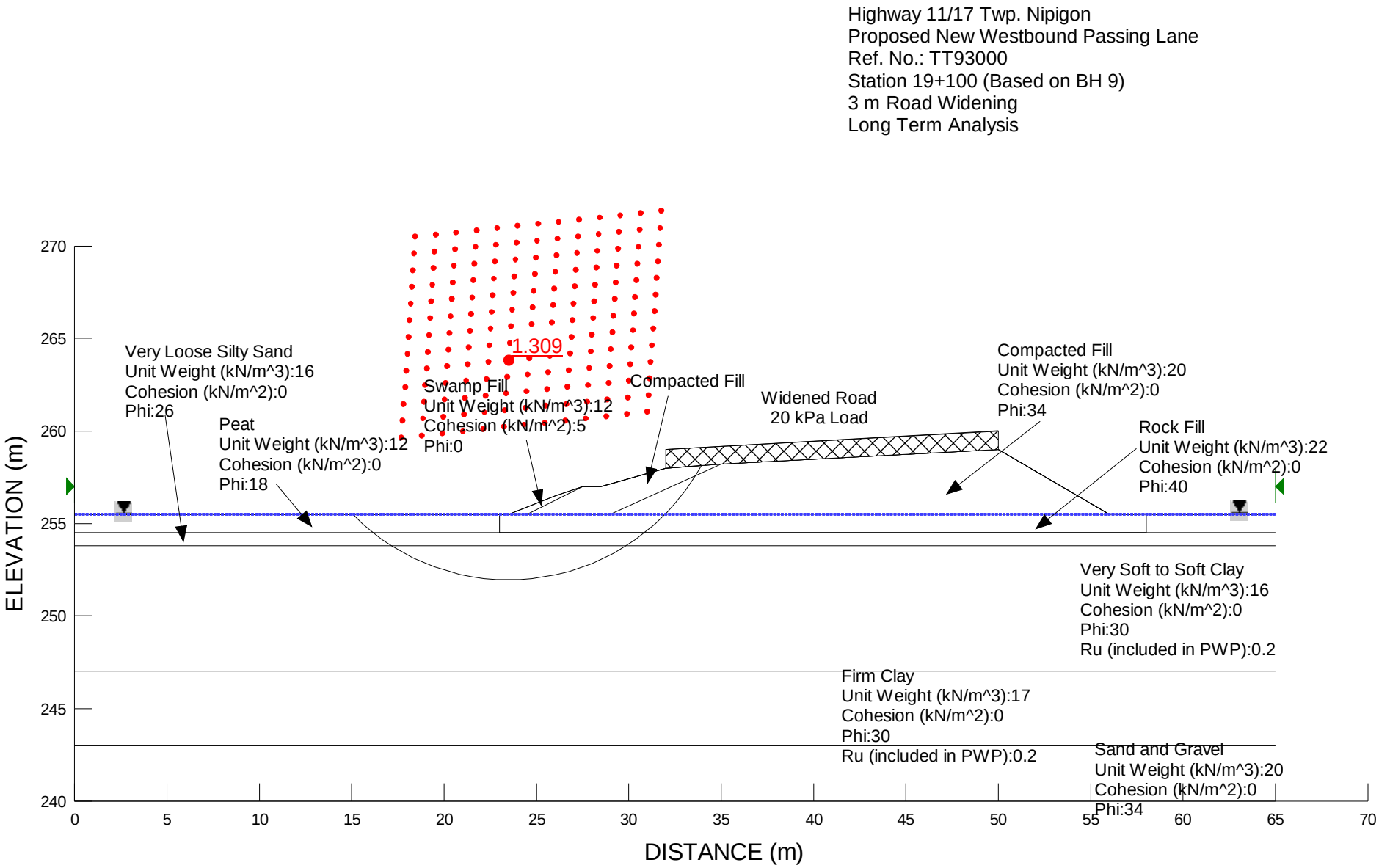


Figure E4.4 Slope Stability Analysis of Section at Station 19+100 with 3.0 m Road Widening (Long Term Analysisi)

APPENDIX F

RESULTS OF SETTLEMENT ANALYSIS

- Results of Immediate Settlement Analysis due to
- Embankment Widening
- Results of Vertical Stress Increment Contours due to
- Embankment Widening
- Results of Calculated Consolidation Settlements

GENERAL PROCEDURES FOR SETTLEMENT ANALYSIS

The general procedures for calculating the settlements are as follows:

Immediate Settlement

1. Set up the finite element mesh for the embankment and the soil layers using the Sigma/W program. The modulus of elasticity (E) and Poisson's Ratio (ν) of the clayey soil layer were selected from the published values (E = 2,000 kN/m^2, ν = 0.35).

2. Perform the finite element analysis. The results are shown in Figure Nos. F1.1 to F1.4. The immediate settlement contours are indicated on the figures.

3. Calculate volume loss over subject sections.

Consolidation Settlement

4. Calculate the compression index (c_c), preconsolidation pressure (p_c), initial void ratio (e_0), final void ratio (e_f), coefficient of consolidation (c_v), swell/rebound index (c_s) and secondary compression index (c_α) from the respective void ratio vs. log pressure curves for the consolidation tests and the creep settlement vs. time curves for the creep test as presented in Figure Nos. B8 to B11 of Appendix B.

5. Set up the finite element mesh for the embankment and the soil layers using the Sigma/W program.

6. Perform the finite element analysis to determine the change in stress at the middle of soft clay layer due to additional load of widened embankment and toe berm. The results are shown in Figure Nos. F2.1 to F2.4. The vertical stress increment contours are indicated on the figures.

7. Calculate the total settlement at each section point as indicated on Figure F2.0 for each analyzed section and loading scheme using Equations 1 to 4 below. The results are presented in Tables F1 to F4 for the as-planned loading scheme and Tables F5 to F8 for the additional 1 m surcharge loading scheme.

In normally consolidated clays, the settlement caused by primary consolidation is calculated based on the following equation (Terzaghi's one-dimensional consolidation theory):

$$\Delta H_{pc} = \frac{C_c H}{1+e_0} \log\left(\frac{p_f}{p_0}\right) \quad \text{.....(1)}$$

where:

ΔH_{pc} : settlement due to primary consolidation (m)
e_o : initial void ratio
p_f : final effective overburden pressure (kN/m^2)
Δp: applied pressure (kN/m^2)
C_c : compression index

and:

$$p_f = p_0 + \Delta p \quad \text{.....(2)}$$

where:

p_o : initial effective overburden pressure (kN/m^2)

Δp: applied pressure (kN/m^2)

In over-consolidated clays where the preconsolidation pressure (p_c) is less than the final effective overburden pressure (p_f), the settlement caused by primary consolidation is calculated based on the following equation:

$$\Delta H_{pc} = \frac{C_s H}{1+e_0}\log\left(\frac{p_c}{p_0}\right) + \frac{C_c H}{1+e_0}\log\left(\frac{p_f}{p_c}\right) \quad \text{.....(3)}$$

where:

p_c : preconsolidation pressure (kN/m^2)

C_s : swell/rebound Index

In over-consolidated clays where the preconsolidation pressure (p_c) is greater than or equal to the final effective overburden pressure (p_f), the settlement caused by primary consolidation is calculated based on the following equation:

$$\Delta H_{pc} = \frac{C_s H}{1+e_0}\log\left(\frac{p_f}{p_0}\right) \quad \text{.....(4)}$$

The parameters e_0, C_c, p_c and C_s are derived from the e – log p curves (Figure Nos. B8 and B9 in Appendix B). The values are listed in the Table 5.1 of Section 5.4.

8. Calculate the degree of consolidation settlement for various time after embankment construction periods at each section point as indicated on Figure F2.0 for each analyzed section and loading scheme using Equations 5 and 6 below. The results are presented in Tables F1 to F4 for the as-planned loading scheme and Tables F5 to F8 for the additional 1 m surcharge loading scheme.

The degree of primary consolidation, $U\%$, is calculated based on the following equation:

$$\frac{U\%}{100} = \frac{\left(\frac{4T_v}{\pi}\right)^{0.5}}{\left[1+\left(\frac{4T_v}{\pi}\right)^{2.8}\right]^{0.179}} \quad \text{.....(5)}$$

where:

$U\%$: average degree of consolidation

T_v : dimensionless time factor

and:

$$T_v = \frac{c_v t}{H_{dr}^2} \quad \text{.....(6)}$$

where:

c_v : coefficient of consolidation (m^2/day)

t : time (days)

H_{dr} : soil drainage distance (m)

The ranges of values for c_v are listed in Table 5.1 of Section 5.4.

9. If the degree of consolidation settlement for the time period after embankment construction is equal to or greater than about 90%, calculate the secondary settlement for each analyzed section and loading scheme using Equation 7 below. The results are presented in Tables F1 to F4 for the as-planned loading scheme and Tables F5 to F8 for the additional 1 m surcharge loading scheme.

The settlement due to creep is calculated by using the following equation:

$$\Delta H_{cs} = \frac{C_\alpha}{(1+e_p)} H \log\left(\frac{t_2}{t_1}\right) \quad .....(7)$$

where:

ΔH_{cs}: creep settlement at time t_2 (m)

H : thickness of existing soil (m)

t_1 : time at completion of embankment construction (years)

t_2 : time after completion of embankment construction (years)

C_α : creep deformation coefficient or secondary compression index

e_p : void ratio at the end of primary consolidation

The parameter C_α was derived from the slopes of the secondary linear portion of the e – log t curves (Figure Nos. B10 and B11 in Appendix B) normalized to e_p using the final void ratios from the respective primary consolidation tests (e_f) as an estimate of e_p. The values are listed in Table 5.1 of Section 5.4.

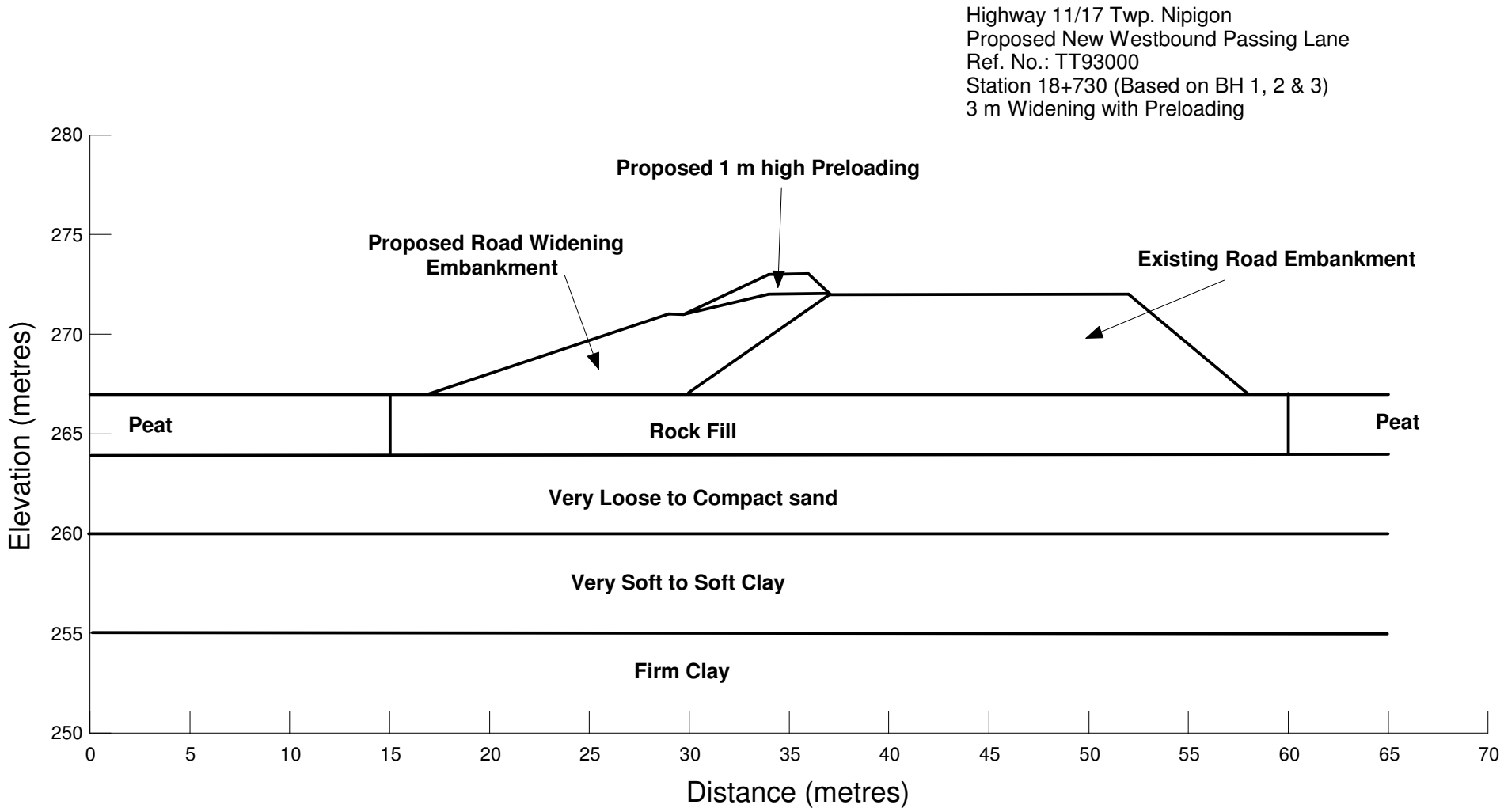


Figure F0.1 Embankment Configuration and Subsoil Profiles - Road Widening with 1 m Preloading at Station 18+730

Highway 11/17 Twp. Nipigon
Proposed New Westbound Passing Lane
Ref. No.: TT93000
Station 18+875 (Based on BH 4)
3.5 m Widening with 5 m Extended Berm

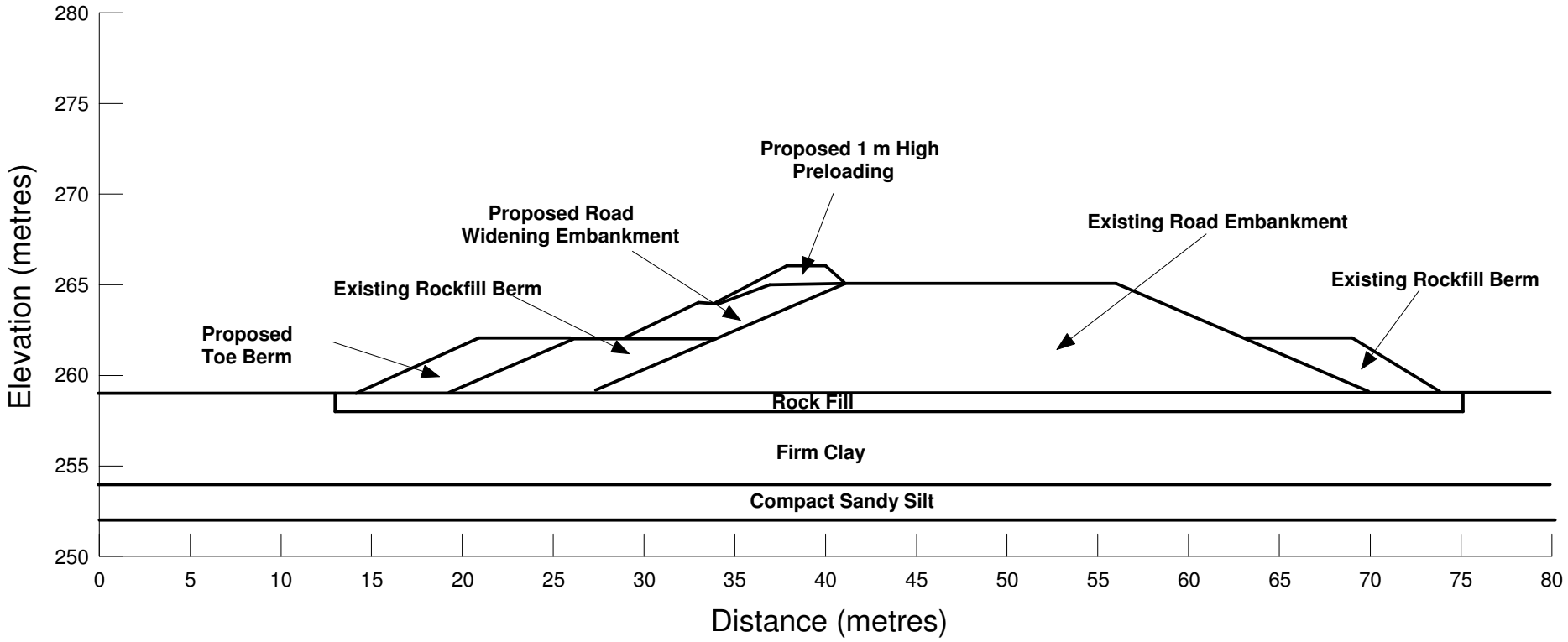


Figure F0.2 Embankment Configuration and Soil Profiles - Road Widening with 1 m Preloading at Station 18+875

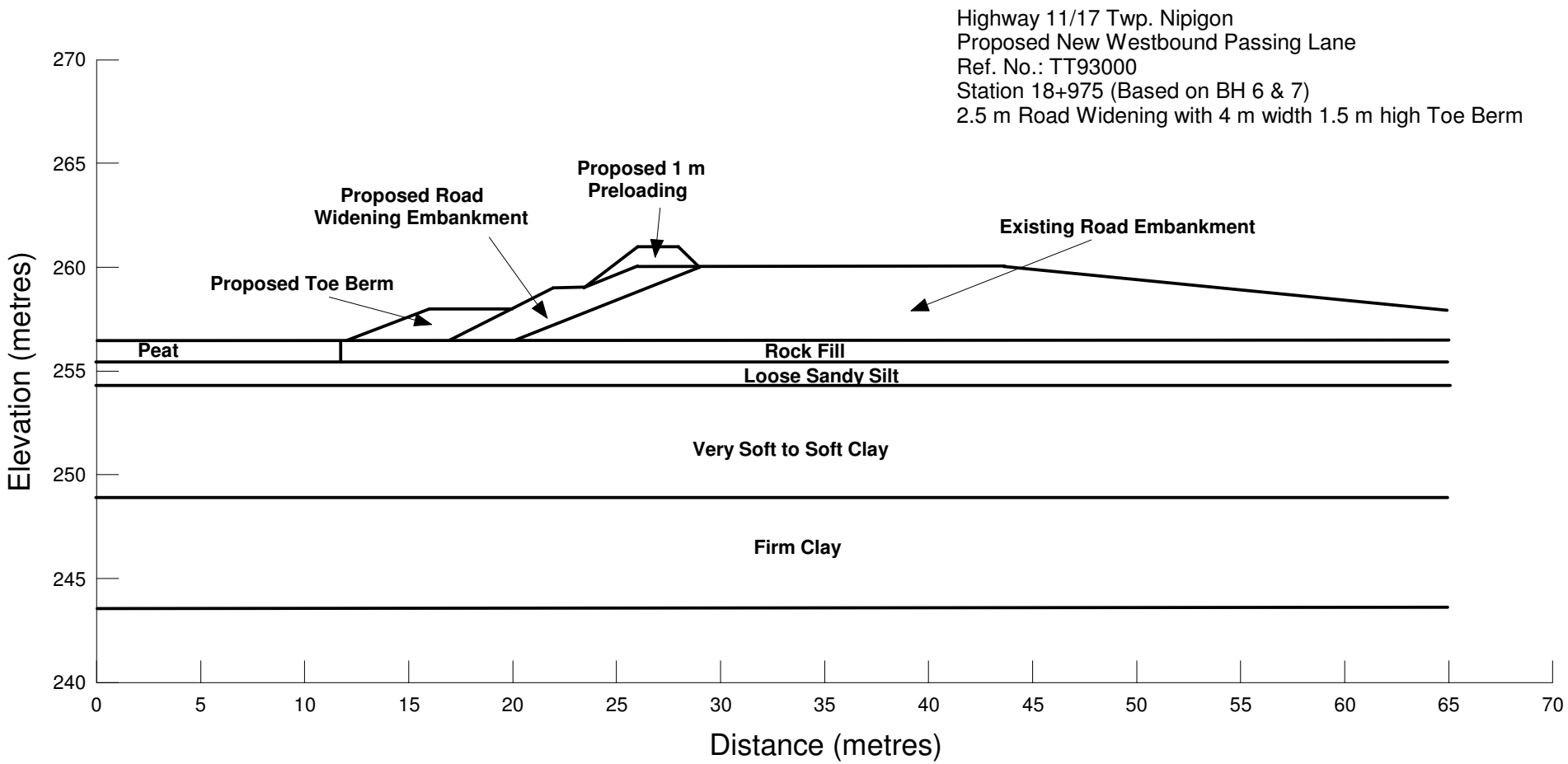


Figure F0.3 Embankment Configuration and Soil Profiles - Road Widening with 1 m Preloading at Station 18+975

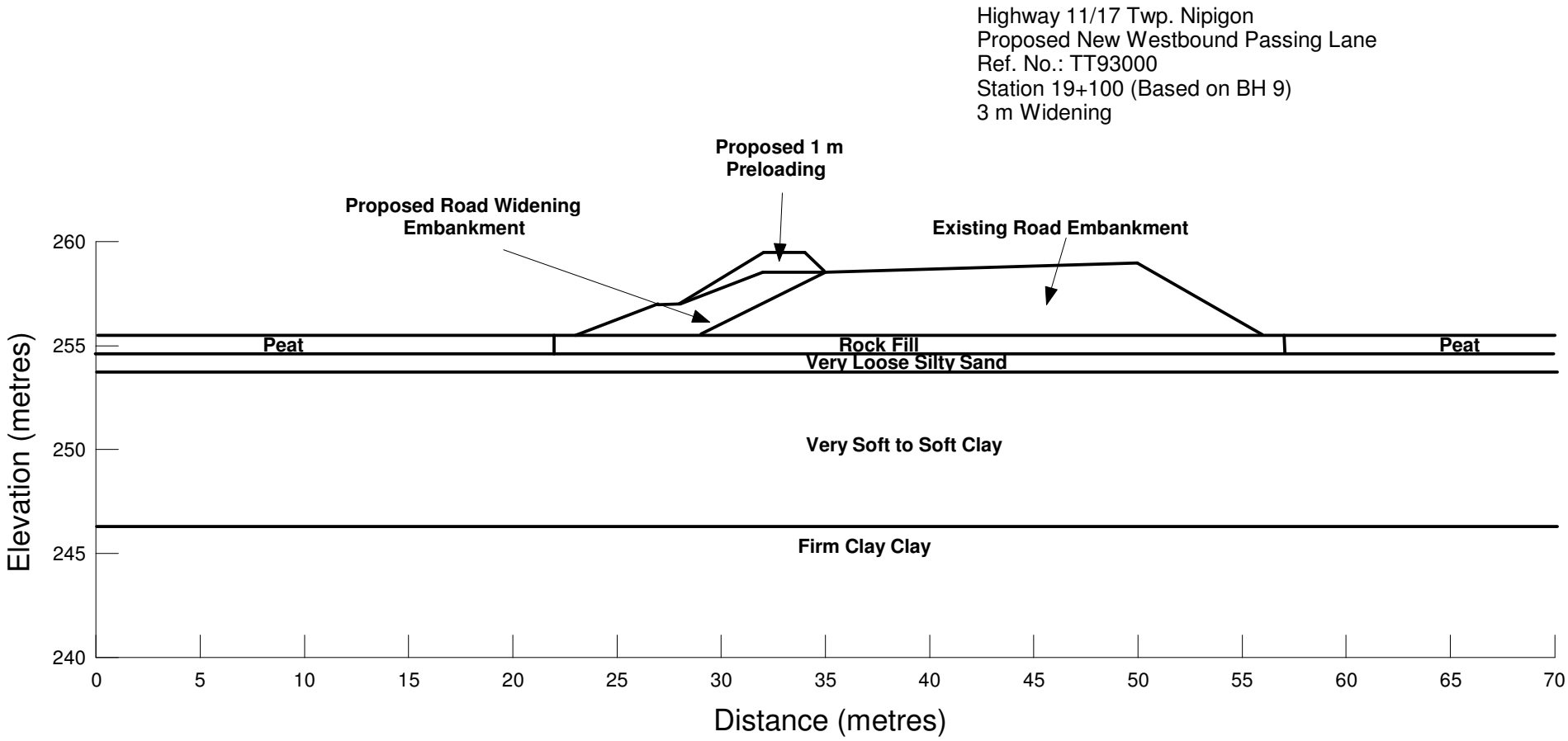


Figure F0.4 Embankment Configuration and Soil Profiles - Road Widening with 1 m Preloading at Station 19+100

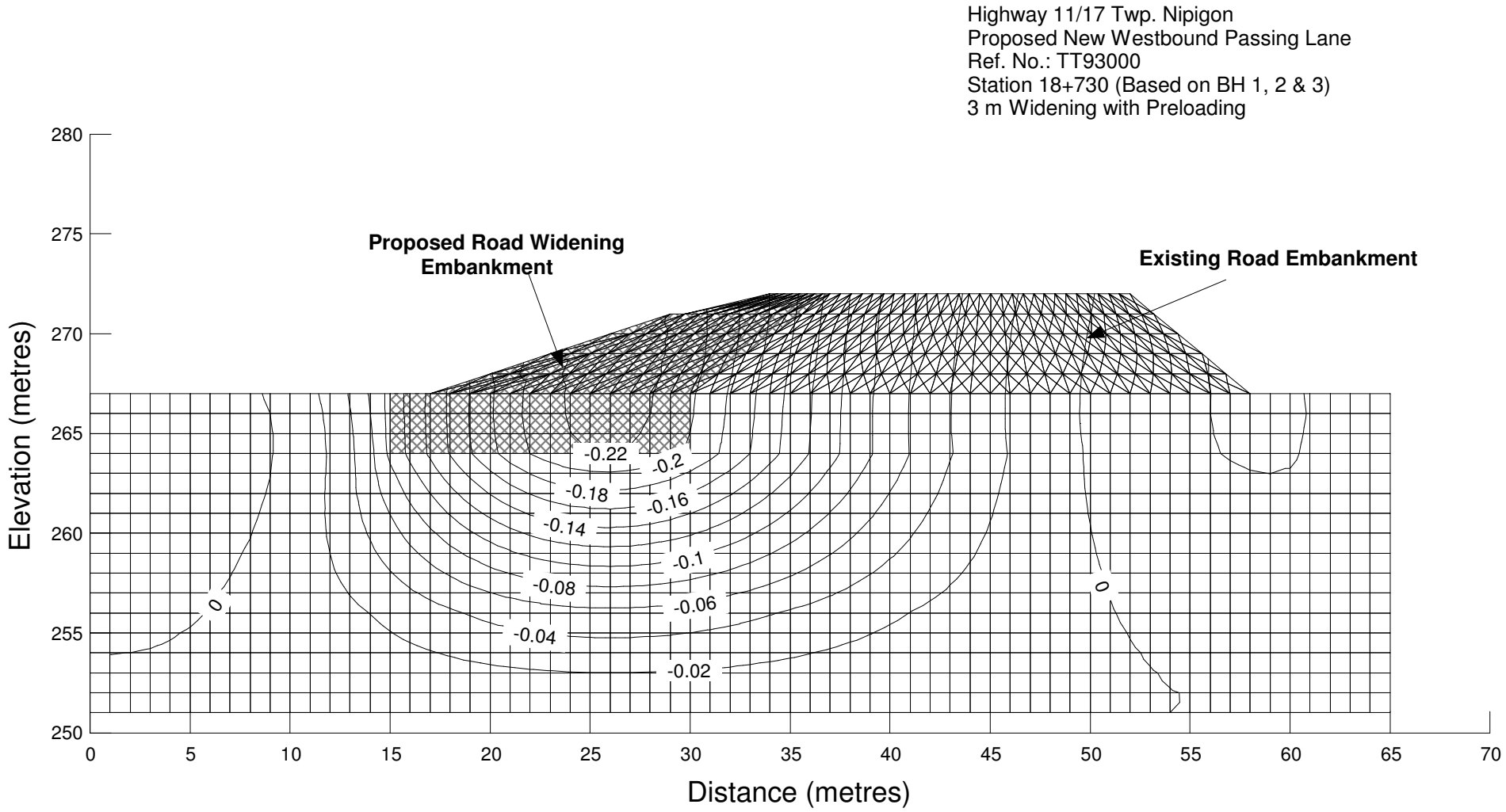


Figure F1.1 Immediate Settlement Contour (m) - Road Widening at Station 18+730

Highway 11/17 Twp. Nipigon
Proposed New Westbound Passing Lane
Ref. No.: TT93000
Station 18+875 (Based on BH 4)
3.5 m Widening with 5 m Extended Berm

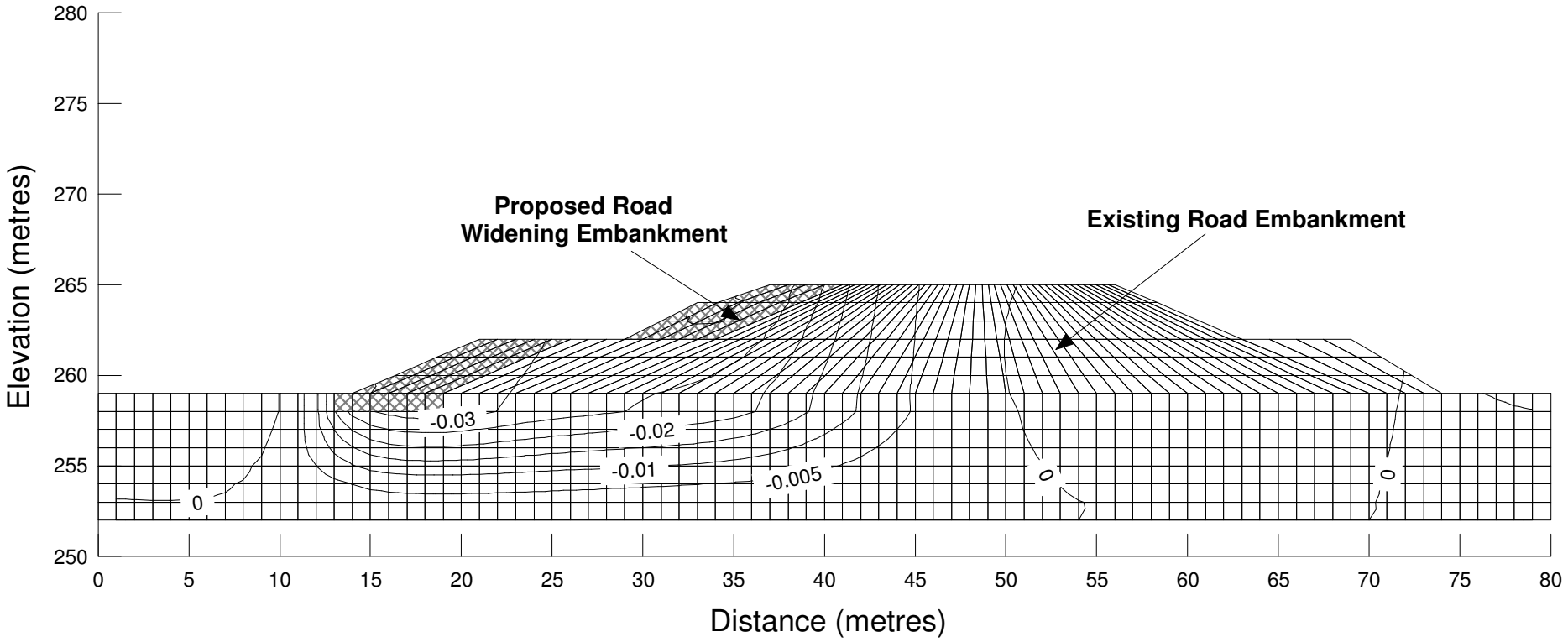


Figure F1.2 Immediate Settlement Contour (m) - Road Widening at Station 18+875

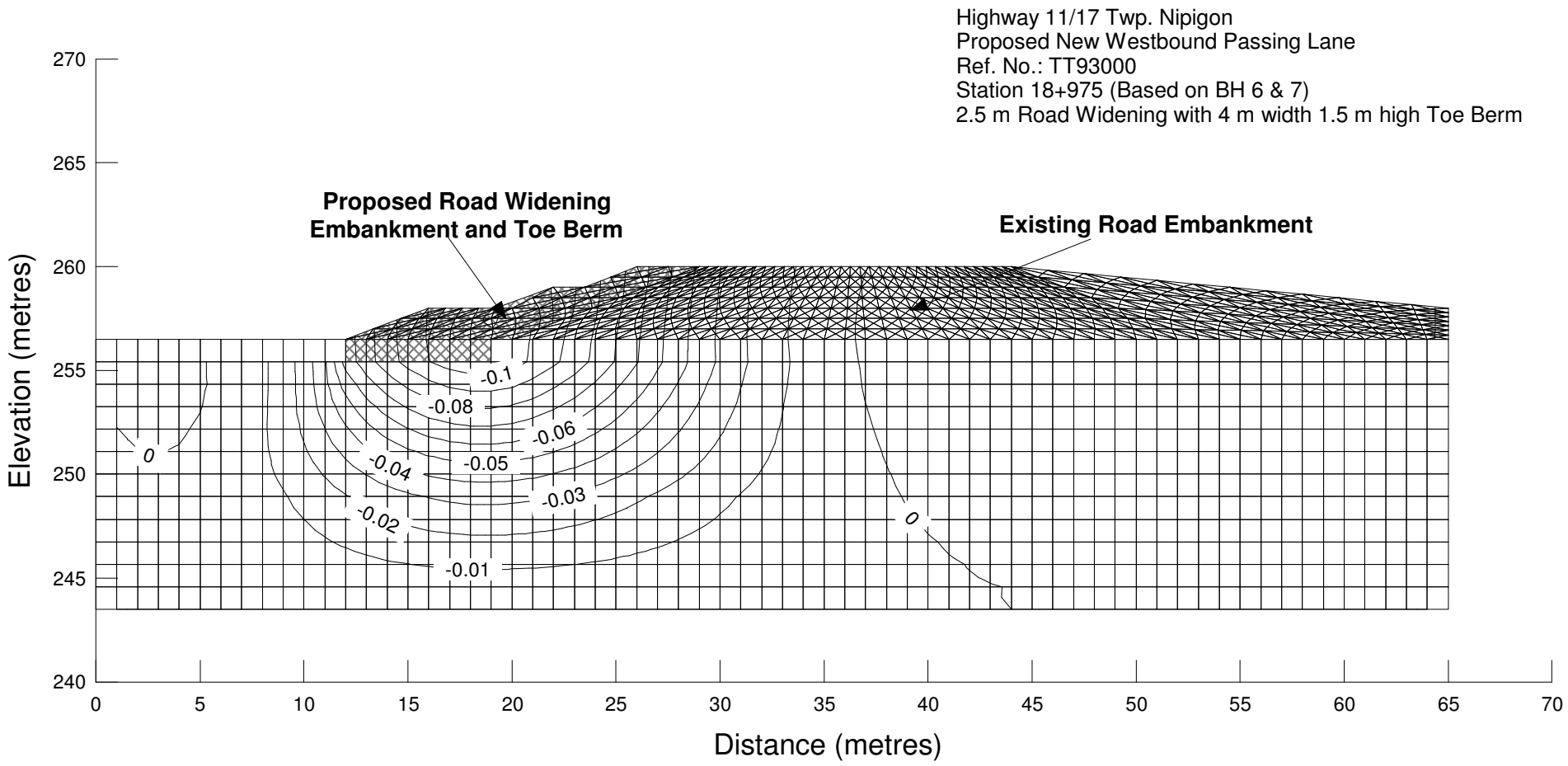


Figure F1.3 Immediate Settlement Contour (m) - Road Widening at Station 18+975

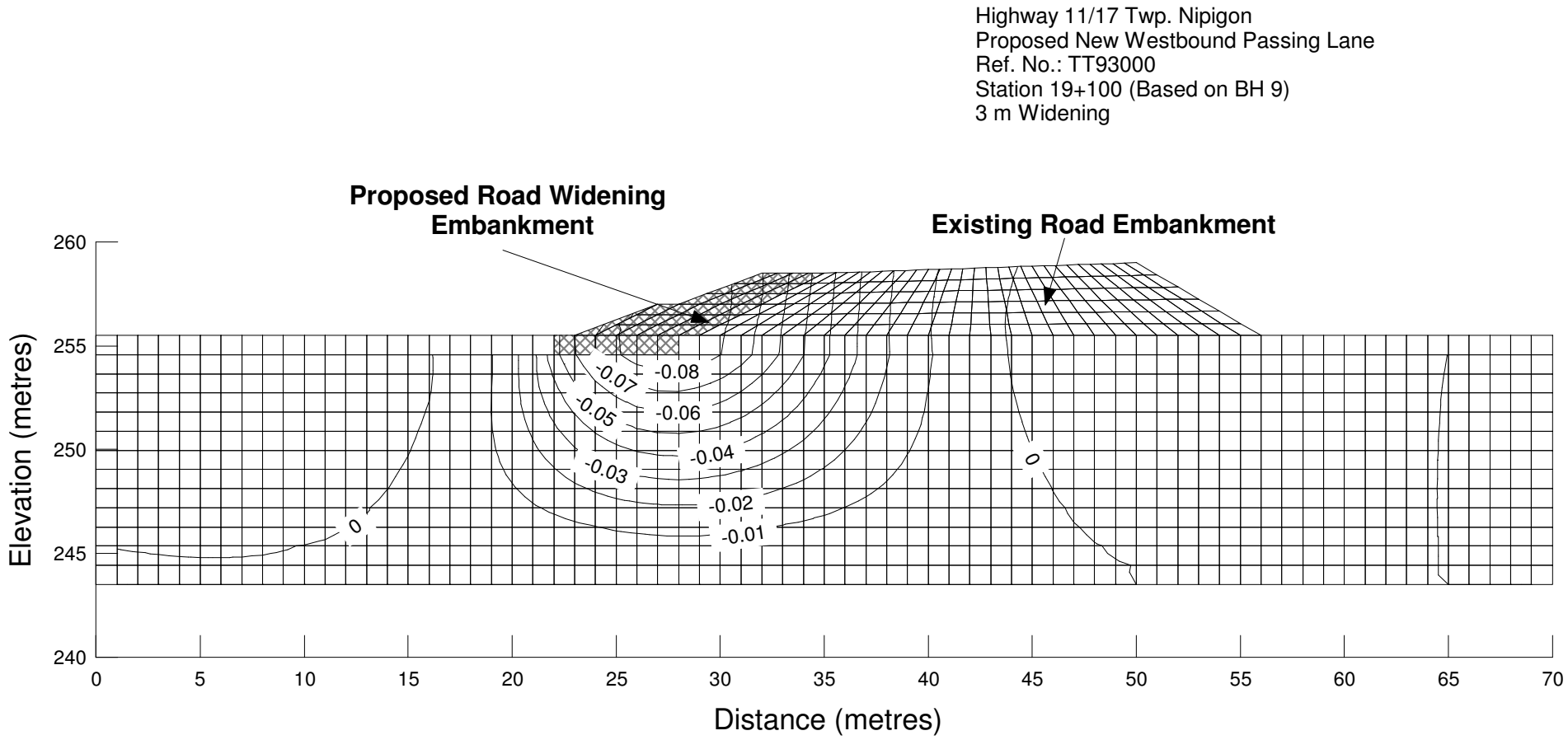


Figure F1.4 Immediate Settlement Contour (m) - Road Widening at Station 19+100

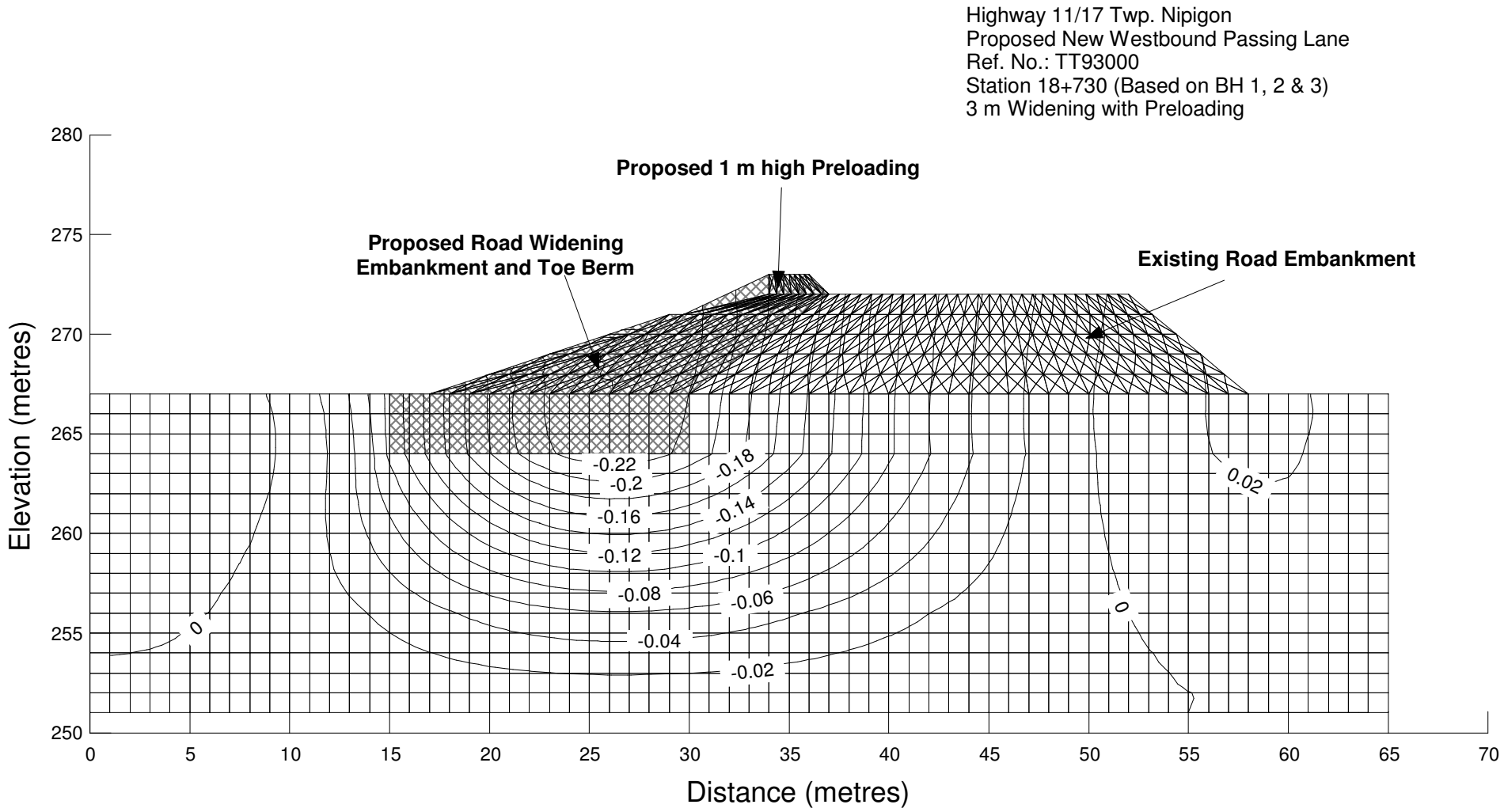


Figure F2.1 Immediate Settlement Contour (m) - Road Widening with 1 m Preloading at Station 18+730

Highway 11/17 Twp. Nipigon
Proposed New Westbound Passing Lane
Ref. No.: TT93000
Station 18+875 (Based on BH 4)
3.5 m Widening with 5 m Extended Berm

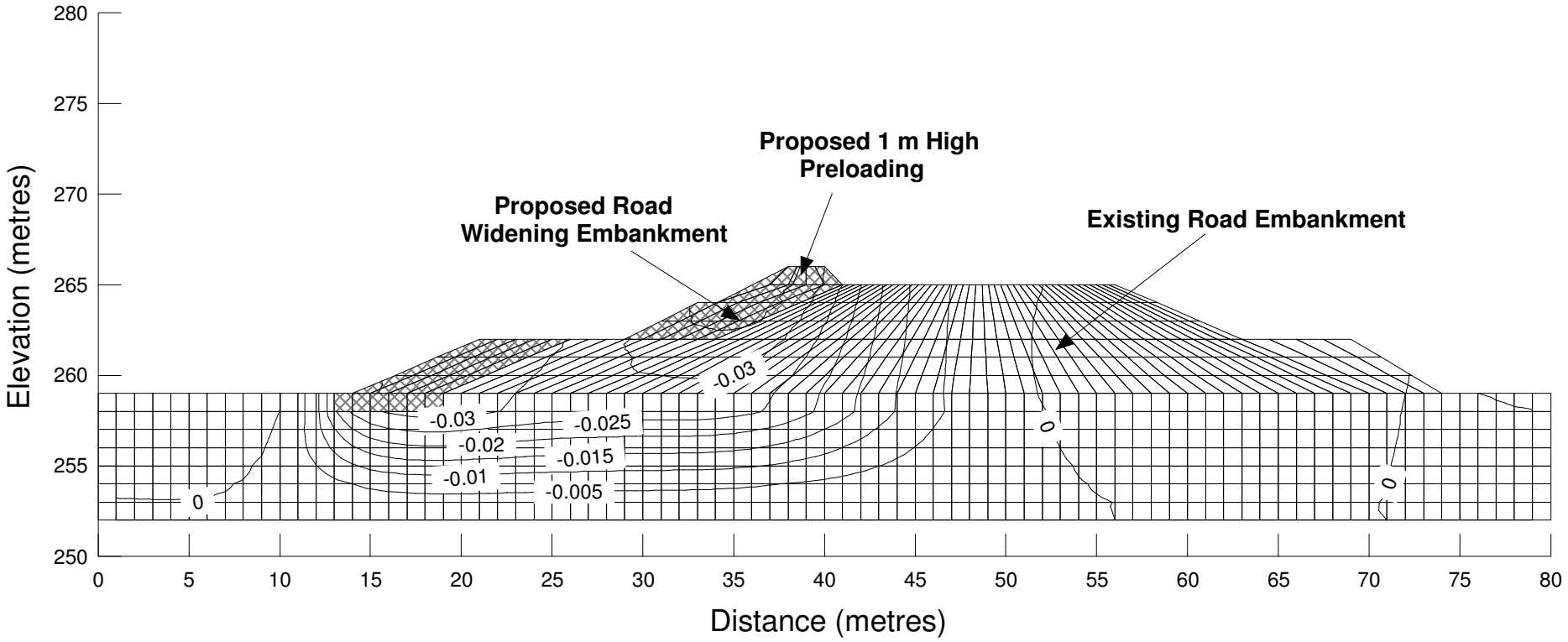


Figure F2.2 Immediate Settlement Contour (m) - Road Widening with 1 m Preloading at Station 18+875

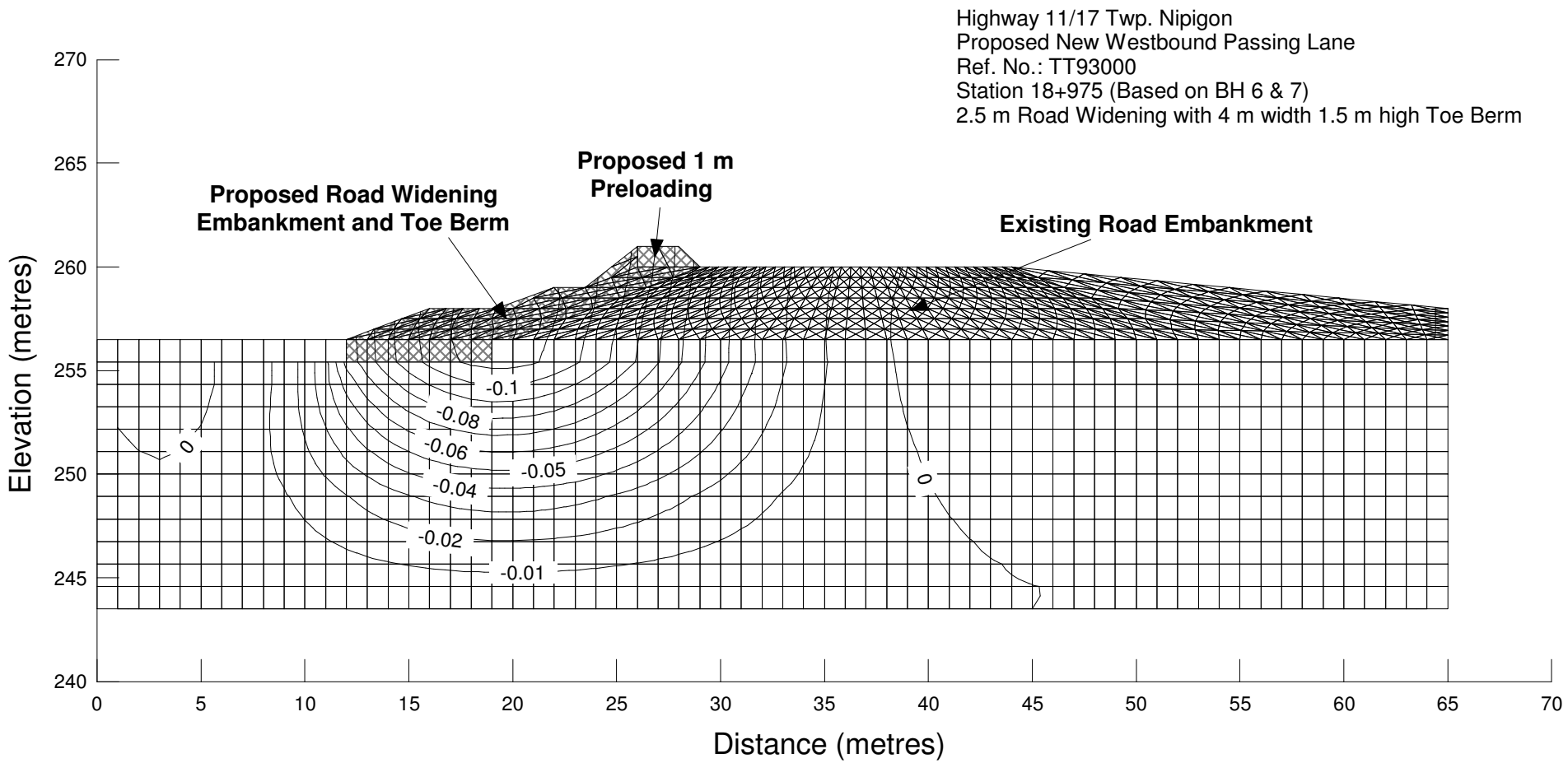


Figure F2.3 Immediate Settlement Contour (m) - Road Widening with 1 m Preloading at Station 18+975

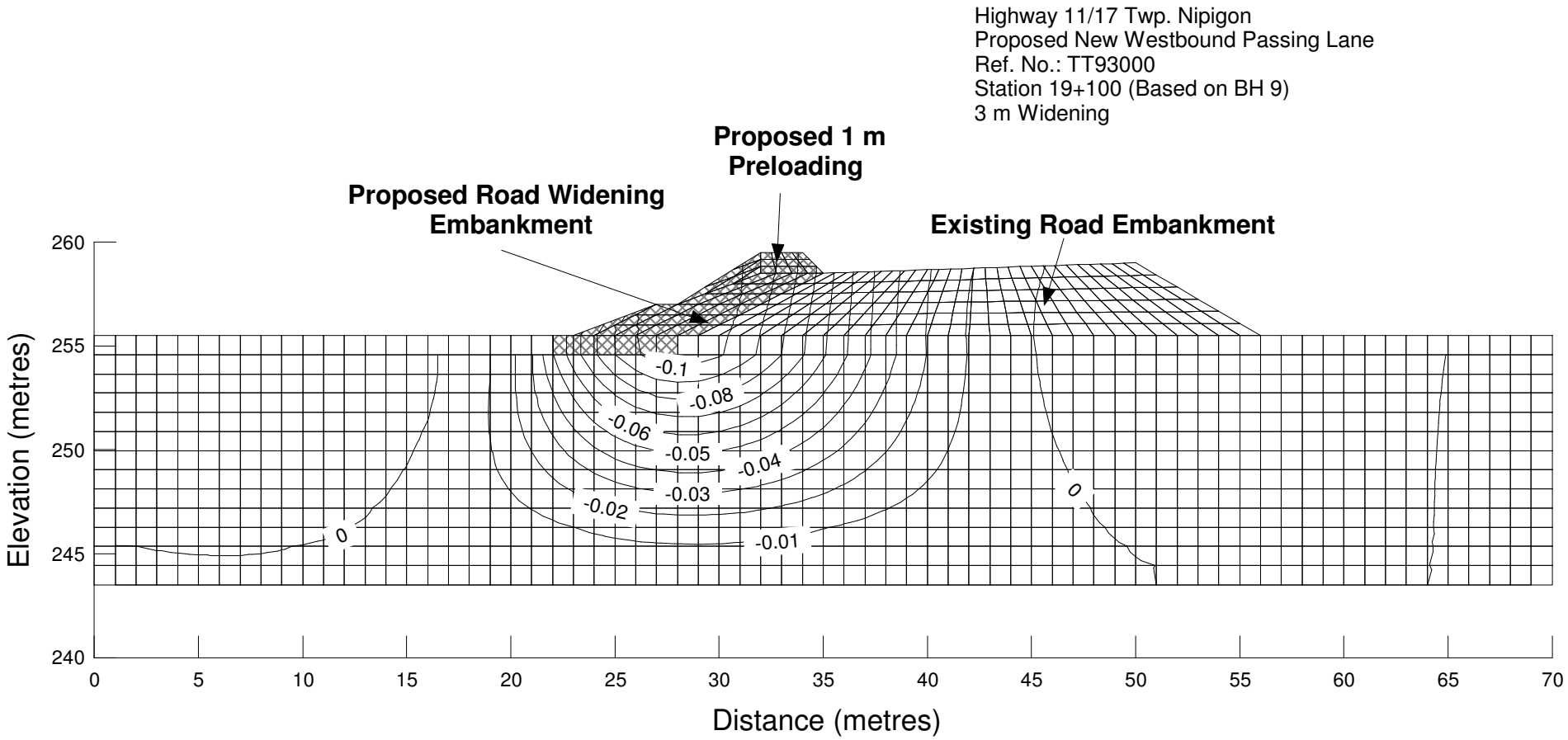


Figure F2.4 Immediate Settlement Contour (m) - Road Widening with 1 m Preloading at Station 19+100

Highway 11/17 Twp. Nipigon
Proposed New Westbound Passing Lane
Ref. No.: TT93000

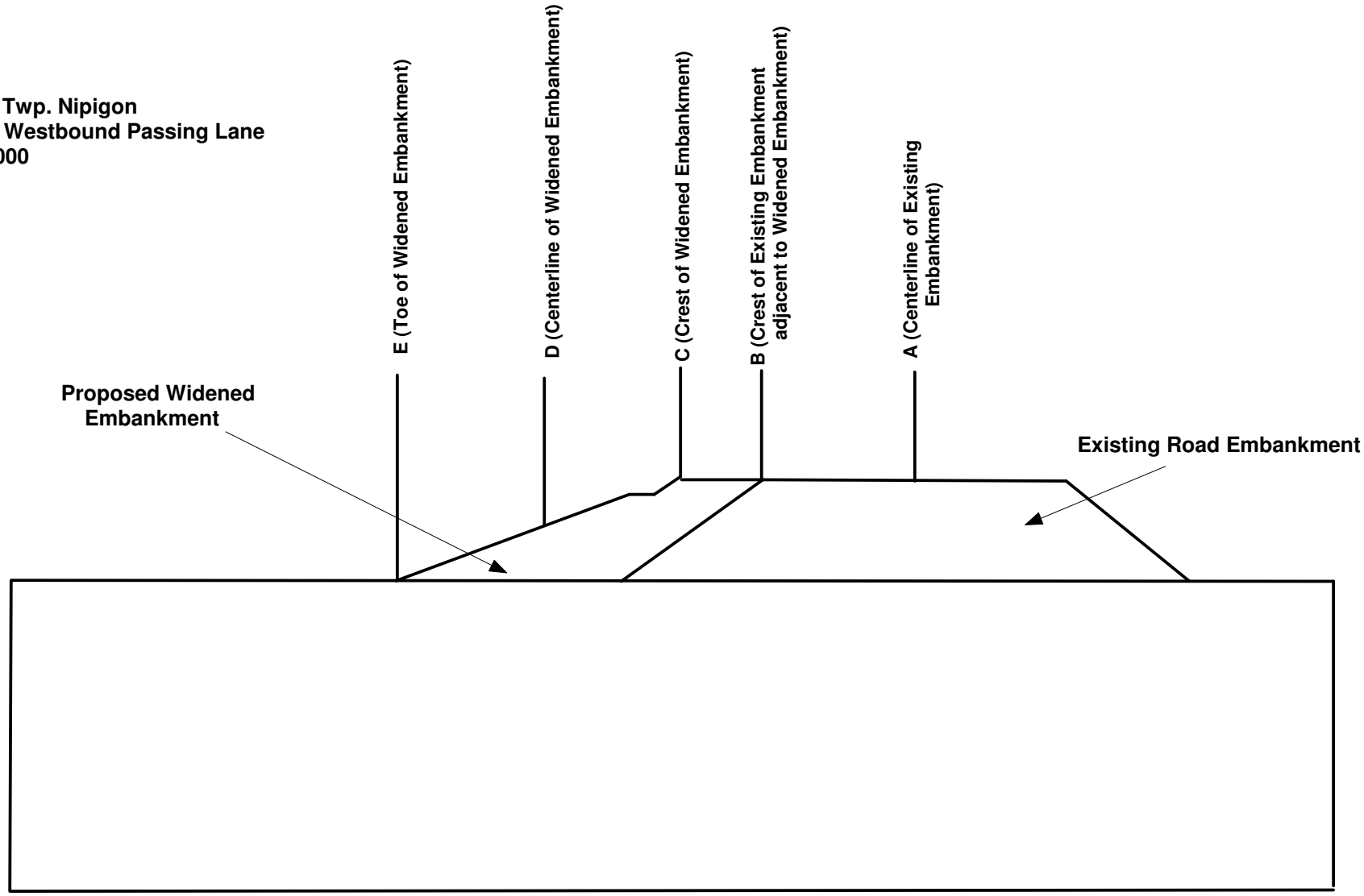


Figure F3.0 Settlement Analysis Points

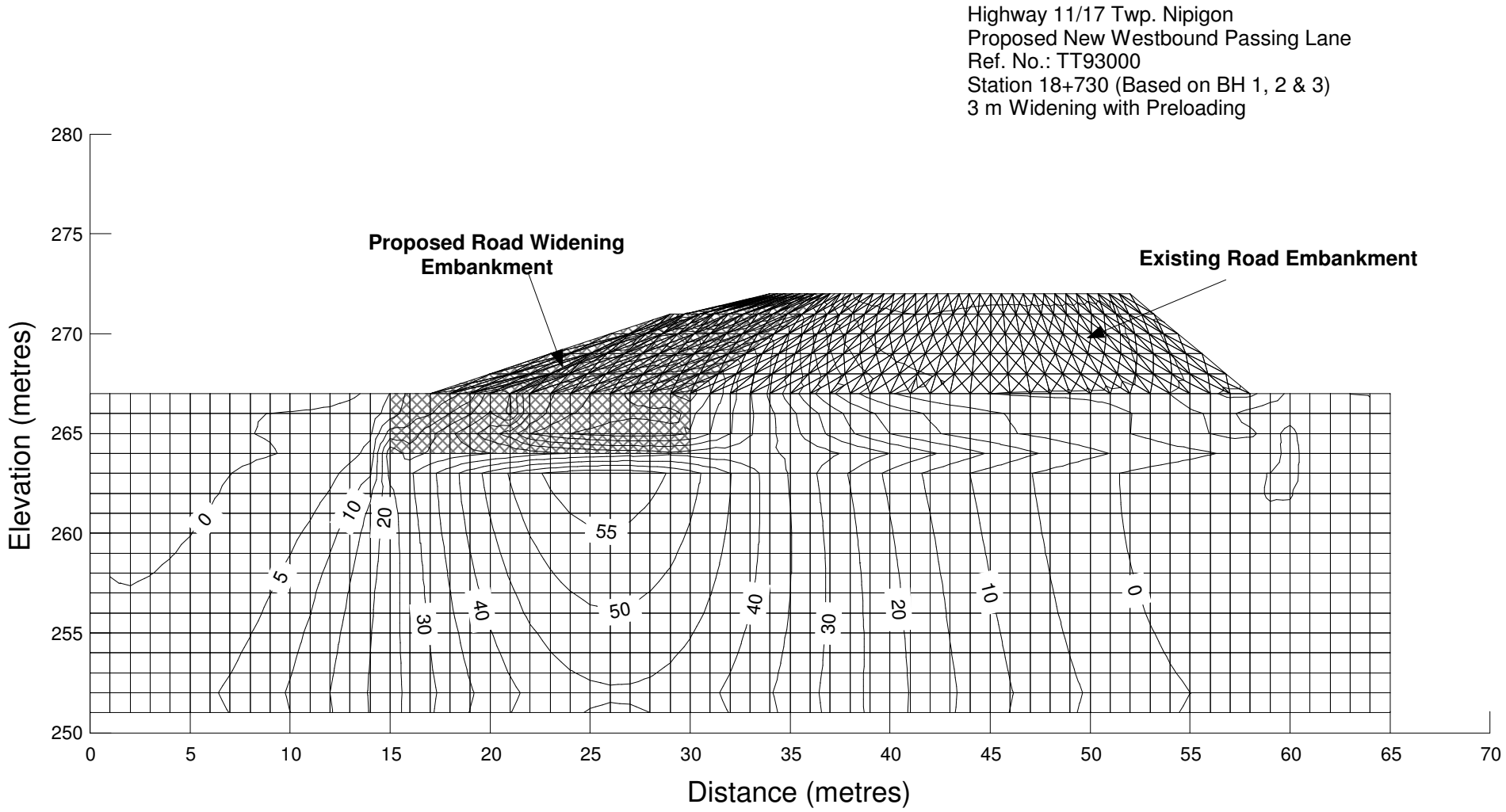


Figure F3.1 Vertical Stress Increment Contour (kPa) - Road Widening at Station 18+730

Highway 11/17 Twp. Nipigon
Proposed New Westbound Passing Lane
Ref. No.: TT93000
Station 18+875 (Based on BH 4)
3.5 m Widening with 5 m Extended Berm

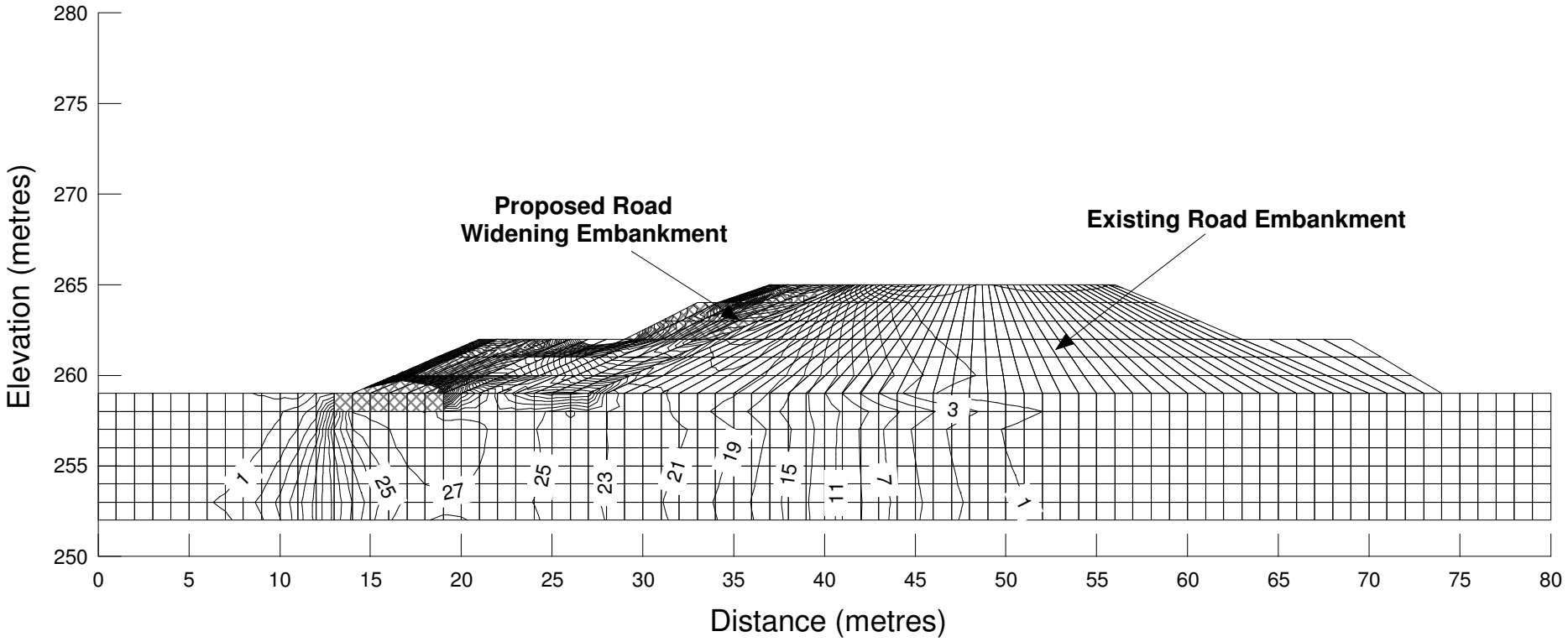


Figure F3.2 Vertical Stress Increment Contour (kPa) - Road Widening at Station 18+875

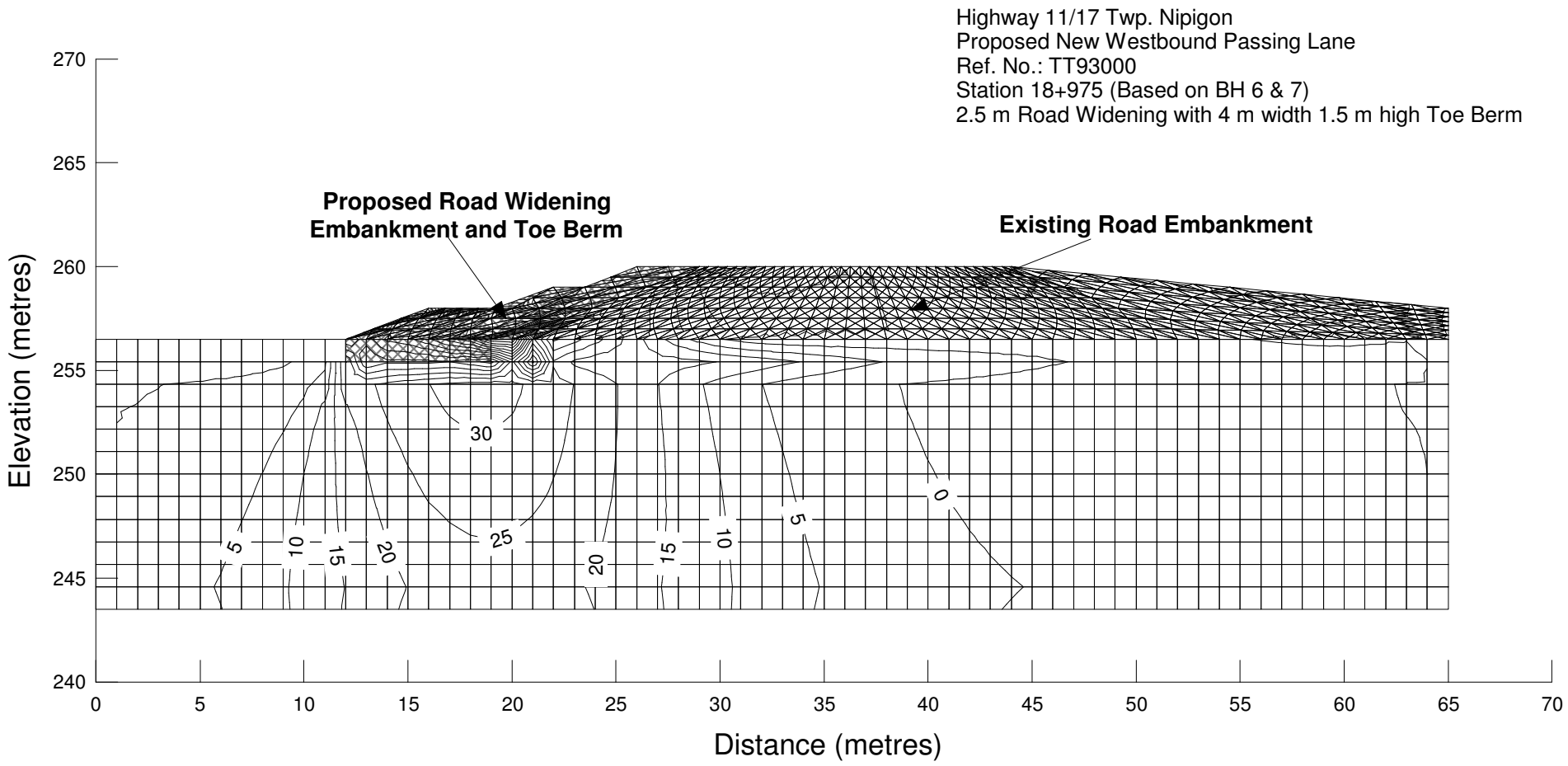


Figure F3.3 Vertical Stress Increment Contour (kPa) - Road Widening at Station 18+975

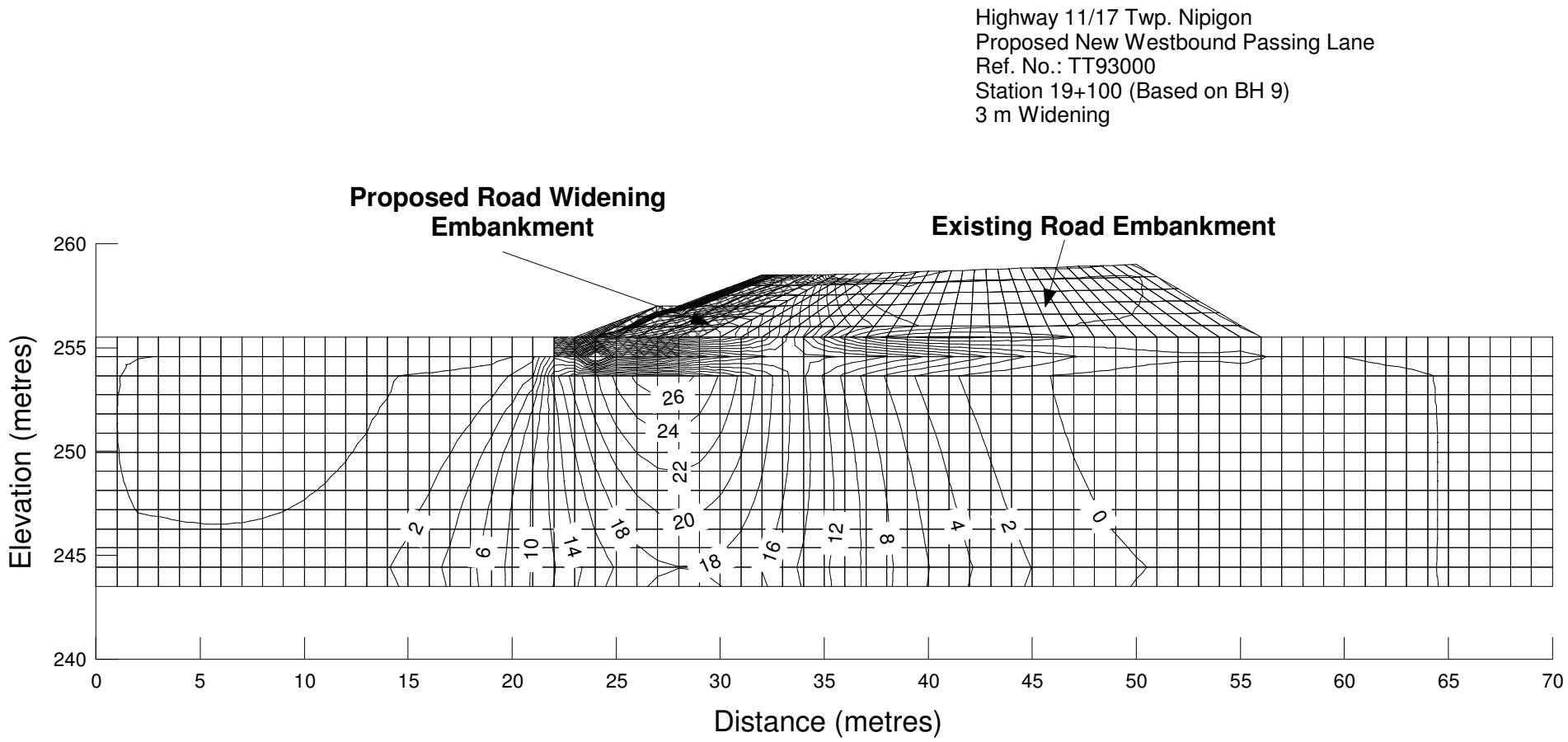


Figure F3.4 Vertical Stress Increment Contour (kPa) - Road Widening at Station 19+100

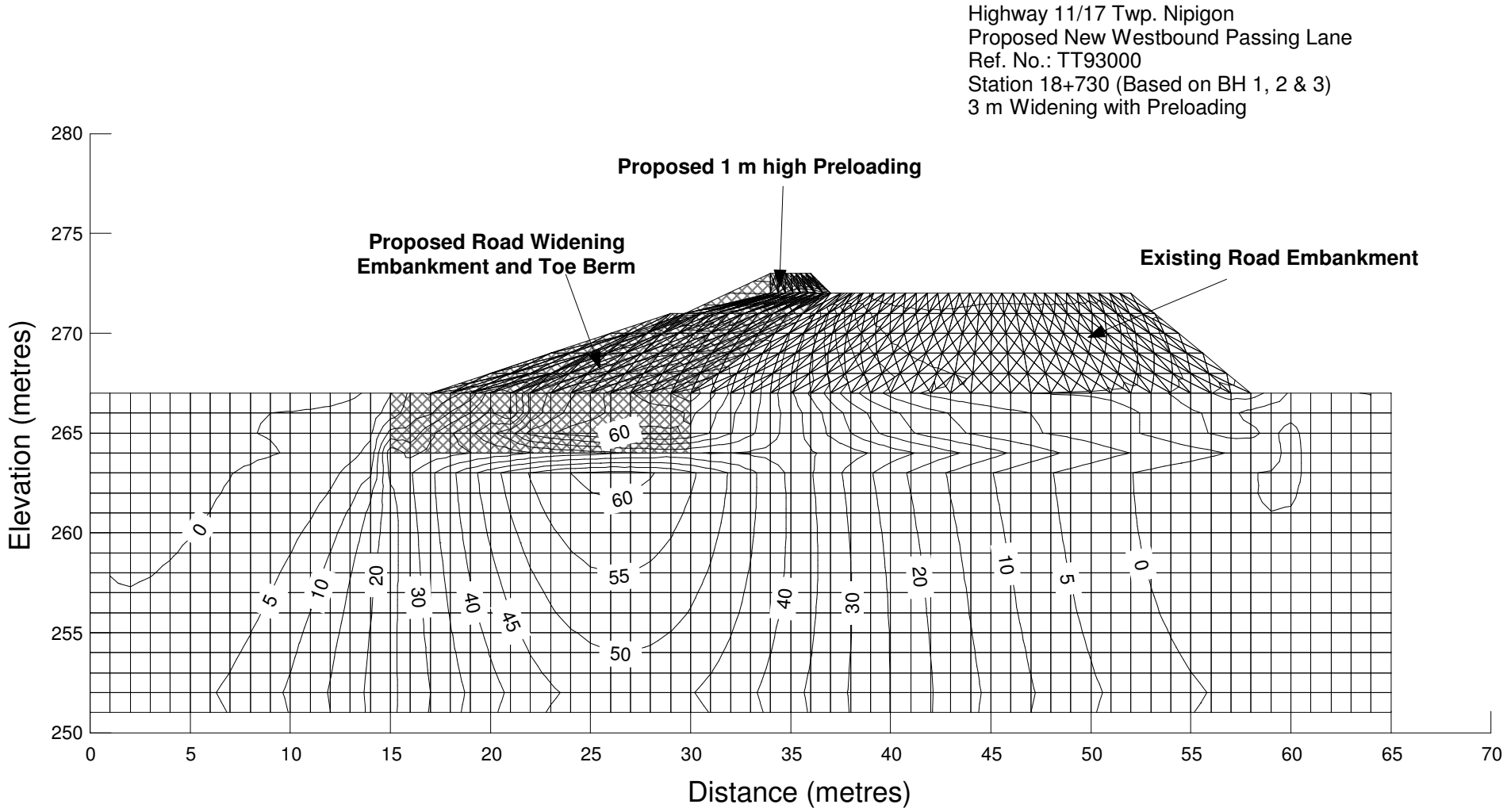


Figure F4.1 Vertical Stress Increment Contour (kPa) - Road Widening with 1 m Preloading at Station 18+730

Highway 11/17 Twp. Nipigon
Proposed New Westbound Passing Lane
Ref. No.: TT93000
Station 18+875 (Based on BH 4)
3.5 m Widening with 5 m Extended Berm

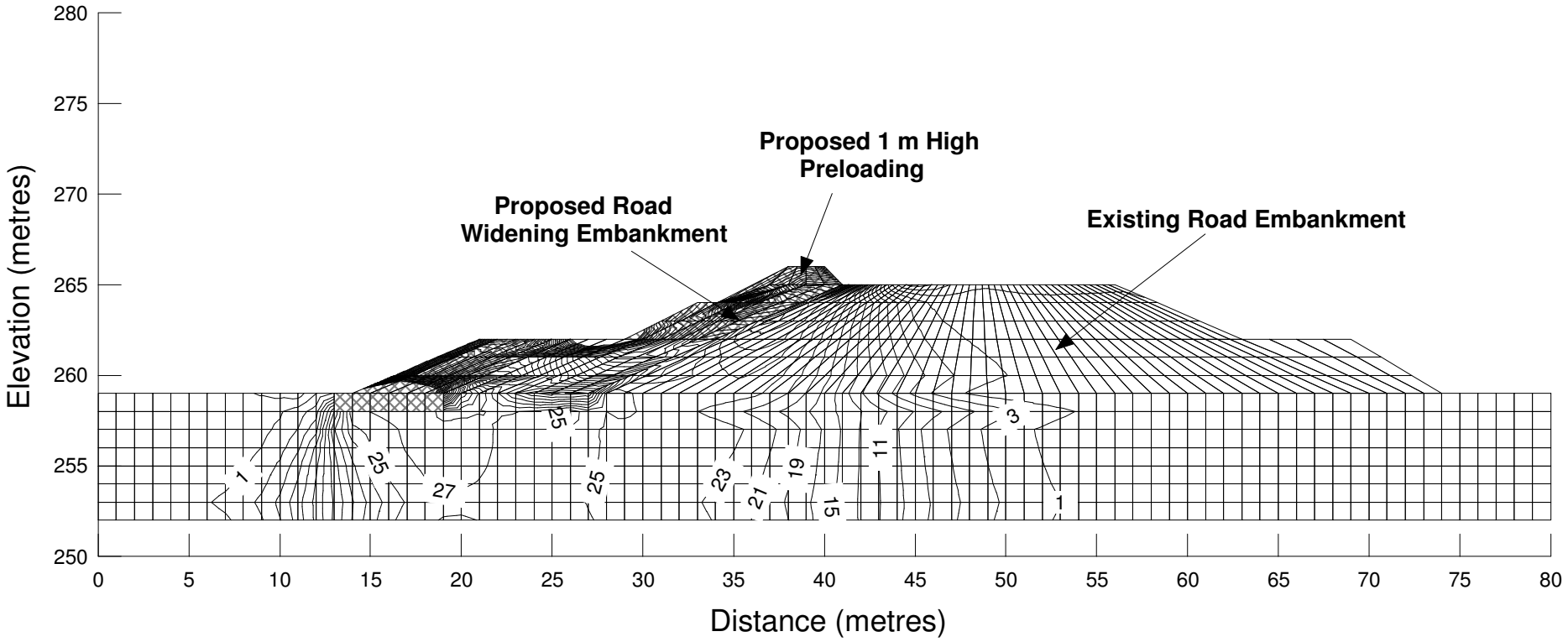


Figure F4.2 Vertical Stress Increment Contour (kPa) - Road Widening with 1 m Preloading at Station 18+875

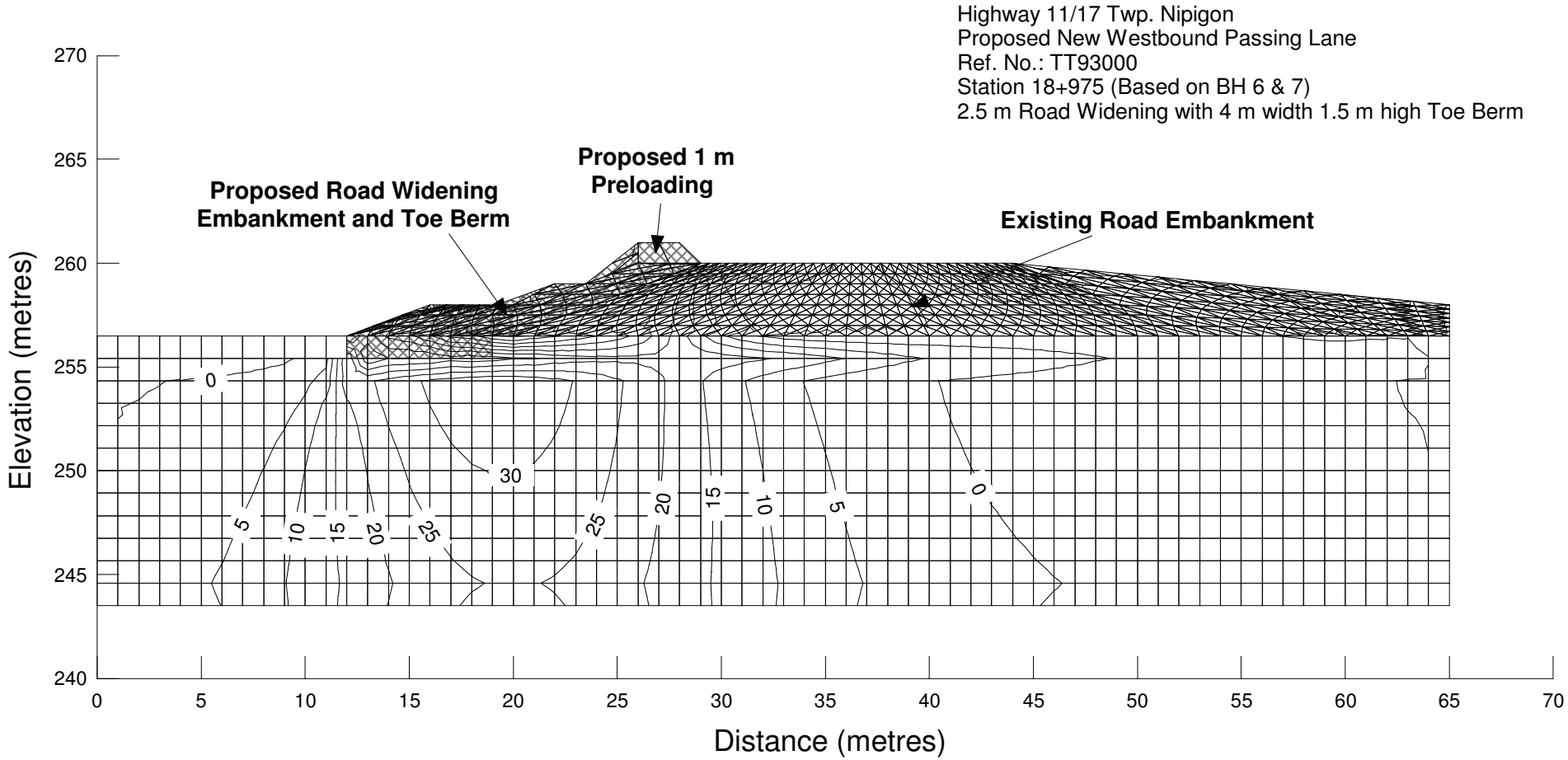


Figure F4.3 Vertical Stress Increment Contour (kPa) - Road Widening with 1 m Preloading at Station 18+975

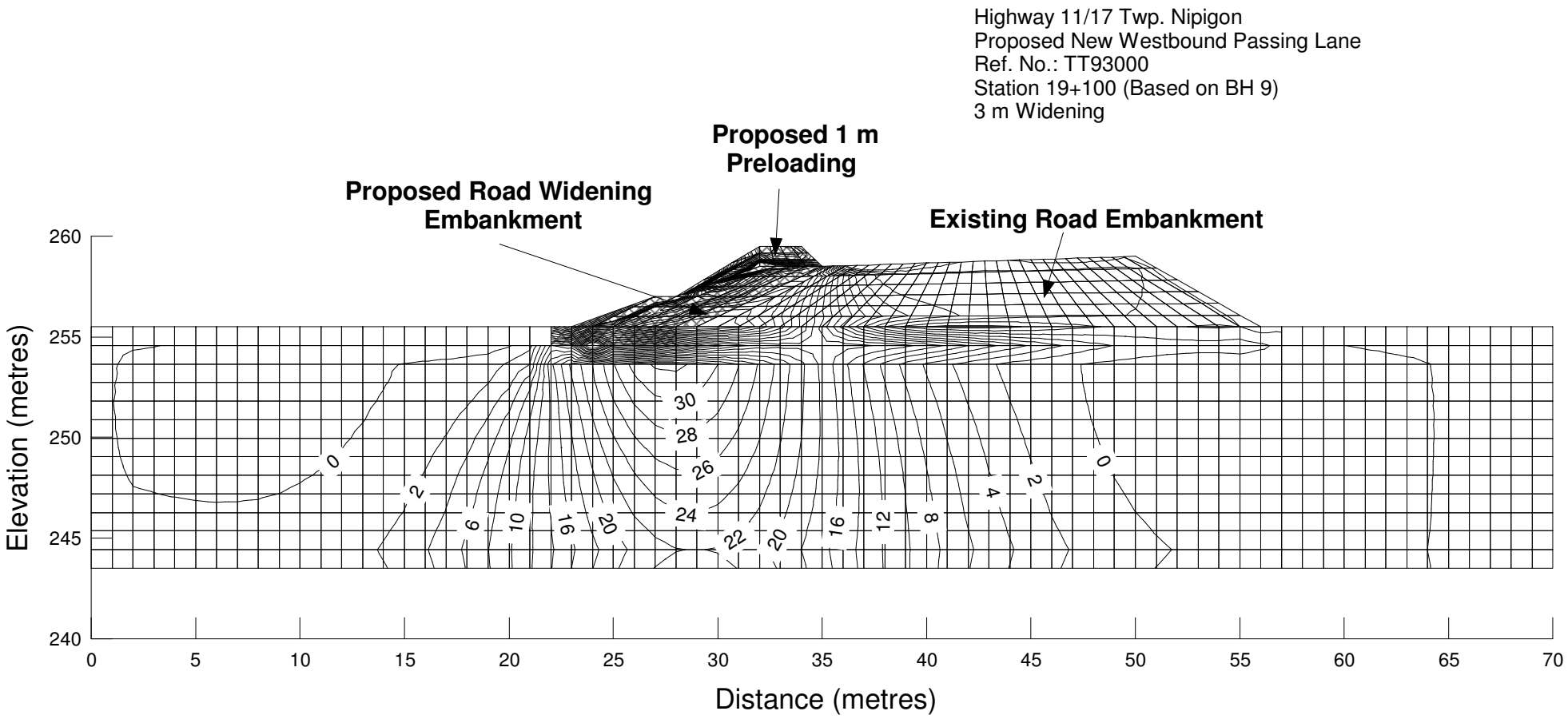
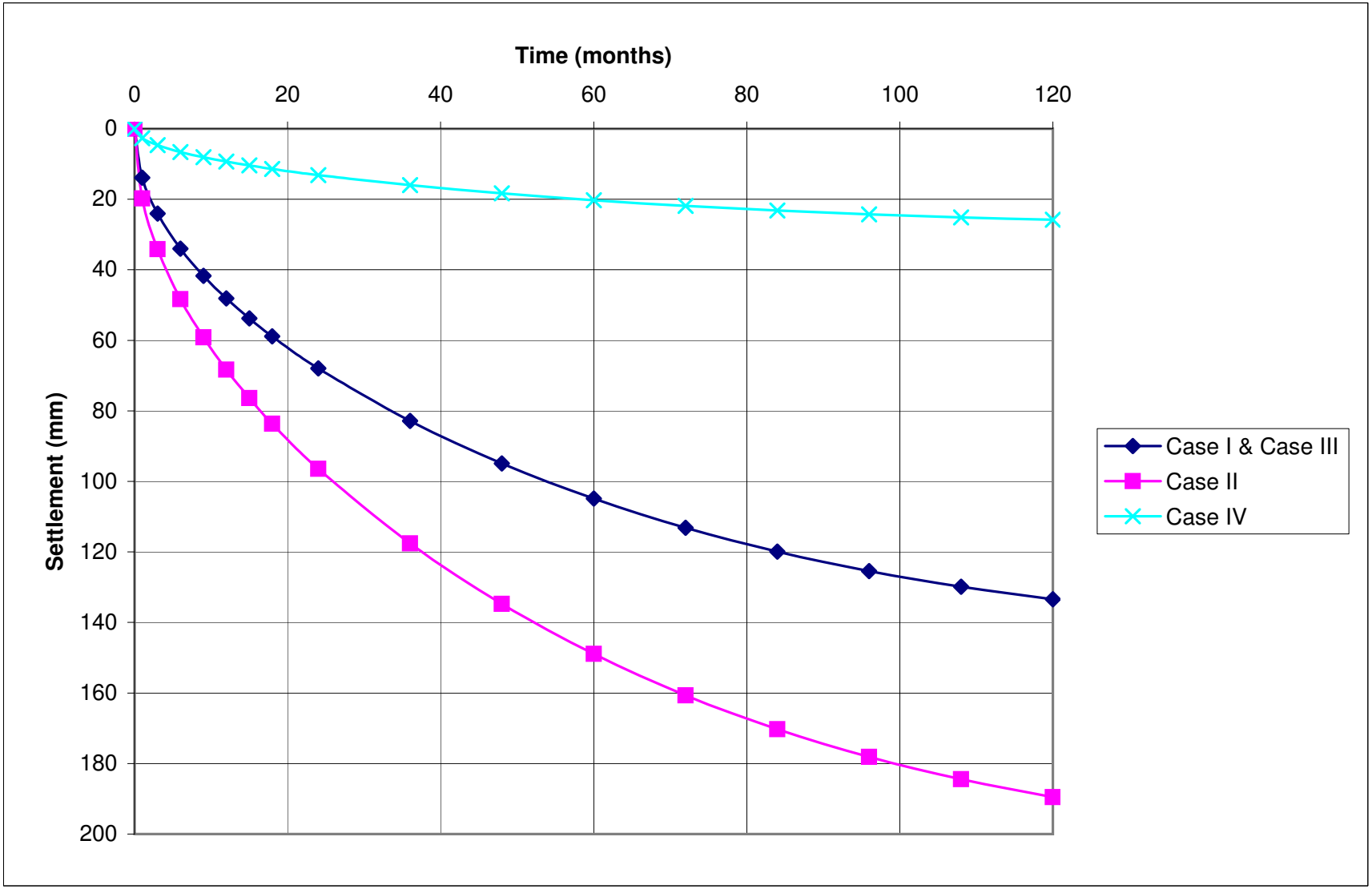
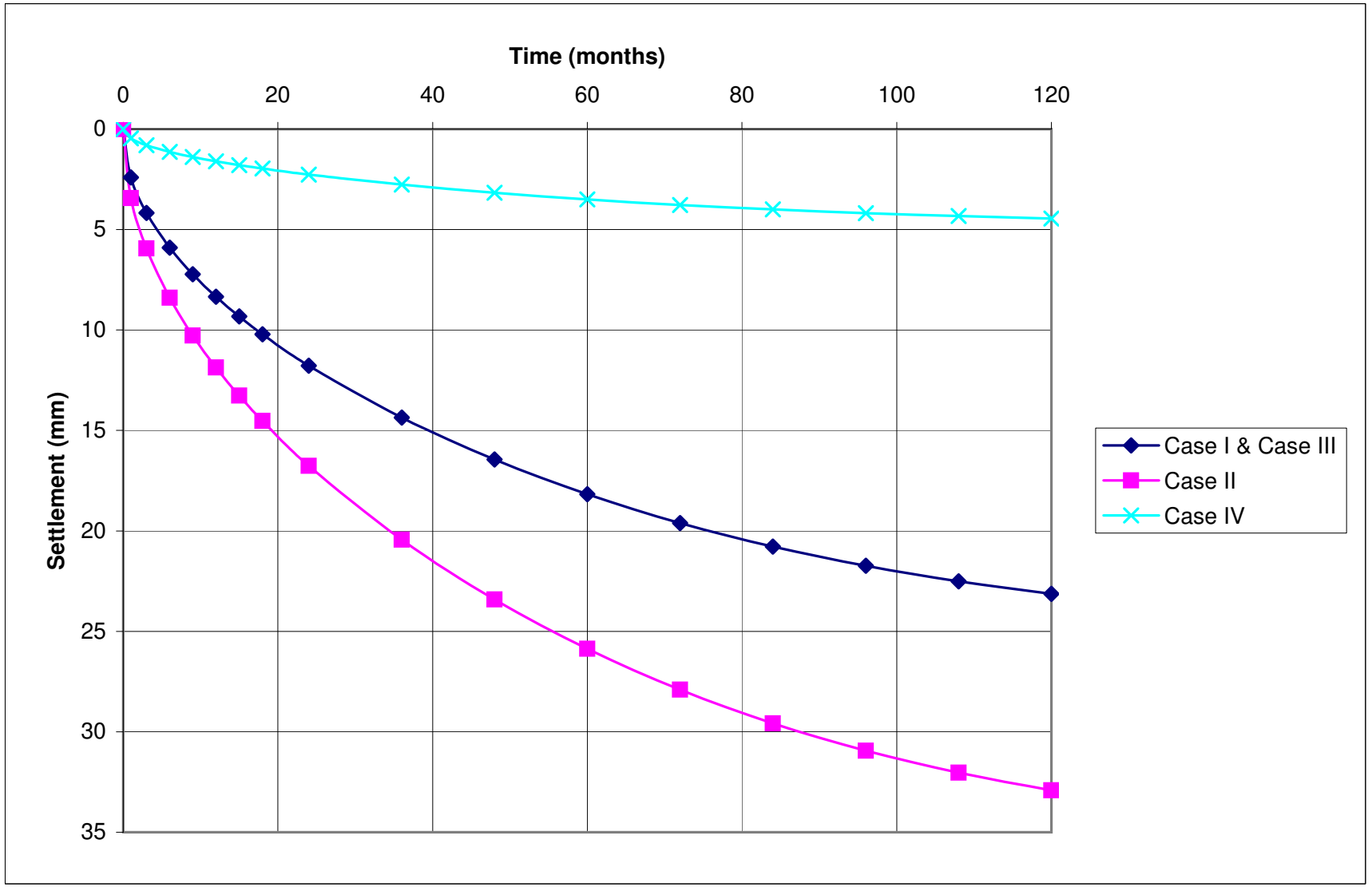


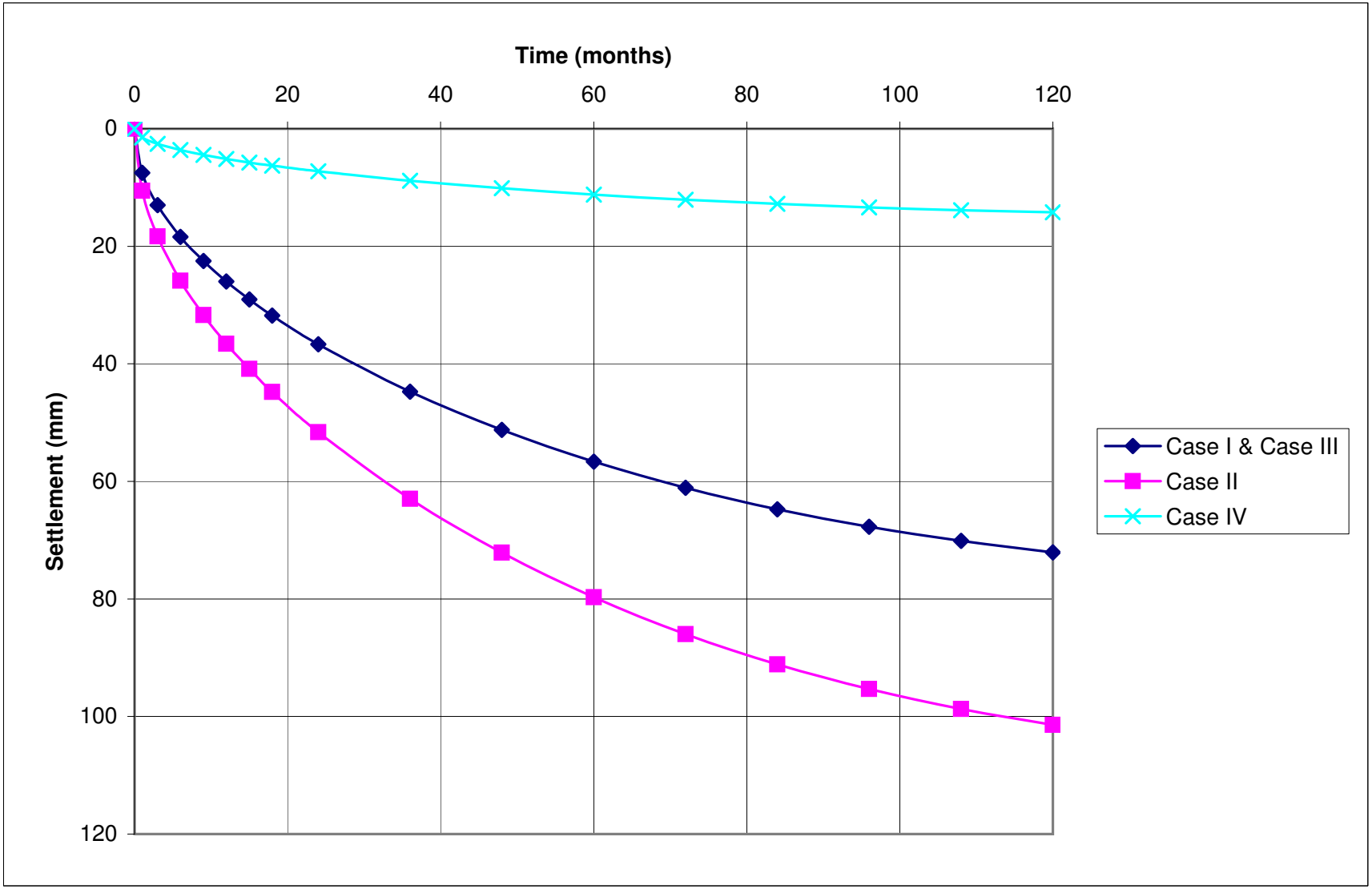
Figure F4.4 Vertical Stress Increment Contour (kPa) - Road Widening with 1 m Preloading at Station 19+100



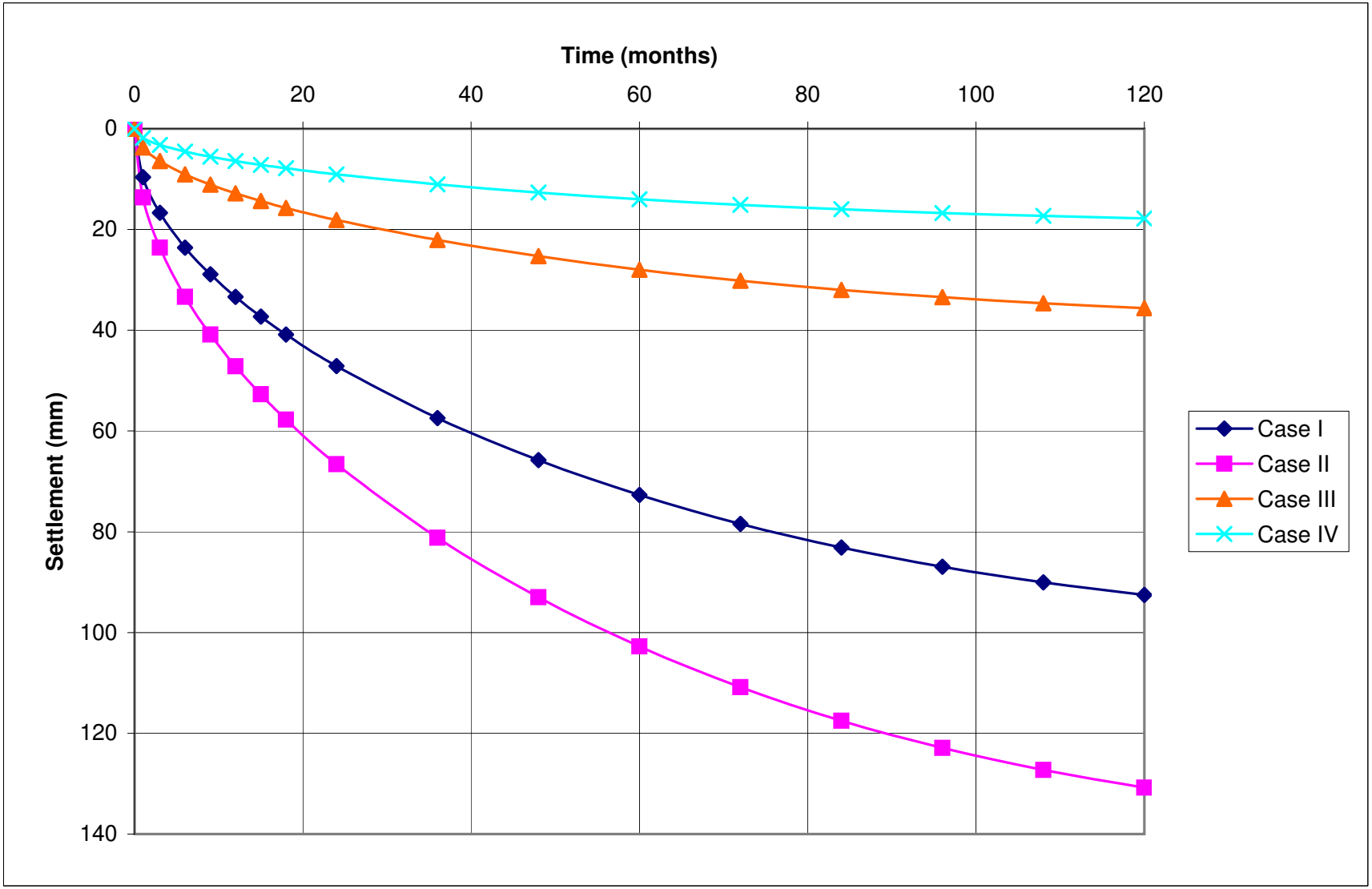
Ontario	**Settlement vs Time** **Station 18+730** **Crest of Widened Embankment**	FIG. NO. F5.1 CA # 6008-E-0027 Hwy. 11/ 17 Twp. Nipigon



Ontario	**Settlement vs Time** **Station 18+875** **Crest of Widened Embankment**	FIG. NO. F5.2 CA # 6008-E-0027 Hwy. 11/ 17 Twp. Nipigon



Ontario	**Settlement vs Time** **Station 18+975** **Crest of Widened Embankment**	FIG. NO. F5.3 CA # 6008-E-0027 Hwy. 11/ 17 Twp. Nipigon



Ontario	**Settlement vs Time** **Station 19+100** **Crest of Widened Embankment**	FIG. NO. F5.4 CA # 6008-E-0027 Hwy. 11/ 17 Twp. Nipigon

Table F1 - Primary Consolidation Calculations

Case I - Normally Consolidated, c_c = 0.18, as-planned load

Sample	C_c	C_s	e_o	e_f	c_v (m^2/day)	time (months)	C_α
TW 7 - BH 1	0.18	0.02	0.77	0.55	0.02	120	0.004

Station	Loc	H	p_0	Δp	p_f	Total Consol.	Settlement (S) 10 years after construction						Differential				
							Tv	%U	Consol.	Hnew	Secondary	Total	Transverse delta	Transverse grade change	Longitudinal delta	Longitudinal Distance	Longitudinal grade change
		m	kPa	kPa	kPa	mm			mm	m	mm	mm	mm	%	mm	m	%
18+730	A	9.5	148	14	162	38	0.809	89	34	9.5	10	43			36	145	0.02%
BH 1, 2 & 3	B	9.5	130	27	157	79	0.809	89	70	9.4	10	80	99	1.1%	61	145	0.04%
	C	9.5	112	48	160	150	0.809	89	133	9.4	10	143			112	145	0.08%
	D	9.5	65	54	119	254	0.809	89	226	9.3	10	235					
	E	9.5	35	34	69	285	0.809	89	253	9.2	10	263					
18+875	A	4.5	144	2	146	3	3.605	100	3	4.5	5	7			12	100	0.01%
BH 4	B	4.5	130	10	140	15	3.605	100	15	4.5	5	19	23	0.2%	30	100	0.03%
	C	4.5	115	16	131	26	3.605	100	26	4.5	5	30			49	100	0.05%
	D	4.5	100	20	120	36	3.605	100	36	4.5	5	41					
	E	4.5	85	22	107	46	3.605	100	46	4.5	5	50					
18+975	A	10	117	3	120	11	0.730	87	10	10.0	10	20			3	125	0.00%
BH 6 & 7	B	10	110	12	122	46	0.730	87	40	10.0	10	50	60	0.5%	12	125	0.01%
	C	10	90	18	108	81	0.730	87	70	9.9	10	80			20	125	0.02%
	D	10	47	23	70	176	0.730	87	152	9.8	10	162					
	E	10	42	26	68	213	0.730	87	184	9.8	10	194					
19+100	A	10	108	2	110	8	0.730	87	7	10.0	10	17					
BH 9	B	10	90	13	103	60	0.730	87	52	9.9	10	62	83	0.8%			
	C	10	68	18	86	104	0.730	87	90	9.9	10	100					
	D	10	48	21	69	160	0.730	87	139	9.9	10	149					
	E	10	38	15	53	147	0.730	87	127	9.9	10	137					

Settlement/Degree of Consolidation versus Time after construction

		3 months		6 months		9 months		12 months		24 months		60 months		120 months	
Station	Loc	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U
18+730	A	6	16	9	23	11	28	12	32	17	45	27	70	34	89
BH 1, 2 & 3	B	13	16	18	23	22	28	25	32	36	45	55	70	70	89
	C	24	16	34	23	42	28	48	32	68	45	105	70	133	89
	D	41	16	58	23	71	28	81	32	115	45	177	70	226	89
	E	46	16	65	23	79	28	91	32	129	45	199	70	253	89
18+875	A	1	34	1	48	2	58	2	66	2	86	3	98	3	100
BH 4	B	5	34	7	48	9	58	10	66	13	86	14	98	15	100
	C	9	34	12	48	15	58	17	66	22	86	25	98	26	100
	D	12	34	17	48	21	58	24	66	31	86	36	98	36	100
	E	15	34	22	48	27	58	30	66	40	86	45	98	46	100
18+975	A	2	15	2	22	3	26	3	30	5	43	7	67	10	87
BH 6 & 7	B	7	15	10	22	12	26	14	30	20	43	31	67	40	87
	C	12	15	17	22	21	26	25	30	35	43	54	67	70	87
	D	27	15	38	22	46	26	54	30	76	43	118	67	152	87
	E	32	15	46	22	56	26	65	30	92	43	142	67	184	87
19+100	A	1	15	2	22	2	26	2	30	3	43	5	67	7	87
BH 9	B	9	15	13	22	16	26	18	30	26	43	40	67	52	87
	C	16	15	22	22	27	26	32	30	45	43	69	67	90	87
	D	24	15	35	22	42	26	49	30	69	43	107	67	139	87
	E	22	15	32	22	39	26	45	30	63	43	98	67	127	87

Table F2 - Primary Consolidation Calculations

Case II - Normally Consolidated, c_c = 0.29, as-planned load

Sample	C_c	C_s	e_o	e_f	c_v (m^2/day)	time (months)	C_α
TW 5 - BH 6	0.29	0.04	1.01	0.76	0.02	120	0.011

							Settlement (S) 10 years after construction						Differential				
						Total							Transverse		Longitudinal		
Station	Loc	H	p_0	Δp	p_f	Consol.	Tv	%U	Consol.	Hnew	Secondary	Total	delta	grade change	delta	Distance	grade change
		m	kPa	kPa	kPa	mm			mm	m	mm	mm	mm	%	mm	m	%
18+730	A	9.5	148	14	162	54	0.809	89	48	9.5	10	57			49	145	0.03%
BH 1, 2 & 3	B	9.5	130	27	157	112	0.809	89	100	9.4	10	110	141	1.6%	84	145	0.06%
	C	9.5	112	48	160	213	0.809	89	189	9.3	10	199			157	145	0.11%
	D	9.5	65	54	119	360	0.809	89	321	9.2	10	330					
	E	9.5	35	34	69	404	0.809	89	360	9.1	10	369					
18+875	A	4.5	144	2	146	4	3.605	100	4	4.5	5	8			15	100	0.02%
BH 4	B	4.5	130	10	140	21	3.605	100	21	4.5	5	25	33	0.3%	41	100	0.04%
	C	4.5	115	16	131	37	3.605	100	37	4.5	5	41			68	100	0.07%
	D	4.5	100	20	120	51	3.605	100	51	4.4	5	56					
	E	4.5	85	22	107	65	3.605	100	65	4.4	5	69					
18+975	A	10	117	3	120	16	0.730	87	14	10.0	10	24			4	125	0.00%
BH 6 & 7	B	10	110	12	122	65	0.730	87	56	9.9	10	66	85	0.8%	17	125	0.01%
	C	10	90	18	108	114	0.730	87	99	9.9	10	109			29	125	0.02%
	D	10	47	23	70	250	0.730	87	217	9.8	10	227					
	E	10	42	26	68	302	0.730	87	262	9.7	10	272					
19+100	A	10	108	2	110	12	0.730	87	10	10.0	10	20					
BH 9	B	10	90	13	103	85	0.730	87	73	9.9	10	83	118	1.1%			
	C	10	68	18	86	147	0.730	87	128	9.9	10	138					
	D	10	48	21	69	228	0.730	87	197	9.8	10	207					
	E	10	38	15	53	209	0.730	87	181	9.8	10	191					

Settlement/Degree of Consolidation versus Time after construction

		3 months		6 months		9 months		12 months		24 months		60 months		120 months	
Station	Loc	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U
18+730	A	9	16	12	23	15	28	17	32	24	45	38	70	48	89
BH 1, 2 & 3	B	18	16	26	23	31	28	36	32	51	45	79	70	100	89
	C	34	16	48	23	59	28	68	32	96	45	149	70	189	89
	D	58	16	82	23	100	28	116	32	163	45	252	70	321	89
	E	65	16	92	23	112	28	130	32	183	45	283	70	360	89
18+875	A	1	34	2	48	2	58	3	66	3	86	4	98	4	100
BH 4	B	7	34	10	48	12	58	14	66	18	86	21	98	21	100
	C	12	34	18	48	21	58	24	66	32	86	36	98	37	100
	D	17	34	25	48	30	58	34	66	44	86	51	98	51	100
	E	22	34	31	48	38	58	43	66	56	86	64	98	65	100
18+975	A	2	15	3	22	4	26	5	30	7	43	11	67	14	87
BH 6 & 7	B	10	15	14	22	17	26	20	30	28	43	43	67	56	87
	C	17	15	25	22	30	26	35	30	49	43	76	67	99	87
	D	38	15	54	22	66	26	76	30	108	43	167	67	217	87
	E	46	15	65	22	80	26	92	30	130	43	202	67	262	87
19+100	A	2	15	2	22	3	26	4	30	5	43	8	67	10	87
BH 9	B	13	15	18	22	22	26	26	30	36	43	57	67	73	87
	C	22	15	32	22	39	26	45	30	63	43	98	67	128	87
	D	35	15	49	22	60	26	69	30	98	43	152	67	197	87
	E	32	15	45	22	55	26	64	30	90	43	139	67	181	87

Table F3 - Primary Consolidation Calculations

Case III - Over Consolidated, p_c = 80 kPa, as-planned load

Sample	C_c	C_s	e_o	e_f	p_c	c_v	time	c_α
					kpa	(m^2/day)	(months)	
TW 7 - BH 1	0.18	0.02	0.77	0.55	80	0.02	120	0.004

Station	Loc	H	p_0	Δp	p_f	Total Consol.	Settlement (S) 10 years after construction						Differential				
							Tv	%U	Consol.	Hnew	Secondary	Total	Transverse		Longitudinal		
													delta	grade change	delta	Distance	grade change
		m	kPa	kPa	kPa	mm			mm	m	mm	mm	mm	%	mm	m	%
18+730	A	9.5	148	14	162	38	0.809	89	34	9.5	10	43			36	145	0.02%
BH 1, 2 & 3	B	9.5	130	27	157	79	0.809	89	70	9.4	10	80	99	1.1%	61	145	0.04%
	C	9.5	112	48	160	150	0.809	89	133	9.4	10	143			112	145	0.08%
	D	9.5	65	54	119	176	0.809	89	157	9.3	10	166					
	E	9.5	35	34	69	32	0.809	89	28	9.5	10	38					
18+875	A	4.5	144	2	146	3	3.605	100	3	4.5	5	7			12	100	0.01%
BH 4	B	4.5	130	10	140	15	3.605	100	15	4.5	5	19	23	0.2%	30	100	0.03%
	C	4.5	115	16	131	26	3.605	100	26	4.5	5	30			49	100	0.05%
	D	4.5	100	20	120	36	3.605	100	36	4.5	5	41					
	E	4.5	85	22	107	46	3.605	100	46	4.5	5	50					
18+975	A	10	117	3	120	11	0.730	87	10	10.0	10	20			3	125	0.00%
BH 6 & 7	B	10	110	12	122	46	0.730	87	40	10.0	10	50	60	0.5%	12	125	0.01%
	C	10	90	18	108	81	0.730	87	70	9.9	10	80			35	125	0.03%
	D	10	47	23	70	20	0.730	87	17	10.0	10	27					
	E	10	42	26	68	24	0.730	87	20	10.0	10	30					
19+100	A	10	108	2	110	8	0.730	87	7	10.0	10	17					
BH 9	B	10	90	13	103	60	0.730	87	52	9.9	10	62	28	0.3%			
	C	10	68	18	86	40	0.730	87	35	10.0	10	45					
	D	10	48	21	69	18	0.730	87	15	10.0	10	25					
	E	10	38	15	53	16	0.730	87	14	10.0	10	24					

Settlement/Degree of Consolidation versus Time after construction

		3 months		6 months		9 months		12 months		24 months		60 months		120 months	
Station	Loc	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U
18+730	A	6	16	9	23	11	28	12	32	17	45	27	70	34	89
BH 1, 2 & 3	B	13	16	18	23	22	28	25	32	36	45	55	70	70	89
	C	24	16	34	23	42	28	48	32	68	45	105	70	133	89
	D	28	16	40	23	49	28	57	32	80	45	123	70	157	89
	E	5	16	7	23	9	28	10	32	14	45	22	70	28	89
18+875	A	1	34	1	48	2	58	2	66	2	86	3	98	3	100
BH 4	B	5	34	7	48	9	58	10	66	13	86	14	98	15	100
	C	9	34	12	48	15	58	17	66	22	86	25	98	26	100
	D	12	34	17	48	21	58	24	66	31	86	36	98	36	100
	E	15	34	22	48	27	58	30	66	40	86	45	98	46	100
18+975	A	2	15	2	22	3	26	3	30	5	43	7	67	10	87
BH 6 & 7	B	7	15	10	22	12	26	14	30	20	43	31	67	40	87
	C	12	15	17	22	21	26	25	30	35	43	54	67	70	87
	D	3	15	4	22	5	26	6	30	8	43	13	67	17	87
	E	4	15	5	22	6	26	7	30	10	43	16	67	20	87
19+100	A	1	15	2	22	2	26	2	30	3	43	5	67	7	87
BH 9	B	9	15	13	22	16	26	18	30	26	43	40	67	52	87
	C	6	15	9	22	11	26	12	30	17	43	27	67	35	87
	D	3	15	4	22	5	26	5	30	8	43	12	67	15	87
	E	2	15	4	22	4	26	5	30	7	43	11	67	14	87

Table F4 - Primary Consolidation Calculations

Case IV - Over Consolidated, p_c = 160 kPa, as-planned load

Sample	C_c	C_s	e_o	e_f	p_c	c_v	time	c_α
					kpa	(m^2/day)	(months)	
TW 5 - BH 6	0.29	0.04	1.01	0.76	160	0.02	120	0.011

Station	Loc	H	p_0	Δp	p_f	Total Consol.	Settlement (S) 10 years after construction						Differential				
							Tv	%U	Consol.	Hnew	Secondary	Total	Transverse		Longitudinal		
													delta	grade change	delta	Distance	grade change
		m	kPa	kPa	kPa	mm			mm	m	mm	mm	mm	%	mm	m	%
18+730	A	9.5	148	14	162	14	0.809	89	12	9.5	10	22			17	145	0.01%
BH 1, 2 & 3	B	9.5	130	27	157	15	0.809	89	14	9.5	10	23	14	0.2%	16	145	0.01%
	C	9.5	112	48	160	29	0.809	89	26	9.5	10	36			26	145	0.02%
	D	9.5	65	54	119	50	0.809	89	44	9.5	10	54					
	E	9.5	35	34	69	56	0.809	89	50	9.5	10	59					
18+875	A	4.5	144	2	146	1	3.605	100	1	4.5	5	5			7	100	0.01%
BH 4	B	4.5	130	10	140	3	3.605	100	3	4.5	5	7	5	0.0%	10	100	0.01%
	C	4.5	115	16	131	5	3.605	100	5	4.5	5	10			14	100	0.01%
	D	4.5	100	20	120	7	3.605	100	7	4.5	5	12					
	E	4.5	85	22	107	9	3.605	100	9	4.5	5	13					
18+975	A	10	117	3	120	2	0.730	87	2	10.0	10	12			1	125	0.00%
BH 6 & 7	B	10	110	12	122	9	0.730	87	8	10.0	10	18	12	0.1%	2	125	0.00%
	C	10	90	18	108	16	0.730	87	14	10.0	10	24			4	125	0.00%
	D	10	47	23	70	34	0.730	87	30	10.0	10	40					
	E	10	42	26	68	42	0.730	87	36	10.0	10	46					
19+100	A	10	108	2	110	2	0.730	87	1	10.0	10	11					
BH 9	B	10	90	13	103	12	0.730	87	10	10.0	10	20	16	0.1%			
	C	10	68	18	86	20	0.730	87	18	10.0	10	28					
	D	10	48	21	69	31	0.730	87	27	10.0	10	37					
	E	10	38	15	53	29	0.730	87	25	10.0	10	35					

Settlement/Degree of Consolidation versus Time after construction

		3 months		6 months		9 months		12 months		24 months		60 months		120 months	
Station	Loc	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U
18+730	A	2	16	3	23	4	28	4	32	6	45	10	70	12	89
BH 1, 2 & 3	B	2	16	3	23	4	28	5	32	7	45	11	70	14	89
	C	5	16	7	23	8	28	9	32	13	45	20	70	26	89
	D	8	16	11	23	14	28	16	32	22	45	35	70	44	89
	E	9	16	13	23	15	28	18	32	25	45	39	70	49	89
18+875	A	0	17	0	24	0	30	0	34	0	49	0	74	0	92
BH 4	B	0	17	1	24	1	30	1	34	1	49	2	74	3	92
	C	1	17	1	24	2	30	2	34	2	49	4	74	5	92
	D	1	17	2	24	2	30	2	34	3	49	5	74	7	92
	E	2	17	2	24	3	30	3	34	4	49	7	74	8	92
18+975	A	0	13	0	19	0	23	1	26	1	37	1	58	2	79
BH 6 & 7	B	1	13	2	19	2	23	2	26	3	37	5	58	7	79
	C	2	13	3	19	4	23	4	26	6	37	9	58	12	79
	D	5	13	6	19	8	23	9	26	13	37	20	58	27	79
	E	5	13	8	19	9	23	11	26	15	37	24	58	33	79
19+100	A	0	12	0	17	0	21	0	24	1	34	1	54	1	74
BH 9	B	1	12	2	17	2	21	3	24	4	34	6	54	9	74
	C	2	12	3	17	4	21	5	24	7	34	11	54	15	74
	D	4	12	5	17	7	21	8	24	11	34	17	54	23	74
	E	3	12	5	17	6	21	7	24	10	34	15	54	21	74

Table F5 - Primary Consolidation Calculations

Case I - Normally Consolidated, c_c = 0.18, additional surcharge load

Sample	C_c	C_s	e_o	e_f	c_v (m^2/day)	time (months)	C_α
TW 7 - BH 1	0.18	0.02	0.77	0.55	0.02	120	0.004

Station	Loc	H	p_0	Δp	p_f	Total Consol.	Settlement (S) 10 years after construction						Differential				
							Tv	%U	Consol.	Hnew	Secondary	Total	Transverse delta	Transverse grade change	Longitudinal delta	Longitudinal Distance	Longitudinal grade change
		m	kPa	kPa	kPa	mm			mm	m	mm	mm	mm	%	mm	m	%
18+730	A	9.5	148	15	163	41	0.809	89	36	9.5	10	46			37	145	0.03%
BH 1, 2 & 3	B	9.5	130	33	163	95	0.809	89	84	9.4	10	94	83	0.9%	68	145	0.05%
	C	9.5	112	42	154	134	0.809	89	119	9.4	10	128			92	145	0.06%
	D	9.5	65	57	122	264	0.809	89	235	9.3	10	245					
	E	9.5	35	35	70	291	0.809	89	259	9.2	10	268					
18+875	A	4.5	144	3	147	4	3.605	100	4	4.5	5	9			14	100	0.01%
BH 4	B	4.5	130	15	145	22	3.605	100	22	4.5	5	26	28	0.2%	36	100	0.04%
	C	4.5	115	20	135	32	3.605	100	32	4.5	5	36			57	100	0.06%
	D	4.5	100	23	123	41	3.605	100	41	4.5	5	45					
	E	4.5	85	25	110	51	3.605	100	51	4.4	5	56					
18+975	A	10	117	4	121	15	0.730	87	13	10.0	10	23			1	125	0.00%
BH 6 & 7	B	10	110	16	126	60	0.730	87	52	9.9	10	62	71	0.6%	18	125	0.01%
	C	10	90	22	112	97	0.730	87	84	9.9	10	94			28	125	0.02%
	D	10	47	27	74	200	0.730	87	174	9.8	10	184					
	E	10	42	29	71	232	0.730	87	201	9.8	10	211					
19+100	A	10	108	4	112	16	0.730	87	14	10.0	10	24					
BH 9	B	10	90	18	108	81	0.730	87	70	9.9	10	80	98	0.9%			
	C	10	68	23	91	129	0.730	87	112	9.9	10	122					
	D	10	48	26	74	191	0.730	87	166	9.8	10	176					
	E	10	38	18	56	171	0.730	87	148	9.9	10	158					

Settlement/Degree of Consolidation versus Time after construction

		3 months		6 months		9 months		12 months		24 months		60 months		120 months	
Station	Loc	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U
18+730	A	6	16	9	23	11	28	13	32	18	45	28	70	36	89
BH 1, 2 & 3	B	15	16	22	23	26	28	30	32	43	45	66	70	84	89
	C	21	16	30	23	37	28	43	32	61	45	93	70	119	89
	D	42	16	60	23	73	28	85	32	120	45	185	70	235	89
	E	47	16	66	23	81	28	93	32	132	45	203	70	259	89
18+875	A	1	34	2	48	2	58	3	66	4	86	4	98	4	100
BH 4	B	7	34	10	48	13	58	14	66	19	86	21	98	22	100
	C	11	34	15	48	19	58	21	66	28	86	31	98	32	100
	D	14	34	20	48	24	58	27	66	36	86	40	98	41	100
	E	17	34	24	48	30	58	34	66	44	86	50	98	51	100
18+975	A	2	15	3	22	4	26	5	30	6	43	10	67	13	87
BH 6 & 7	B	9	15	13	22	16	26	18	30	26	43	40	67	52	87
	C	15	15	21	22	25	26	29	30	42	43	65	67	84	87
	D	31	15	43	22	53	26	61	30	86	43	134	67	174	87
	E	35	15	50	22	61	26	71	30	100	43	155	67	201	87
19+100	A	2	15	3	22	4	26	5	30	7	43	11	67	14	87
BH 9	B	12	15	17	22	21	26	25	30	35	43	54	67	70	87
	C	20	15	28	22	34	26	39	30	55	43	86	67	112	87
	D	29	15	41	22	50	26	58	30	82	43	128	67	166	87
	E	26	15	37	22	45	26	52	30	74	43	114	67	148	87

Table F6 - Primary Consolidation Calculations

Case II - Normally Consolidated, c_c = 0.29, additional surcharge load

Sample	C_c	C_s	e_o	e_f	c_v (m^2/day)	time (months)	C_α
TW 5 - BH 6	0.29	0.04	1.01	0.76	0.02	120	0.011

							Settlement (S) 10 years after construction						Differential				
						Total							Transverse		Longitudinal		
Station	Loc	H	p_0	Δp	p_f	Consol.	Tv	%U	Consol.	Hnew	Secondary	Total	delta	grade change	delta	Distance	grade change
		m	kPa	kPa	kPa	mm			mm	m	mm	mm	mm	%	mm	m	%
18+730	A	9.5	148	15	163	58	0.809	89	51	9.4	10	61			50	145	0.03%
BH 1, 2 & 3	B	9.5	130	33	163	135	0.809	89	120	9.4	10	129	118	1.3%	94	145	0.06%
	C	9.5	112	42	154	190	0.809	89	169	9.3	10	178			129	145	0.09%
	D	9.5	65	57	122	375	0.809	89	334	9.2	10	343					
	E	9.5	35	35	70	413	0.809	89	367	9.1	10	377					
18+875	A	4.5	144	3	147	6	3.605	100	6	4.5	5	10			18	100	0.02%
BH 4	B	4.5	130	15	145	31	3.605	100	31	4.5	5	35	39	0.3%	49	100	0.05%
	C	4.5	115	20	135	45	3.605	100	45	4.5	5	50			79	100	0.08%
	D	4.5	100	23	123	58	3.605	100	58	4.4	5	63					
	E	4.5	85	25	110	73	3.605	100	72	4.4	5	77					
18+975	A	10	117	4	121	21	0.730	87	18	10.0	10	28			1	125	0.00%
BH 6 & 7	B	10	110	16	126	85	0.730	87	74	9.9	10	84	101	0.9%	25	125	0.02%
	C	10	90	22	112	137	0.730	87	119	9.9	10	129			39	125	0.03%
	D	10	47	27	74	285	0.730	87	247	9.8	10	257					
	E	10	42	29	71	329	0.730	87	285	9.7	10	295					
19+100	A	10	108	4	112	23	0.730	87	20	10.0	10	30					
BH 9	B	10	90	18	108	114	0.730	87	99	9.9	10	109	139	1.3%			
	C	10	68	23	91	183	0.730	87	158	9.8	10	168					
	D	10	48	26	74	272	0.730	87	235	9.8	10	245					
	E	10	38	18	56	243	0.730	87	211	9.8	10	221					

Settlement/Degree of Consolidation versus Time after construction

		3 months		6 months		9 months		12 months		24 months		60 months		120 months	
Station	Loc	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U
18+730	A	9	16	13	23	16	28	18	32	26	45	40	70	51	89
BH 1, 2 & 3	B	22	16	31	23	37	28	43	32	61	45	94	70	120	89
	C	30	16	43	23	53	28	61	32	86	45	133	70	169	89
	D	60	16	85	23	104	28	120	32	170	45	262	70	334	89
	E	66	16	94	23	115	28	133	32	187	45	289	70	367	89
18+875	A	2	34	3	48	3	58	4	66	5	86	6	98	6	100
BH 4	B	10	34	15	48	18	58	20	66	27	86	30	98	31	100
	C	15	34	22	48	26	58	30	66	39	86	44	98	45	100
	D	20	34	28	48	34	58	39	66	50	86	57	98	58	100
	E	25	34	35	48	42	58	48	66	63	86	71	98	72	100
18+975	A	3	15	5	22	6	26	6	30	9	43	14	67	18	87
BH 6 & 7	B	13	15	18	22	22	26	26	30	37	43	57	67	74	87
	C	21	15	30	22	36	26	42	30	59	43	92	67	119	87
	D	43	15	61	22	75	26	87	30	123	43	190	67	247	87
	E	50	15	71	22	87	26	100	30	142	43	220	67	285	87
19+100	A	3	15	5	22	6	26	7	30	10	43	15	67	20	87
BH 9	B	17	15	25	22	30	26	35	30	49	43	76	67	99	87
	C	28	15	39	22	48	26	56	30	79	43	122	67	158	87
	D	41	15	59	22	72	26	83	30	117	43	181	67	235	87
	E	37	15	52	22	64	26	74	30	105	43	163	67	211	87

Table F7 - Primary Consolidation Calculations

Case II - Over Consolidated, p_c = 80 kPa, additional surcharge load

Sample	C_c	C_s	e_o	e_f	p_c	c_v	time	c_α
					kpa	(m^2/day)	(months)	
TW 7 - BH 1	0.18	0.02	0.77	0.55	80	0.02	120	0.004

							Settlement (S) 10 years after construction						Differential				
						Total							Transverse		Longitudinal		
Station	Loc	H	p_0	Δp	p_f	Consol.	Tv	%U	Consol.	Hnew	Secondary	Total	delta	grade change	delta	Distance	grade change
		m	kPa	kPa	kPa	mm			mm	m	mm	mm	mm	%	mm	m	%
18+730	A	9.5	148	15	163	41	0.809	89	36	9.5	10	46			37	145	0.03%
BH 1, 2 & 3	B	9.5	130	33	163	95	0.809	89	84	9.4	10	94	83	0.9%	68	145	0.05%
	C	9.5	112	42	154	134	0.809	89	119	9.4	10	128			92	145	0.06%
	D	9.5	65	57	122	187	0.809	89	166	9.3	10	176					
	E	9.5	35	35	70	32	0.809	89	29	9.5	10	38					
18+875	A	4.5	144	3	147	4	3.605	100	4	4.5	5	9			14	100	0.01%
BH 4	B	4.5	130	15	145	22	3.605	100	22	4.5	5	26	28	0.2%	36	100	0.04%
	C	4.5	115	20	135	32	3.605	100	32	4.5	5	36			57	100	0.06%
	D	4.5	100	23	123	41	3.605	100	41	4.5	5	45					
	E	4.5	85	25	110	51	3.605	100	51	4.4	5	56					
18+975	A	10	117	4	121	15	0.730	87	13	10.0	10	23			1	125	0.00%
BH 6 & 7	B	10	110	16	126	60	0.730	87	52	9.9	10	62	71	0.6%	18	125	0.01%
	C	10	90	22	112	97	0.730	87	84	9.9	10	94			27	125	0.02%
	D	10	47	27	74	22	0.730	87	19	10.0	10	29					
	E	10	42	29	71	26	0.730	87	22	10.0	10	32					
19+100	A	10	108	4	112	16	0.730	87	14	10.0	10	24					
BH 9	B	10	90	18	108	81	0.730	87	70	9.9	10	80	42	0.4%			
	C	10	68	23	91	65	0.730	87	56	9.9	10	66					
	D	10	48	26	74	21	0.730	87	18	10.0	10	28					
	E	10	38	18	56	19	0.730	87	16	10.0	10	26					

Settlement/Degree of Consolidation versus Time after construction

		3 months		6 months		9 months		12 months		24 months		60 months		120 months	
Station	Loc	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U
18+730	A	6	16	9	23	11	28	13	32	18	45	28	70	36	89
BH 1, 2 & 3	B	15	16	22	23	26	28	30	32	43	45	66	70	84	89
	C	21	16	30	23	37	28	43	32	61	45	93	70	119	89
	D	30	16	42	23	52	28	60	32	85	45	131	70	166	89
	E	5	16	7	23	9	28	10	32	15	45	23	70	29	89
18+875	A	1	34	2	48	2	58	3	66	4	86	4	98	4	100
BH 4	B	7	34	10	48	13	58	14	66	19	86	21	98	22	100
	C	11	34	15	48	19	58	21	66	28	86	31	98	32	100
	D	14	34	20	48	24	58	27	66	36	86	40	98	41	100
	E	17	34	24	48	30	58	34	66	44	86	50	98	51	100
18+975	A	2	15	3	22	4	26	5	30	6	43	10	67	13	87
BH 6 & 7	B	9	15	13	22	16	26	18	30	26	43	40	67	52	87
	C	15	15	21	22	25	26	29	30	42	43	65	67	84	87
	D	3	15	5	22	6	26	7	30	10	43	15	67	19	87
	E	4	15	6	22	7	26	8	30	11	43	17	67	22	87
19+100	A	2	15	3	22	4	26	5	30	7	43	11	67	14	87
BH 9	B	12	15	17	22	21	26	25	30	35	43	54	67	70	87
	C	10	15	14	22	17	26	20	30	28	43	43	67	56	87
	D	3	15	5	22	6	26	6	30	9	43	14	67	18	87
	E	3	15	4	22	5	26	6	30	8	43	13	67	16	87

Table F8 - Primary Consolidation Calculations

Case III - Over Consolidated, p_c = 160 kPa, additional surcharge load

Sample	C_c	C_s	e_o	e_f	p_c	c_v	time	c_α
					kpa	(m^2/day)	(months)	
TW 5 - BH 6	0.29	0.04	1.01	0.76	160	0.02	120	0.011

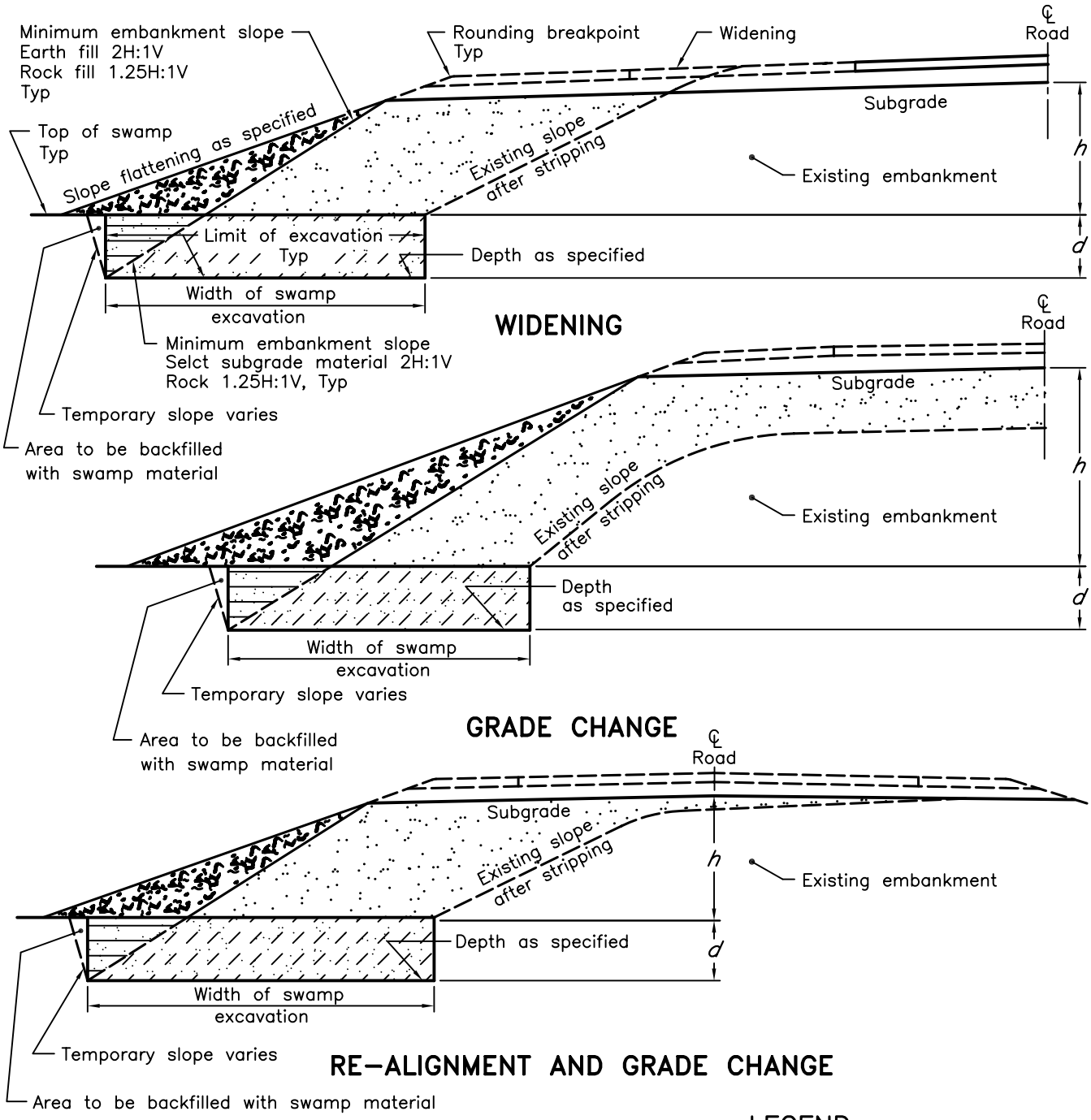
Station	Loc	H	p_0	Δp	p_f	Total Consol.	Settlement (S) 10 years after construction						Differential				
							Tv	%U	Consol.	Hnew	Secondary	Total	Transverse		Longitudinal		
													delta	grade change	delta	Distance	grade change
		m	kPa	kPa	kPa	mm			mm	m	mm	mm	mm	%	mm	m	%
18+730	A	9.5	148	15	163	17	0.809	89	16	9.5	10	25			20	145	0.01%
BH 1, 2 & 3	B	9.5	130	33	163	28	0.809	89	25	9.5	10	35	8	0.1%	26	145	0.02%
	C	9.5	112	42	154	26	0.809	89	23	9.5	10	33			22	145	0.02%
	D	9.5	65	57	122	52	0.809	89	46	9.5	10	55					
	E	9.5	35	35	70	57	0.809	89	51	9.4	10	60					
18+875	A	4.5	144	3	147	1	3.605	100	1	4.5	5	5			7	100	0.01%
BH 4	B	4.5	130	15	145	4	3.605	100	4	4.5	5	9	5	0.0%	11	100	0.01%
	C	4.5	115	20	135	6	3.605	100	6	4.5	5	11			16	100	0.02%
	D	4.5	100	23	123	8	3.605	100	8	4.5	5	13					
	E	4.5	85	25	110	10	3.605	100	10	4.5	5	14					
18+975	A	10	117	4	121	3	0.730	87	3	10.0	10	13			0	125	0.00%
BH 6 & 7	B	10	110	16	126	12	0.730	87	10	10.0	10	20	14	0.1%	3	125	0.00%
	C	10	90	22	112	19	0.730	87	16	10.0	10	26			5	125	0.00%
	D	10	47	27	74	39	0.730	87	34	10.0	10	44					
	E	10	42	29	71	45	0.730	87	39	10.0	10	49					
19+100	A	10	108	4	112	3	0.730	87	3	10.0	10	13					
BH 9	B	10	90	18	108	16	0.730	87	14	10.0	10	24	19	0.2%			
	C	10	68	23	91	25	0.730	87	22	10.0	10	32					
	D	10	48	26	74	37	0.730	87	32	10.0	10	42					
	E	10	38	18	56	34	0.730	87	29	10.0	10	39					

Settlement/Degree of Consolidation versus Time after construction

		3 months		6 months		9 months		12 months		24 months		60 months		120 months	
Station	Loc	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U	S (mm)	%U
18+730	A	3	16	4	23	5	28	6	32	8	45	12	70	16	89
BH 1, 2 & 3	B	5	16	6	23	8	28	9	32	13	45	20	70	25	89
	C	4	16	6	23	7	28	8	32	12	45	18	70	23	89
	D	8	16	12	23	14	28	17	32	23	45	36	70	46	89
	E	9	16	13	23	16	28	18	32	26	45	40	70	51	89
18+875	A	0	34	0	48	0	58	1	66	1	86	1	98	1	100
BH 4	B	1	34	2	48	2	58	3	66	4	86	4	98	4	100
	C	2	34	3	48	4	58	4	66	5	86	6	98	6	100
	D	3	34	4	48	5	58	5	66	7	86	8	98	8	100
	E	3	34	5	48	6	58	7	66	9	86	10	98	10	100
18+975	A	0	15	1	22	1	26	1	30	1	43	2	67	3	87
BH 6 & 7	B	2	15	3	22	3	26	4	30	5	43	8	67	10	87
	C	3	15	4	22	5	26	6	30	8	43	13	67	16	87
	D	6	15	8	22	10	26	12	30	17	43	26	67	34	87
	E	7	15	10	22	12	26	14	30	20	43	30	67	39	87
19+100	A	0	15	1	22	1	26	1	30	1	43	2	67	3	87
BH 9	B	2	15	3	22	4	26	5	30	7	43	11	67	14	87
	C	4	15	5	22	7	26	8	30	11	43	17	67	22	87
	D	6	15	8	22	10	26	11	30	16	43	25	67	32	87
	E	5	15	7	22	9	26	10	30	14	43	22	67	29	87

APPENDIX G

OPSD – 203.030



NOTES:

A For this OPSD, h must be $\leq$4.5m and d must be $\leq$6.0m.

B Topsoil shall be stripped from existing slopes.

C Height of fill is the vertical difference between top of subgrade and top of swamp elevation measured at new road centreline.

D Widening of existing earth embankments shall be benched according to OPSD–208.010.

E All dimensions are in millimetres unless otherwise shown.

LEGEND:

h – Height of fill

d – Depth of sub–excavation

- Embankment materials as specified
- Excavated swamp material
- Excavate and backfill as specified
- Excavate and backfill with swamp material

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EMBANKMENTS OVER SWAMP

EXISTING SLOPES MAINTAINED

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OPSD – 203.030

ADDENDUM

SUMMARIZED RECOMMENDATIONS FOR THE WIDENED ROAD CONFIGURATIONS FOR DESIGN

ADDENDUM

This Addendum provides the summarized recommendations for the widened road configurations for design. According to MTO's requirements, the widened road embankment will generally consist of approximately 1 m thick road pavement structure with 3H:1V side slope underlain by rock fill. The results of slope stability analyses and the settlement calculations supporting the recommendations are compiled in Appendix-X of this Addendum. All other relevant recommendations and supporting analyses are provided in the main report.

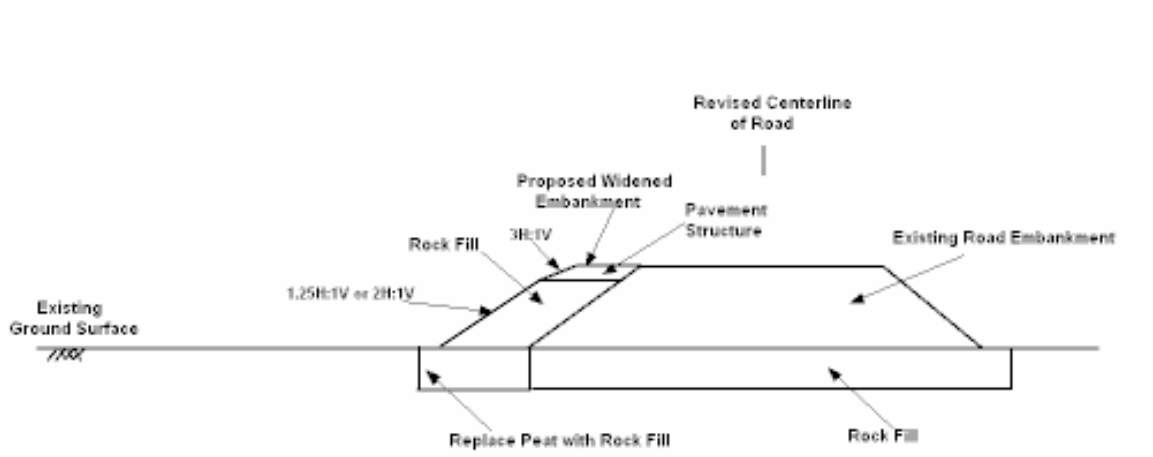
This Addendum consists of the following Table of Contents:

Table A Recommended Widened Embankment Configuration
(see Figures at the end of Table)

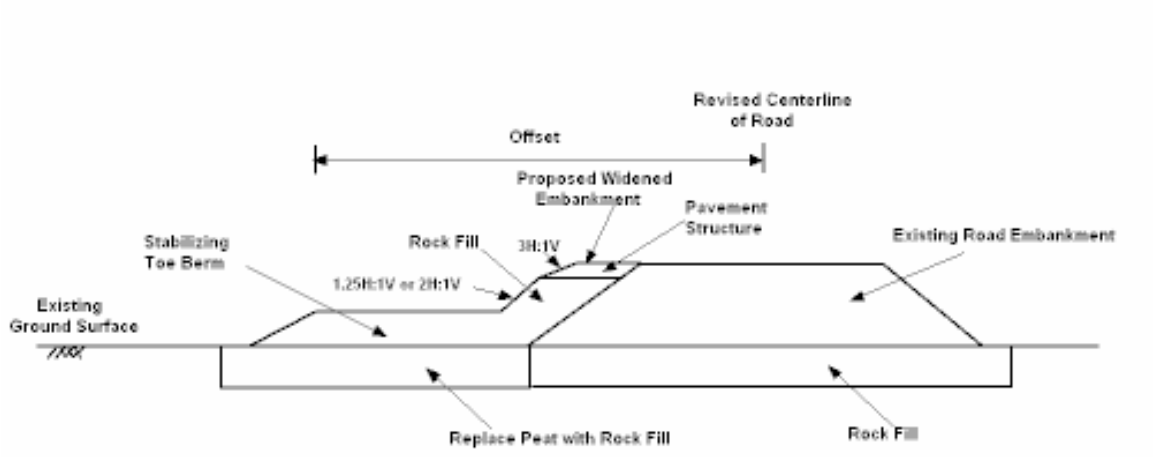
Station	Proposed Road Widening[1]		Stabilizing Toe Berm		Proposed Widened Rock Fill Slope	Remarks
	Top Width	Widened Embankment Height	Top Elevation	Offset to Edge of Toe Berm from Centerline of Existing Road		
(m)	(m)	(m)	(m)	(m)	(H:V)	
18+700	4.0	2.0	-	-	2:1	The peat layer (about 2.1 m to 3.7 m thick) underneath the widened embankment footprint shall be replaced with rock fill as per OPSD 203.030.
18+725	4.0	4.8	-	-	2:1	
18+729 (culvert location)	3.0	4.8	-	-	2:1	
18+750	2.5	3.5	-	-	2:1	
18+775	3.5	2.0	-	-		Cut section in bed rock.
18+800	4.0	1.3	-	-		
18+825	3.0	2.2	265.5*	20*	1.25:1	*Existing rock fill stabilizing berm section. ** Extending the existing rock fill berm should be considered.
18+850	2.5	3.0	263.0*	23*	1.25:1	
18+875	3.5	3.0	262.0*	21.5*	1.25:1	
18+900	3.5	2.5	261.0*	22*	1.25:1	
18+925	3.0	3.0	259.5*	26*	1.25:1	
18-950	2.5	3.0	258.5*	23*	1.25:1	
18+975	2.5	4.0	?**	?**	2:1	
19+000	2.0	3.5	?**	?**	2:1	
19+025	2.0	3.0			2:1	The peat layer (about 0.6 m to 0.9 m thick) underneath the widened embankment footprint shall be replaced with

Station	Proposed Road Widening[1]		Stabilizing Toe Berm		Proposed Widened Rock Fill Slope	Remarks
	Top Width	Widened Embankment Height	Top Elevation	Offset to Edge of Toe Berm from Centerline of Existing Road		
(m)	(m)	(m)	(m)	(m)	(H:V)	
						rock fill as per OPSD 203.030.
19+050	2.5	2.5	-	-	2:1	The peat layer (about 1.4 m to 1.8 m thick) underneath the widened embankment footprint shall be replaced with rock fill as per OPSD 203.030.
19+075	2.5	2.5	-	-	2:1	
19+100 (approx. culvert location)	2.5	2.5	-	-	2:1	
19+125	2.5	2.5	-	-	2:1	
19+150	2.5	2.5	-	-	2:1	
19+175	2.5	2.5	-	-	2:1	
19+200	2.0	3.5	-	-	2:1	

Table A Contd.



Proposed Widened Embankment Configuration (without Stabilizing Toe Berm)



Proposed Widened Embankment Configuration (with Stabilizing Toe Berm)

Notes:

1. The proposed road widening sections are provided by MTO on 30 March 2009.
2. The proposed widened embankment and stabilizing toe berm (where required) will be constructed almost close to the final elevation.
3. Staged construction, where a relatively long preloading period for porewater dissipation before placing a new fill layer, is not required.

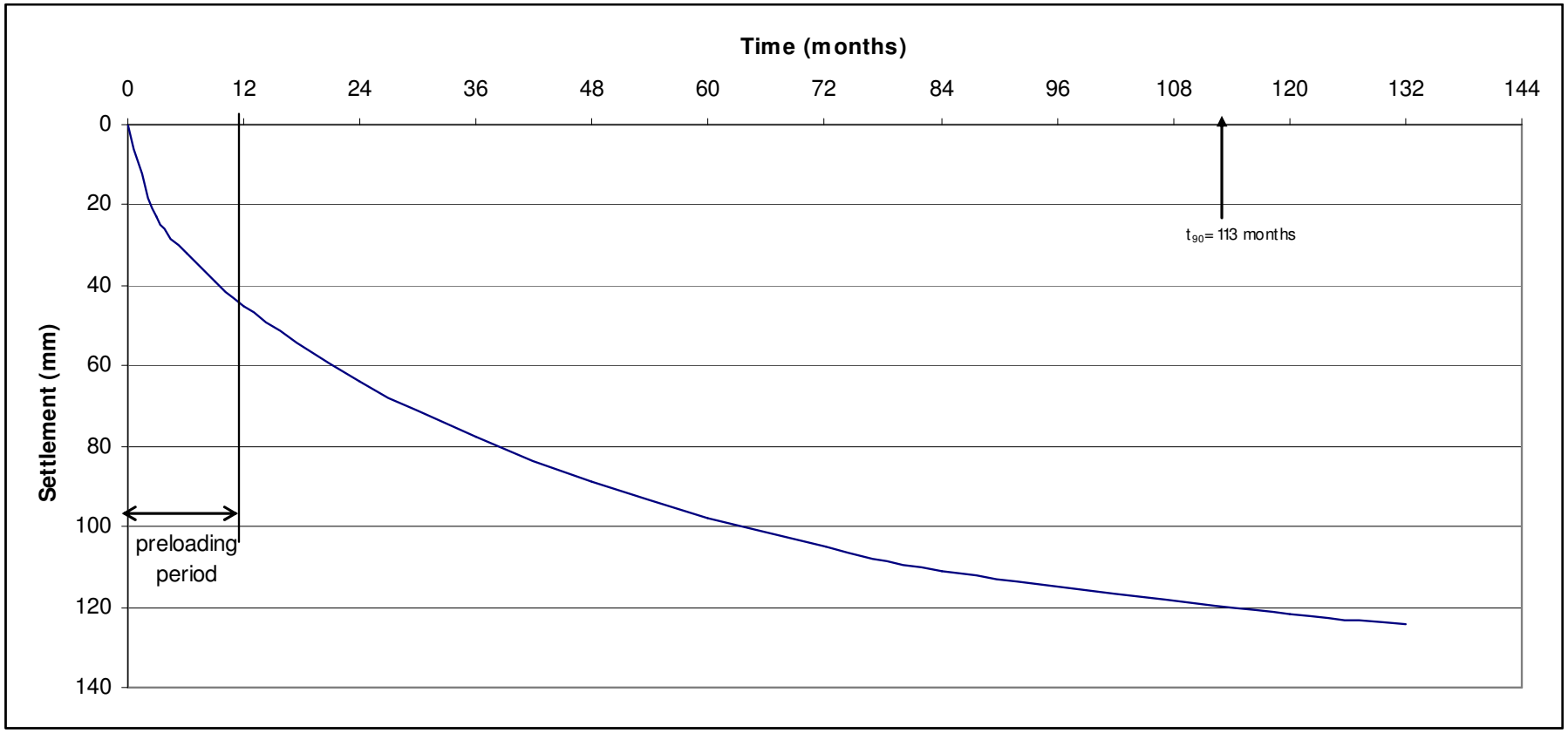
Table B Summary of Settlement Analyses

Station	Thickness of Compressible Layer (m)	Average Immediate Settlement - During Construction (mm)	c_c [1]	Consolidation Settlement (mm)				t_{90} (months)
				3 months Preloading Period	6 months Preloading Period	12 months Preloading Period	End of 10-Year Pavement Service[2]	
18+700 - 18+775	9	190	0.29	23	32	45	79	113
18+825 - 18+925	4	30	0.18	5	7	10	3	22
18+925 - 19+040	10	90	0.29	10	14	20	39	139
19+040 - 19+200	10	90	0.29	11	15	21	41	139

Notes:

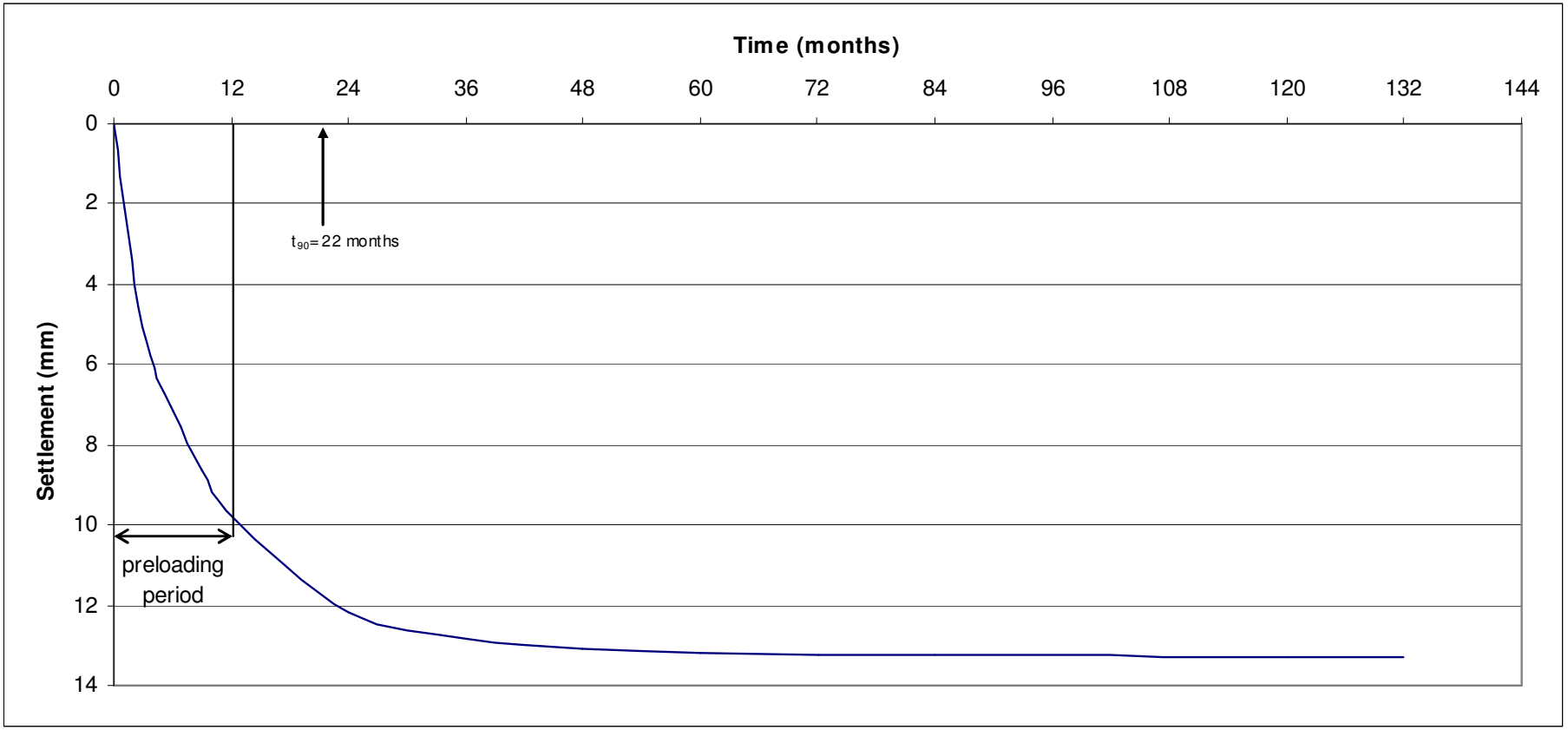
1) As per consolidation test results, c_c of 0.29 for silty clay and c_c of 0.18 for clayey silt.
2) Consolidation settlement over 10-year pavement service refers to remaining settlement after end of 12 month preloading period.

Figure A1 Station 18+700 to 18+775 – Settlement versus Time Plot



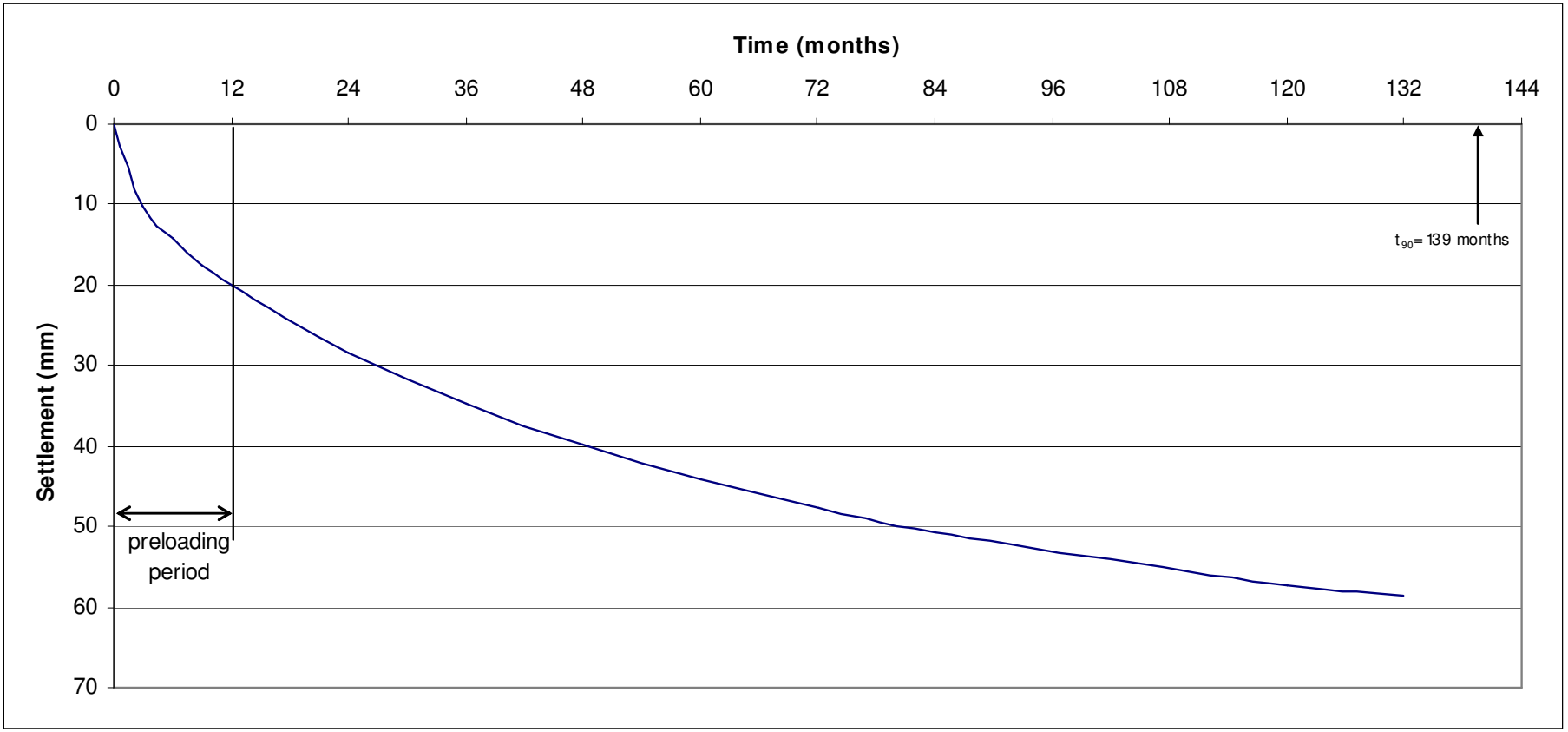
Time “0” refers to “End of Construction”

Figure A2 Station 18+825 to 18+925 – Settlement versus Time Plot



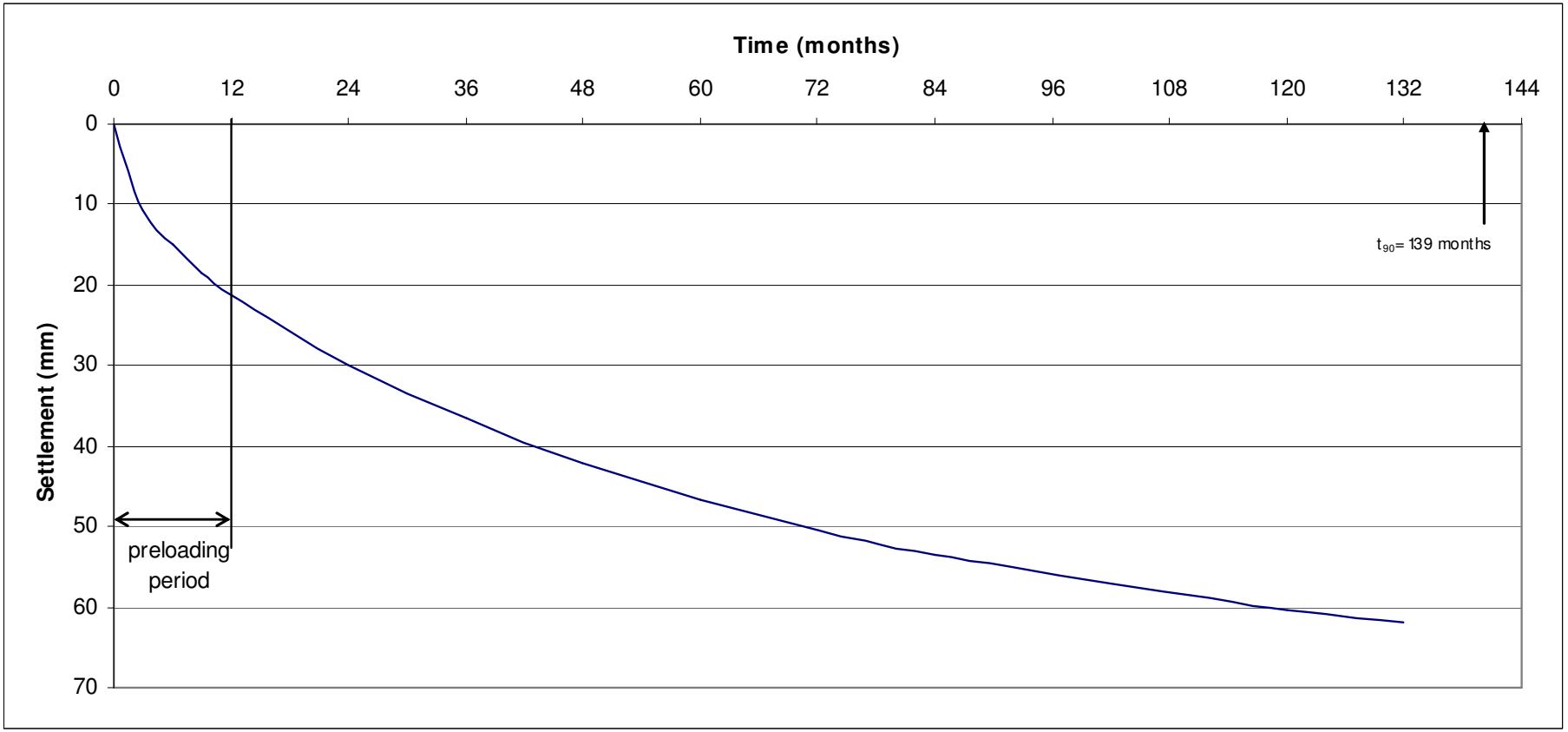
Time “0” refers to “End of Construction”

Figure A3 Station 18+925 to 19+040 – Settlement versus Time Plot

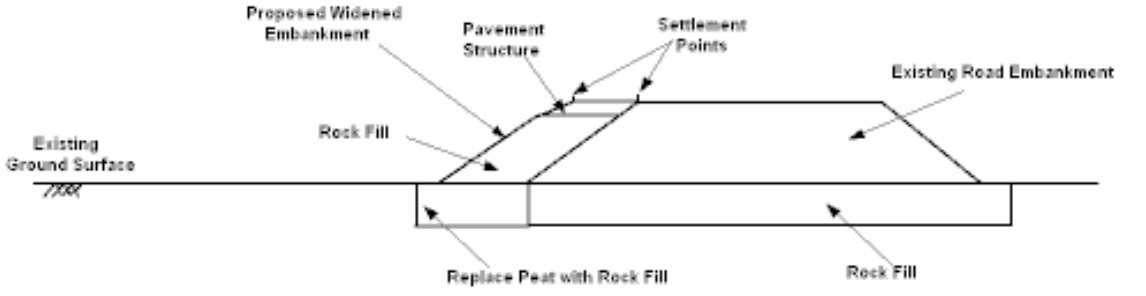


Time “0” refers to “End of Construction”

Figure A4 Station 19+040 to 19+200 – Settlement versus Time Plot



Time “0” refers to “End of Construction”



Notes:

1. The settlement points should be installed at the edge of the existing asphalt pavement and at the edge of the widened road with 25 m spacing.
2. The suggested monitoring program is as follows:
   a. Daily during construction.
   b. Once a week after completing (a) for 3 months.
   c. Once a month after completing (b), until termination by Engineer.

Figure B Settlement Monitoring Locations

APPENDIX - X

Table X1.1 - Summary of Slope Stability Analyses

Station (m)	Description of Analysis	Reference	Calculated Factor of Safety (FOS)	Remarks
18+725 (Based on BH 3)	Existing Condition – Global critical slip surface	Figure X1.1a	1.11	
	Existing Condition – Slip surface within the westbound lane	Figure X1.1b	1.59	
	End of Construction - 4 m widening with 1.25H:1V rock fill slope	Figure X1.2	1.41	
	End of Construction - 4 m widening with 1.5H:1V rock fill slope	Figure X1.3	1.40	
	End of Construction - 4 m widening with 2H:1V rock fill slope	Figure X1.4	1.41	***Recommended because of wider extent of peat to be replaced by rock fill and high groundwater level.***[1]
18+875 (Based on BH 4)	Existing Condition – Global critical slip surface	Figure X2.1a	1.58	
	Existing Condition – Slip surface within the westbound lane	Figure X2.1b	1.61	
	End of Construction - 3 m widening with 1.25H:1V rock fill slope	Figure X2.2	1.46	***Recommended because of lower driving force on existing rock berm.***[1]
	End of Construction - 3 m widening with 1.5H:1V rock fill slope	Figure X2.3	1.45	
	End of Construction - 3 m widening with 2H:1V rock fill slope	Figure X2.4	1.45	
18+975 (Based on BH 6 & 7)	Existing Condition – Global critical slip surface	Figure X3.1a	1.19	
	Existing Condition – Slip surface within the westbound lane	Figure X3.1b	1.39	
	End of Construction - 2.5 m widening with 1.25H:1V rock fill slope	Figure X3.2	1.31	
	End of Construction - 2.5 m widening with 1.5H:1V rock fill slope	Figure X3.3	1.31	

Station (m)	Description of Analysis	Reference	Calculated Factor of Safety (FOS)	Remarks
	End of Construction - 2.5 m widening with 2H:1V rock fill slope	Figure X3.4	1.32	***Recommended because of high groundwater level.***[1]
19+100 (Based on BH 9)	Existing Condition – Global critical slip surface	Figure X4.1a	1.14	
	Existing Condition – Slip surface within the westbound lane	Figure X4.1b	1.32	
	End of Construction - 2.5 m widening with 1.25H:1V rock fill slope	Figure X4.2	1.32	
	End of Construction - 2.5 m widening with 1.5H:1V rock fill slope	Figure X4.3	1.31	
	End of Construction - 2.5 m widening with 2H:1V rock fill slope	Figure X4.4	1.33	***Recommended because of high groundwater level.***[1]

Note:

1. The FOS of the widened road is lower than the FOS of the existing road. This could result in some movement of the slope during and/or after construction.

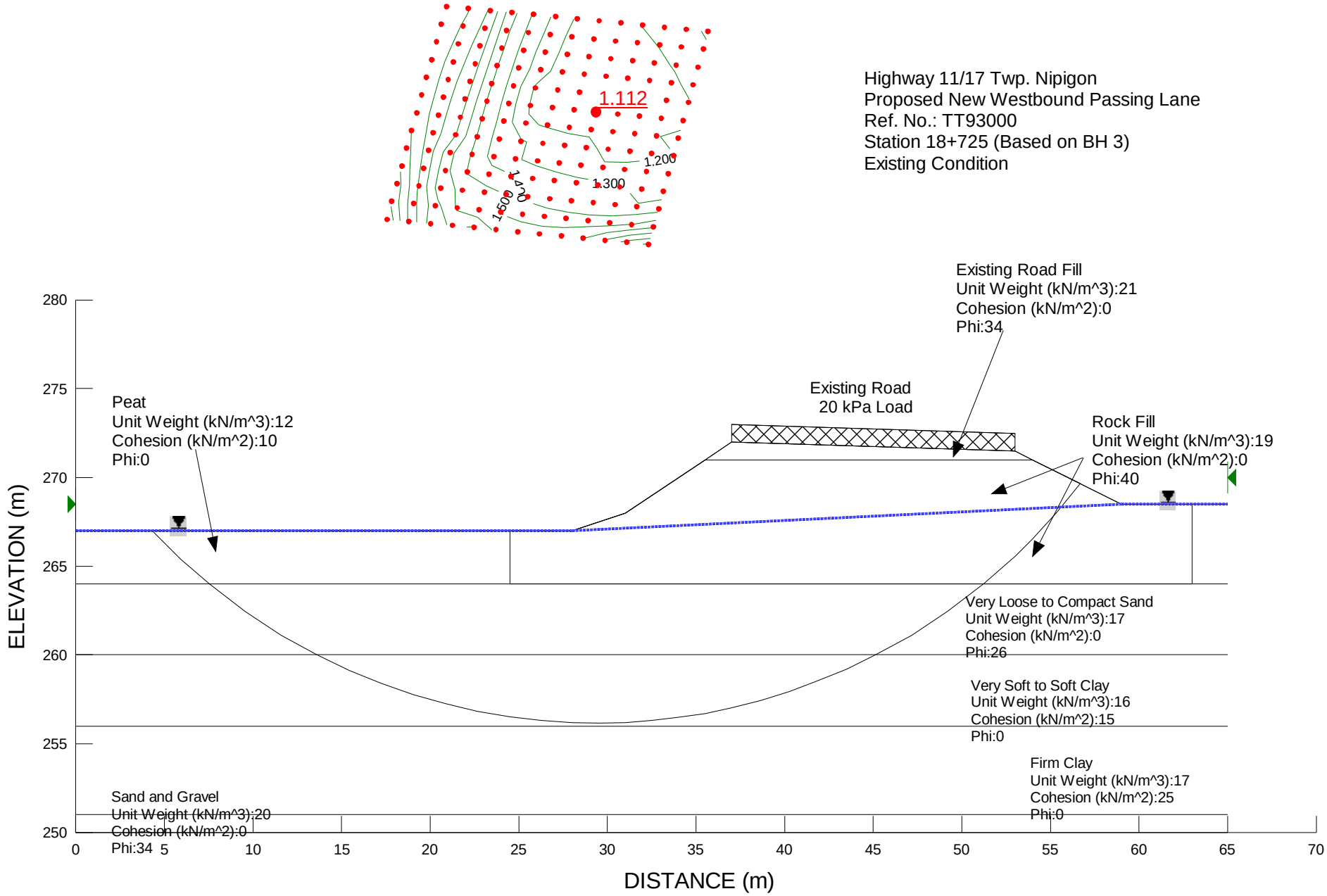


Figure X1.1a Slope Stability Analysis of Section at Station 18+725
(Existing Condition - Critical Slip Surface)

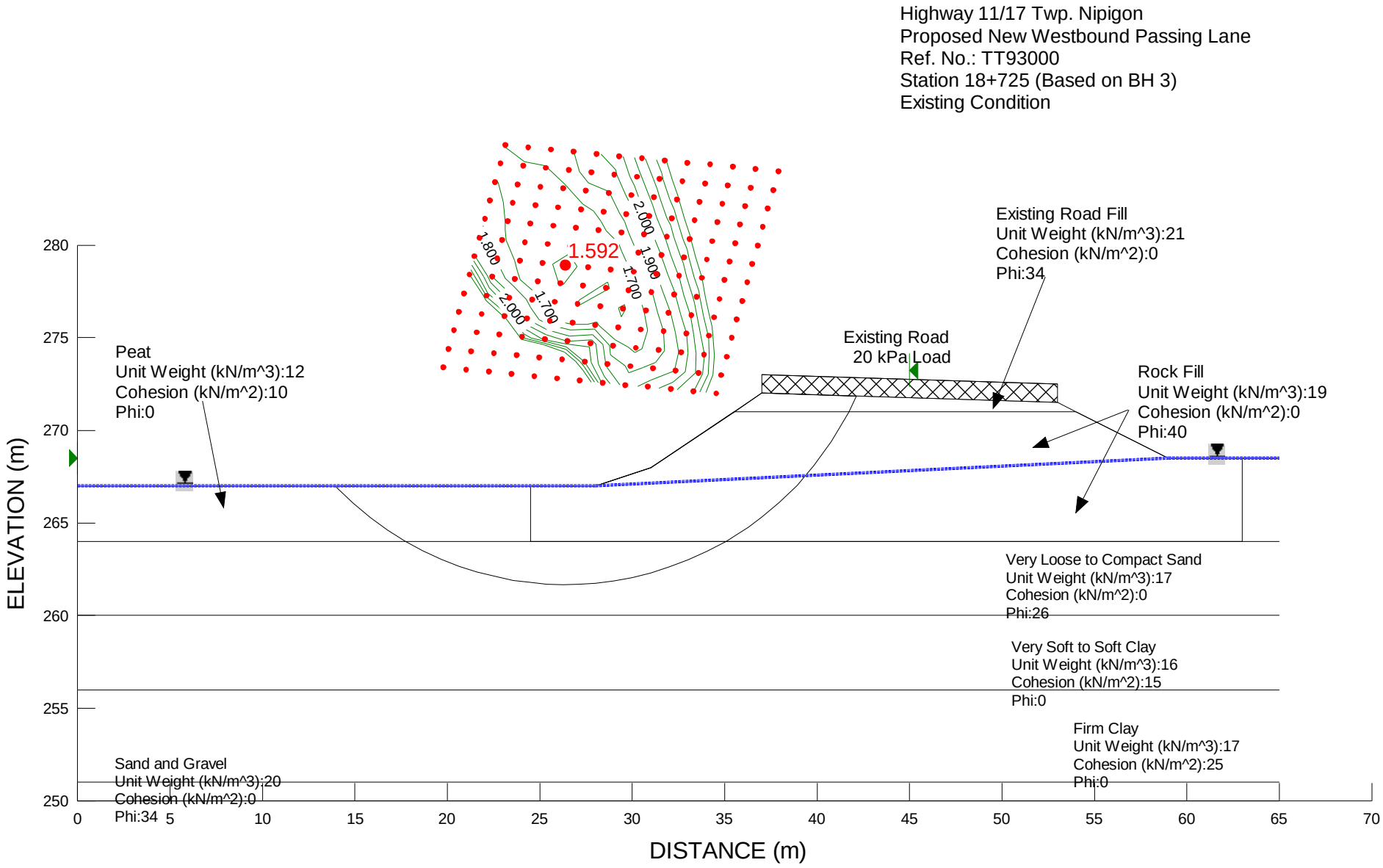


Figure X1.1b Slope Stability Analysis of Section at Station 18+725
(Existing Condition - Slip Surface within Westbound Lane)

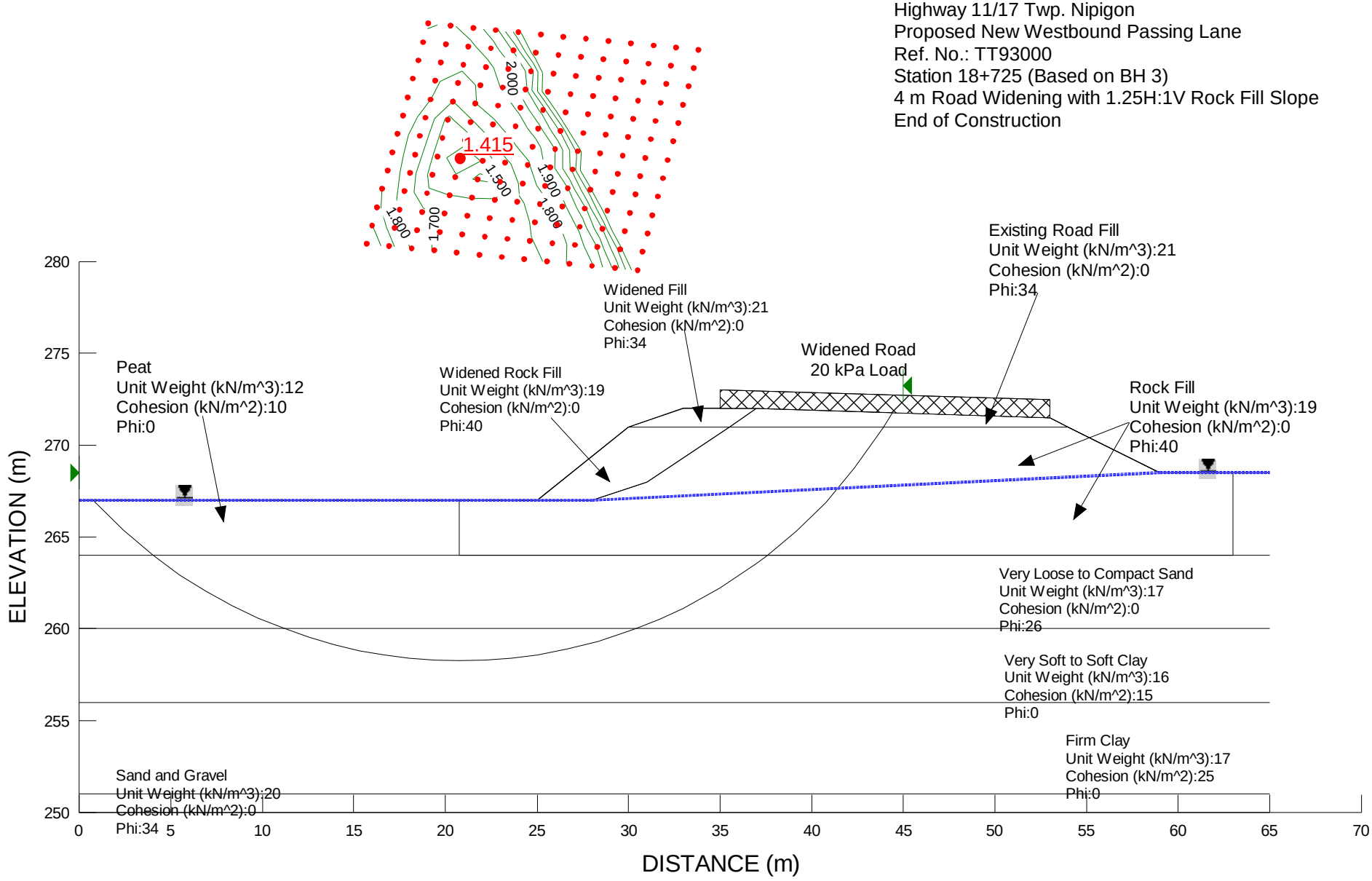


Figure X1.2 Slope Stability Analysis of Section at Station 18+725 with 4 m Road Widening (End of Construction - Slip Surface within Westbound Lane)

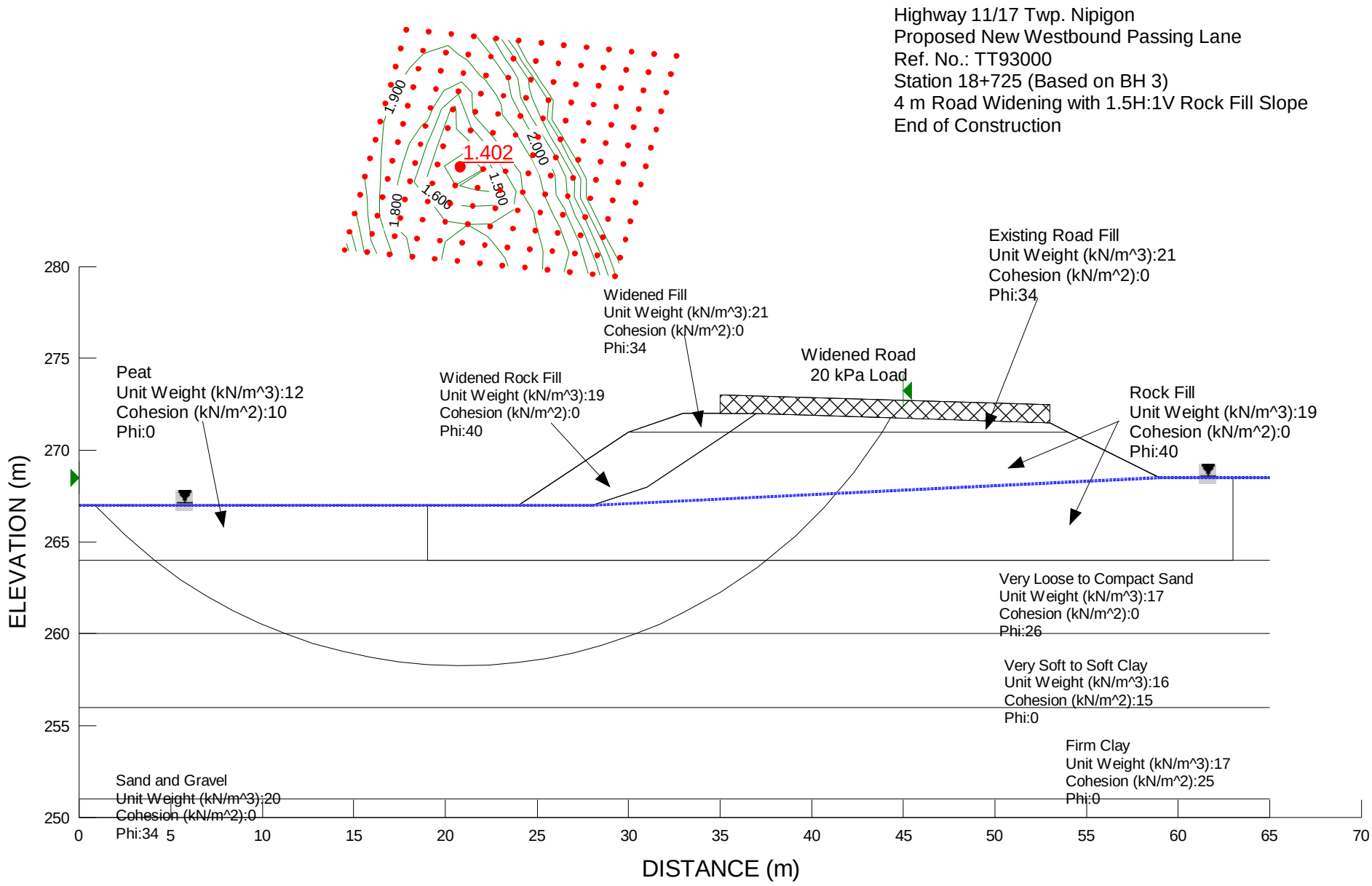


Figure X1.3 Slope Stability Analysis of Section at Station 18+725 with 4 m Road Widening (End of Construction - Slip Surface within Westbound Lane)

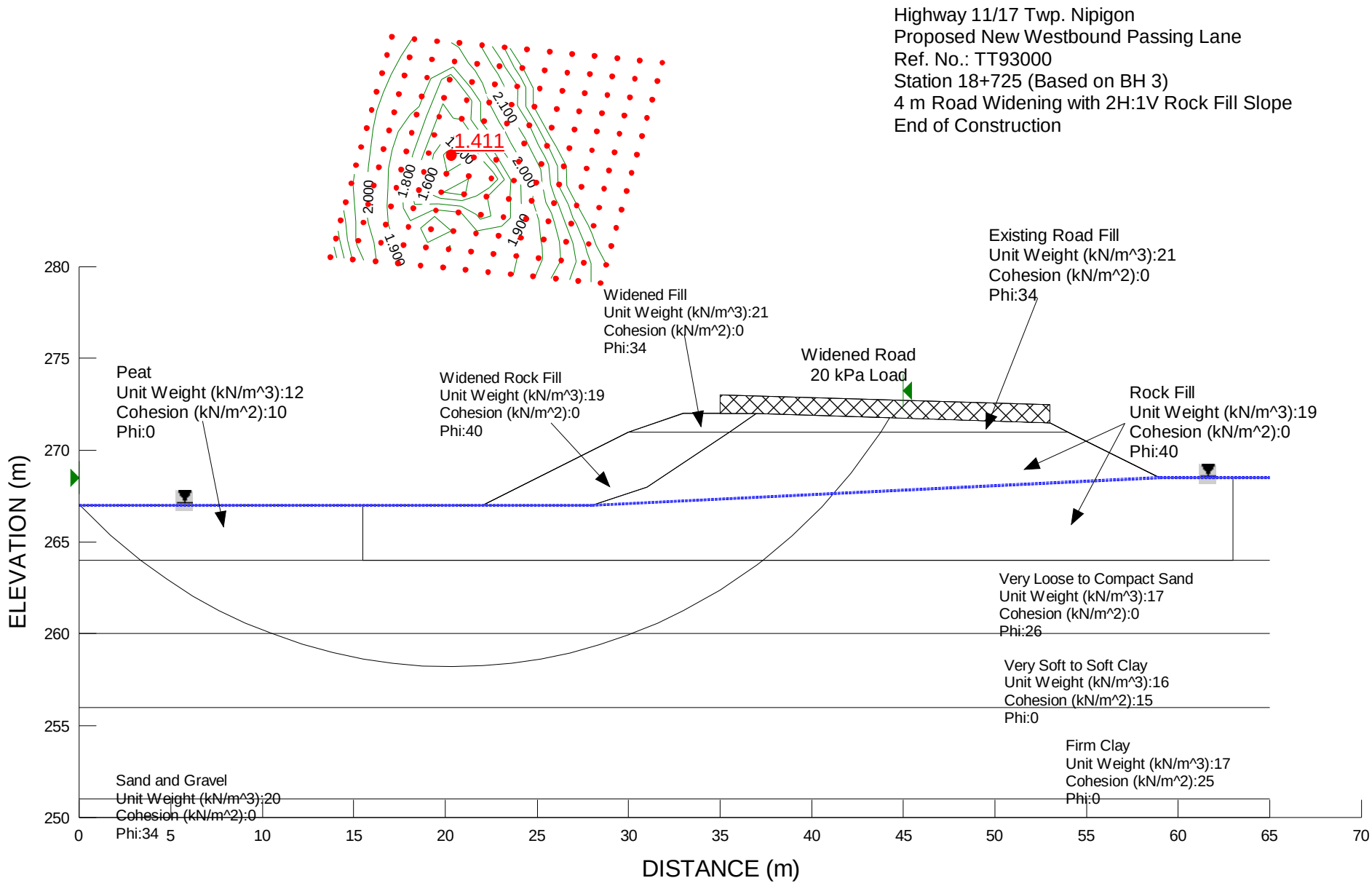


Figure X1.4 Slope Stability Analysis of Section at Station 18+725 with 4 m Road Widening (End of Construction - Slip Surface within Westbound Lane)

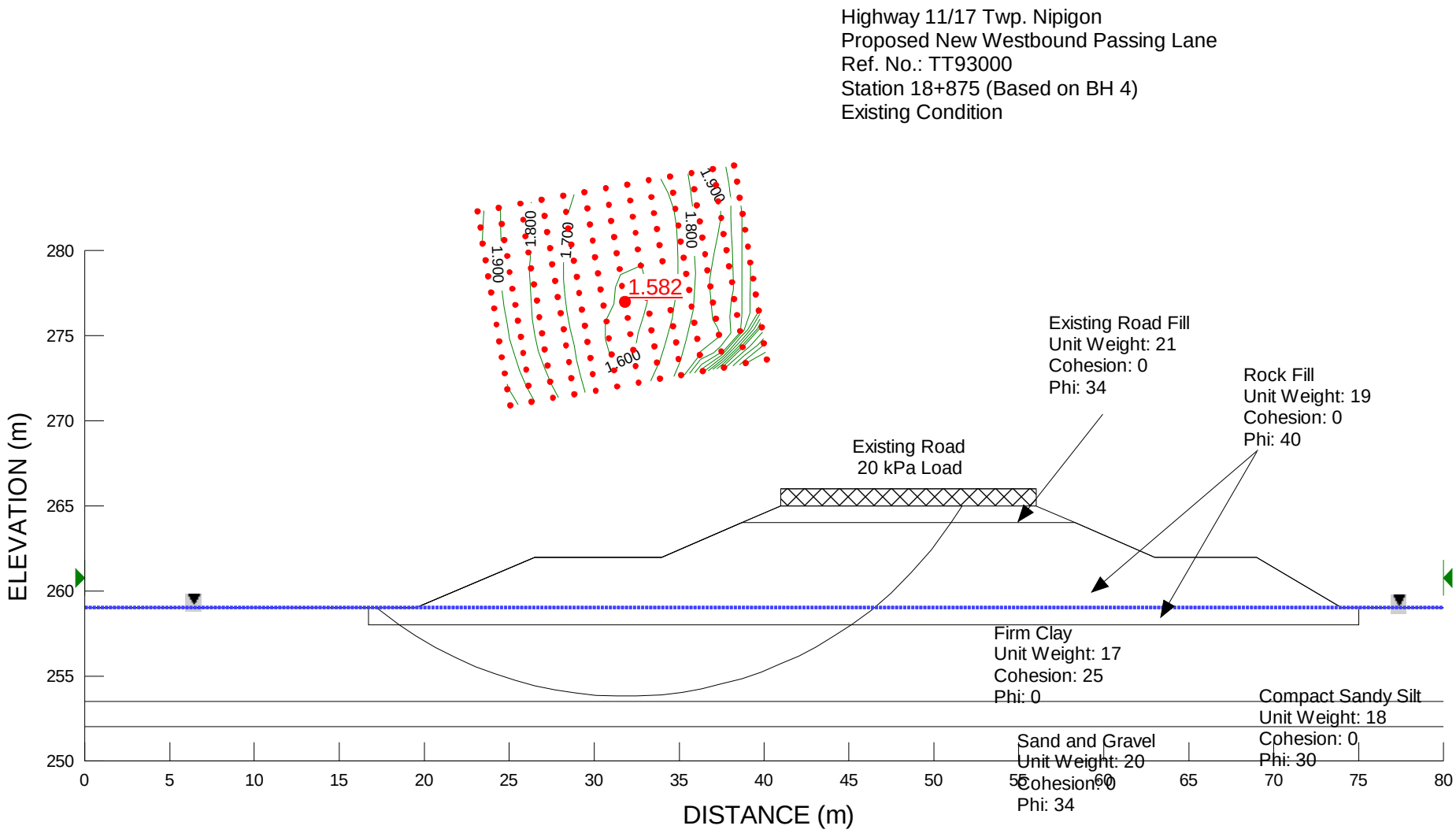


Figure X2.1a Slope Stability Analysis of Section at Station 18+875 (Existing Condition - Critical Slip Surface)

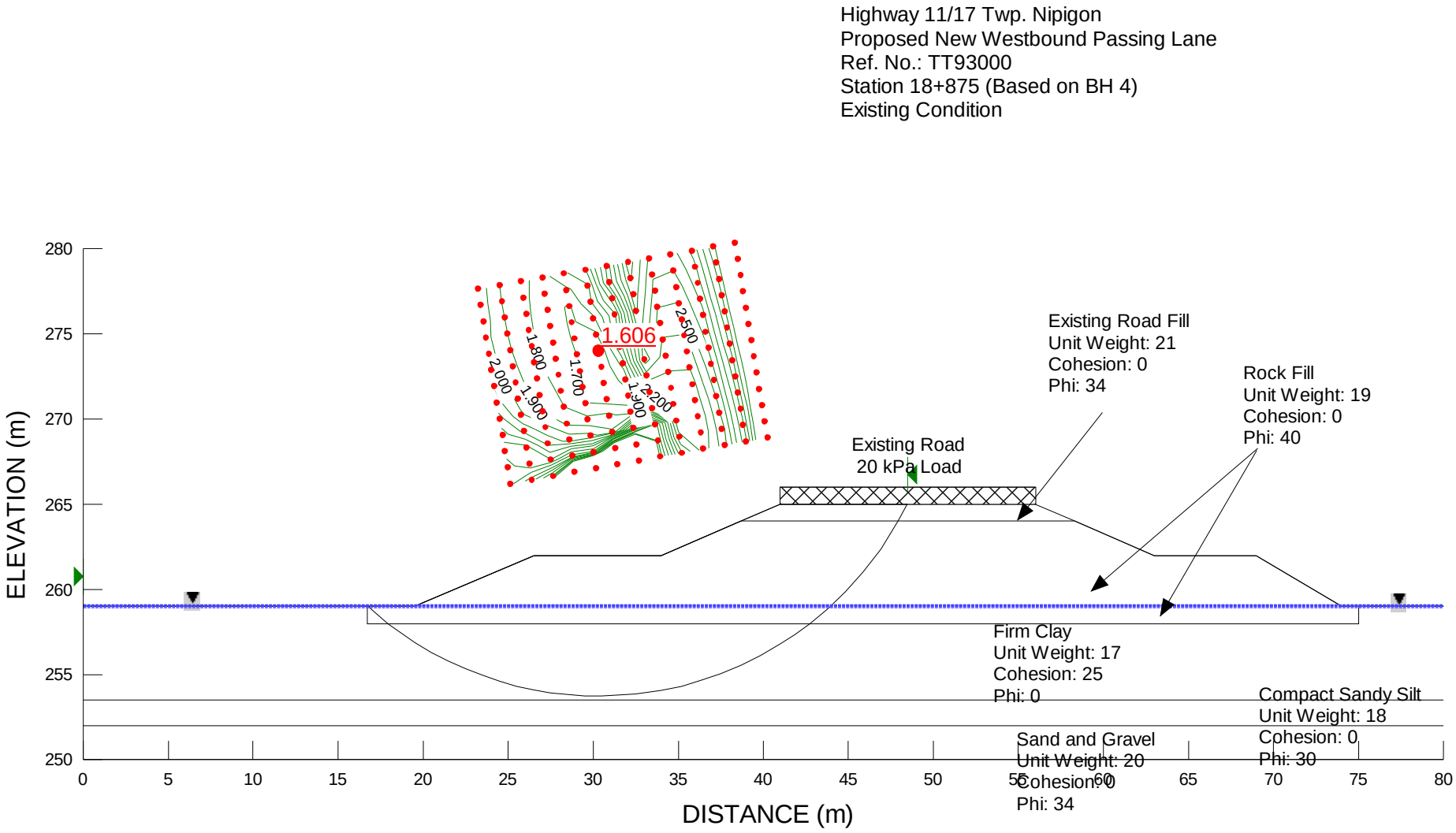


Figure X2.1b Slope Stability Analysis of Section at Station 18+875
(Existing Condition - Slip Surface within Westbound Lane)

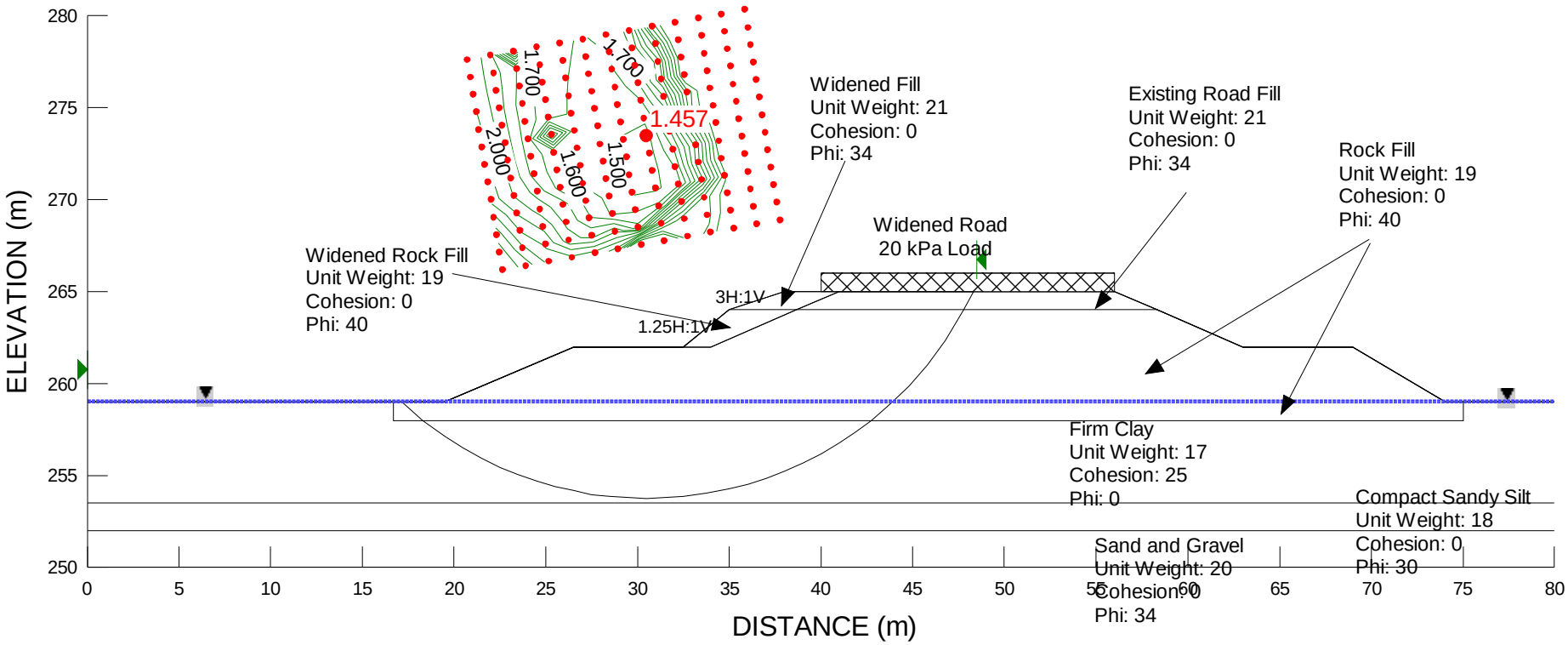


Figure X2.2 Slope Stability Analysis of Section at Station 18+875 - 3.0 m Road Widening (End of Construction - Slip Surface within Westbound Lane)

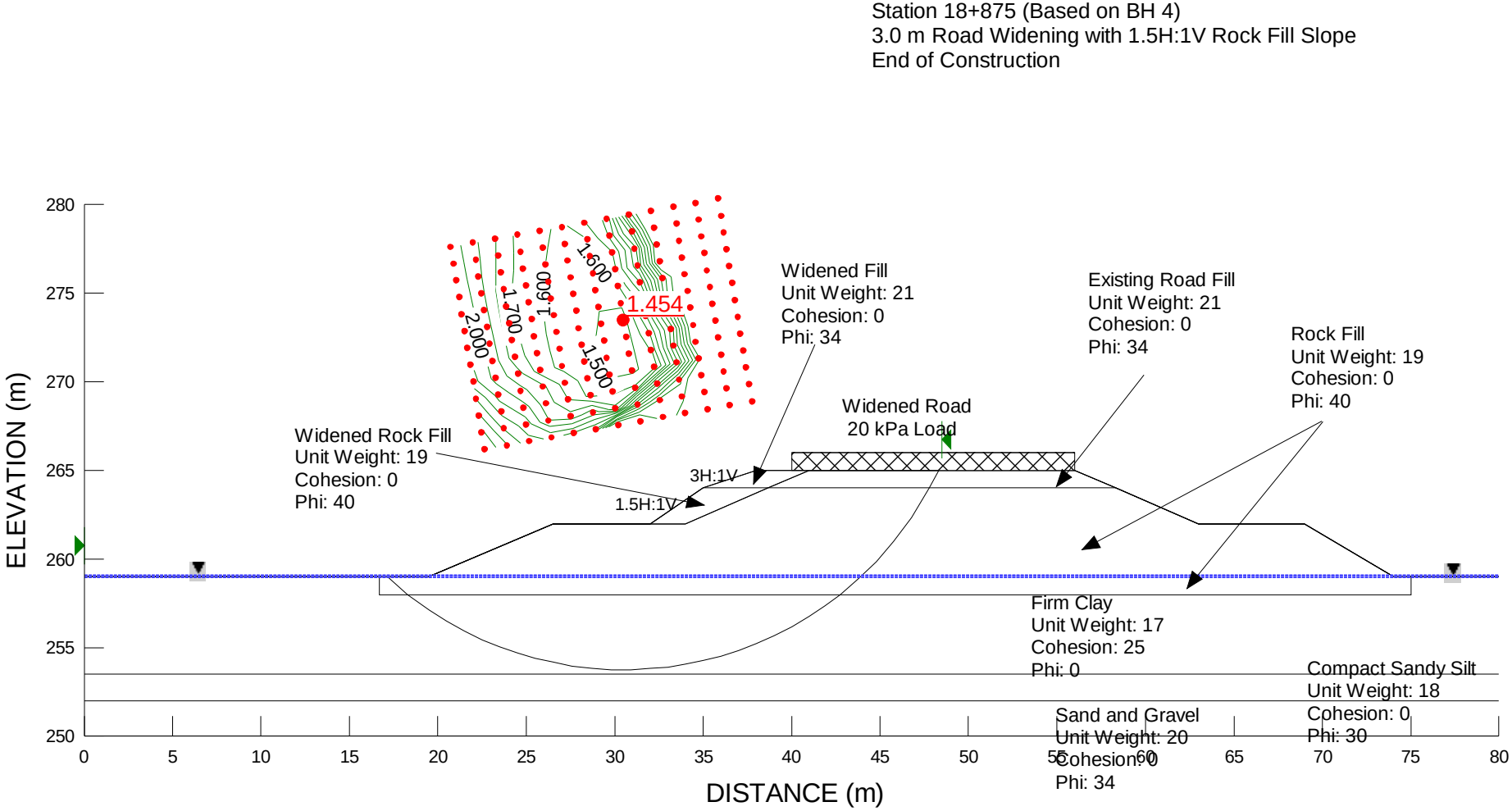


Figure X2.3 Slope Stability Analysis of Section at Station 18+875 - 3.0 m Road Widening (End of Construction - Slip Surface within Westbound Lane)

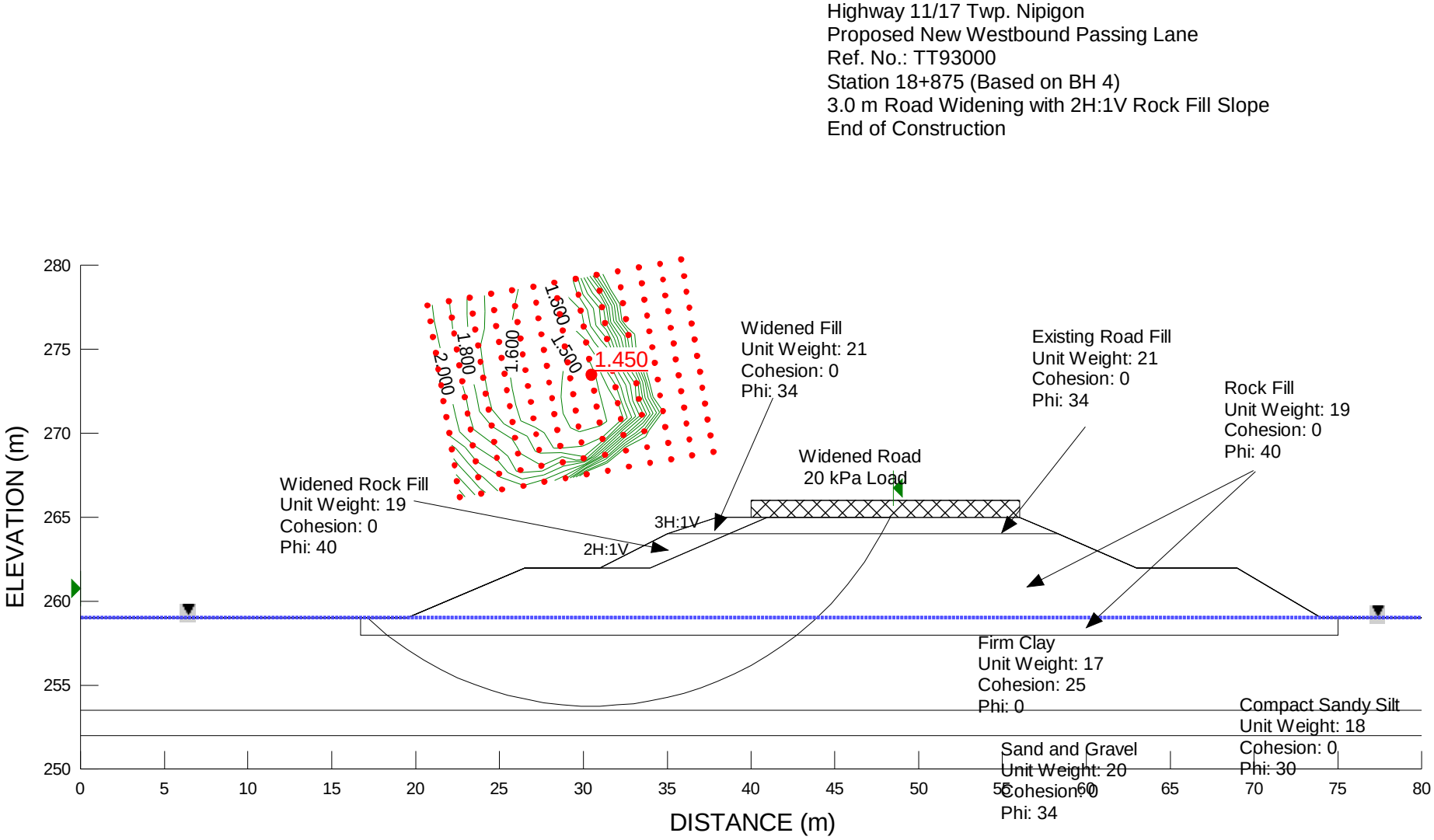


Figure X2.4 Slope Stability Analysis of Section at Station 18+875 - 3.0 m Road Widening (End of Construction - Slip Surface within Westbound Lane)

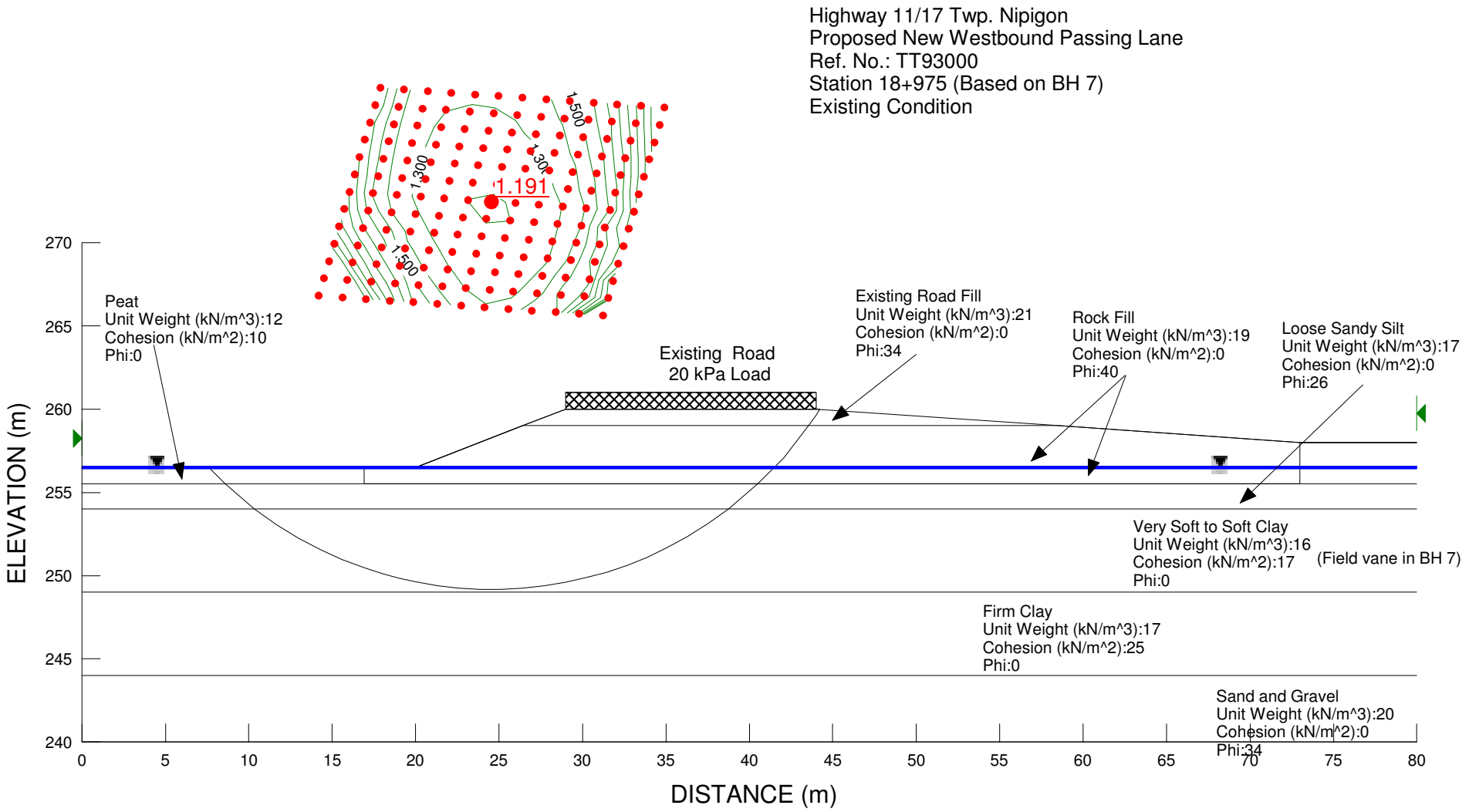


Figure X3.1a Slope Stability Analysis of Section at Station 18+975 (Existing Condition - Critical Slip Surface)

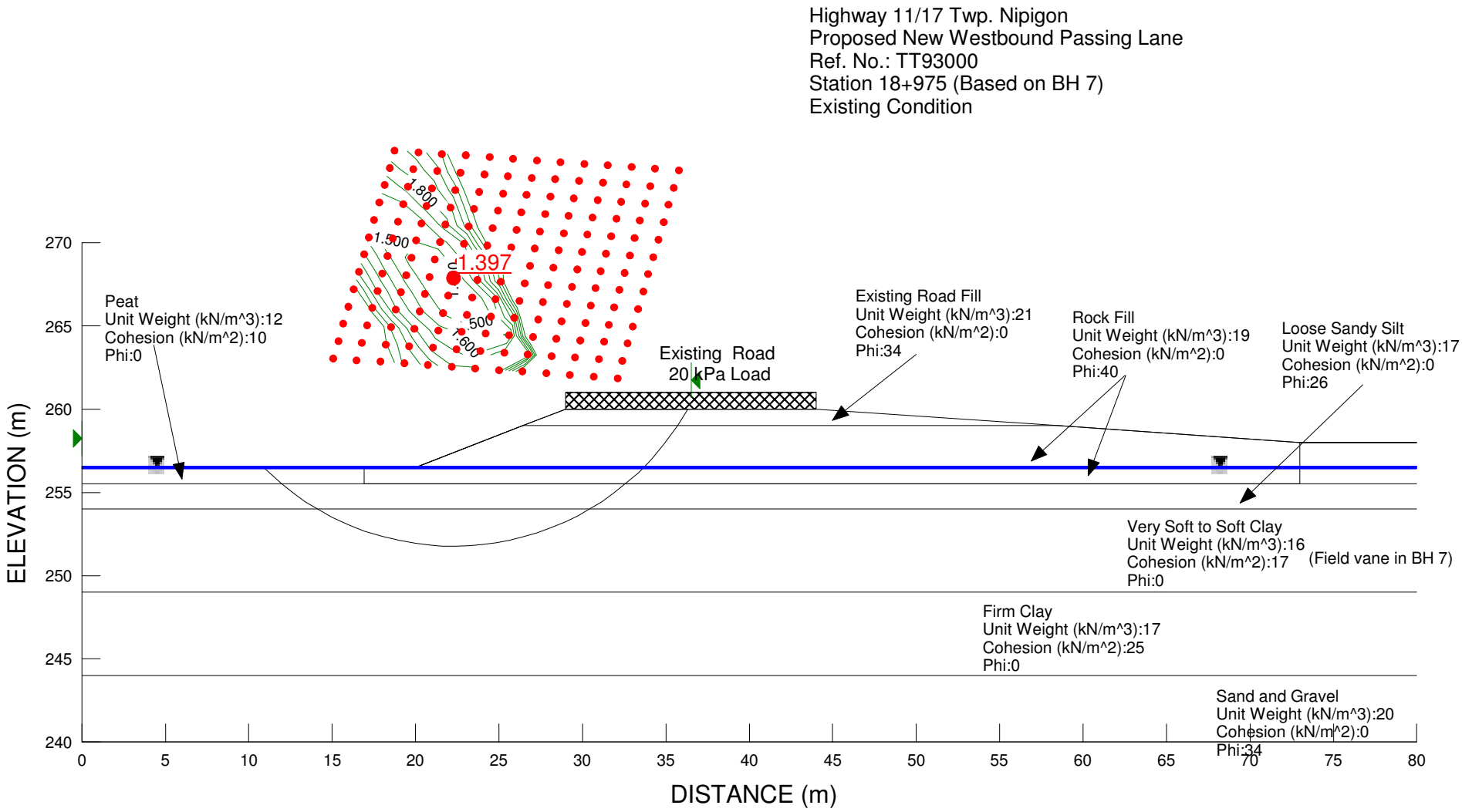


Figure X3.1b Slope Stability Analysis of Section at Station 18+975
(Existing Condition - Slip Surface within Westbound Lane)

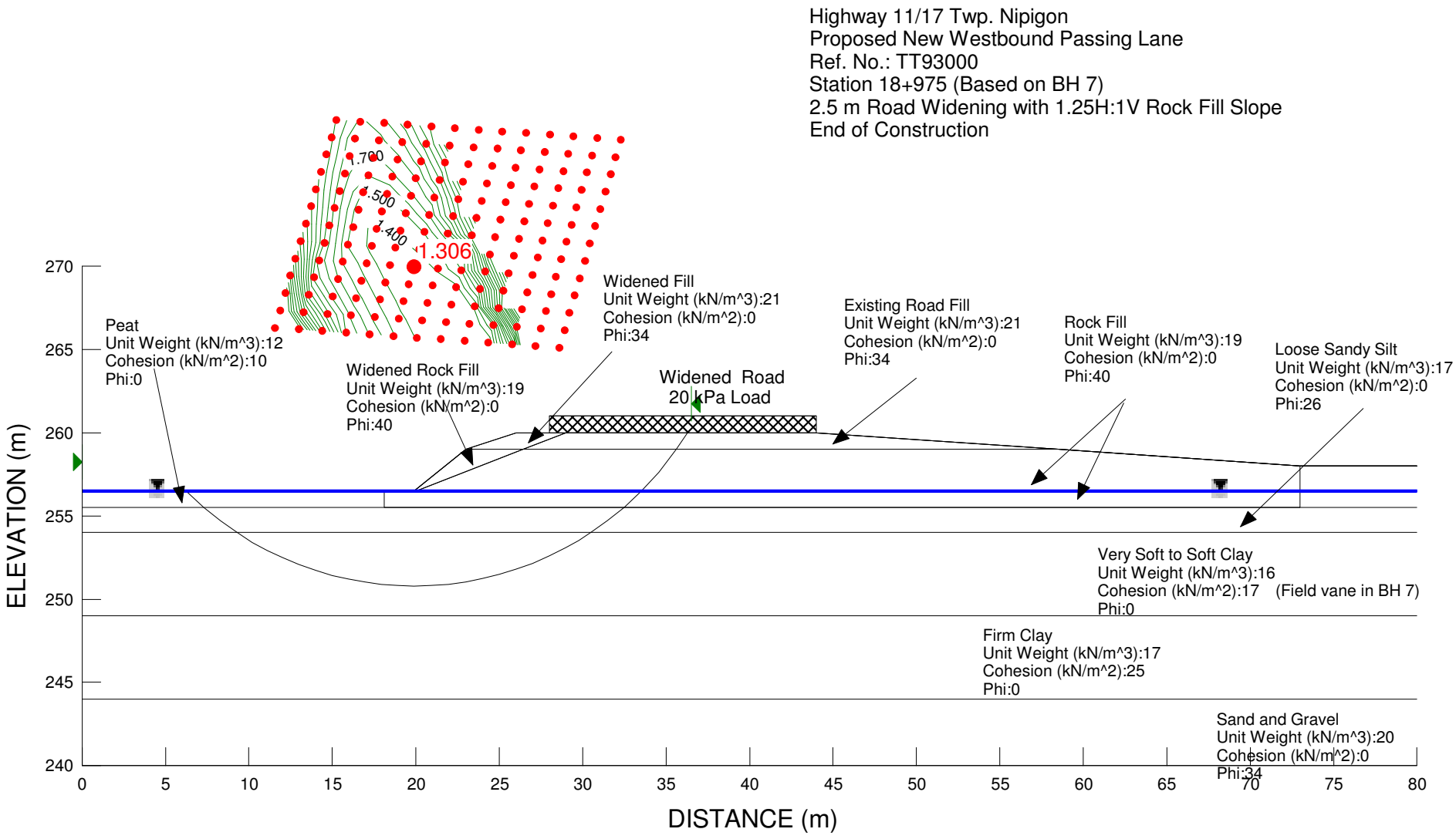


Figure X3.2 Slope Stability Analysis of Section at Station 18+975 - 2.5 m Road Widening (End of Construction - Slip Surface within Westbound Lane)

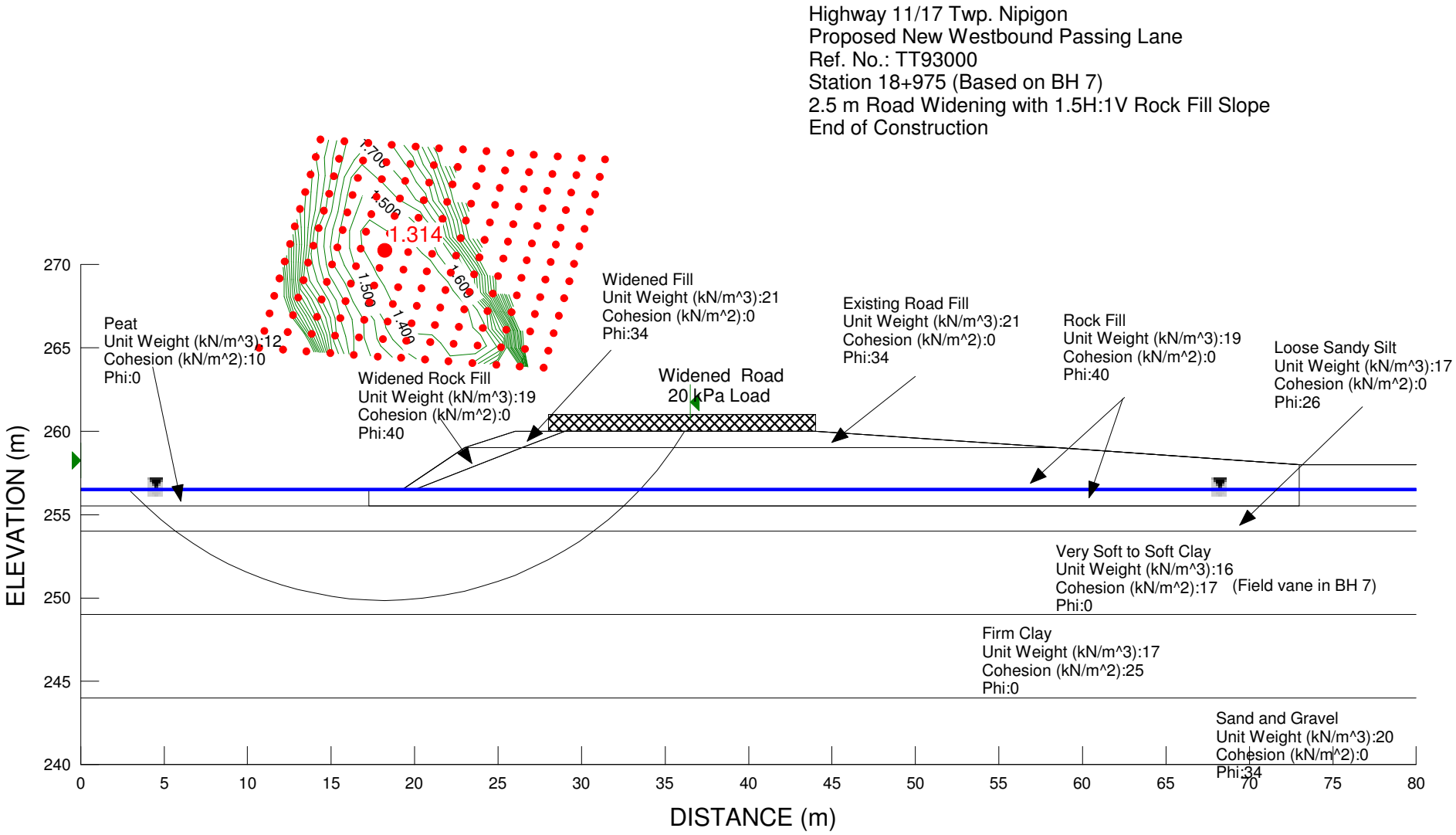


Figure X3.3 Slope Stability Analysis of Section at Station 18+975 - 2.5 m Road Widening (End of Construction - Slip Surface within Westbound Lane)

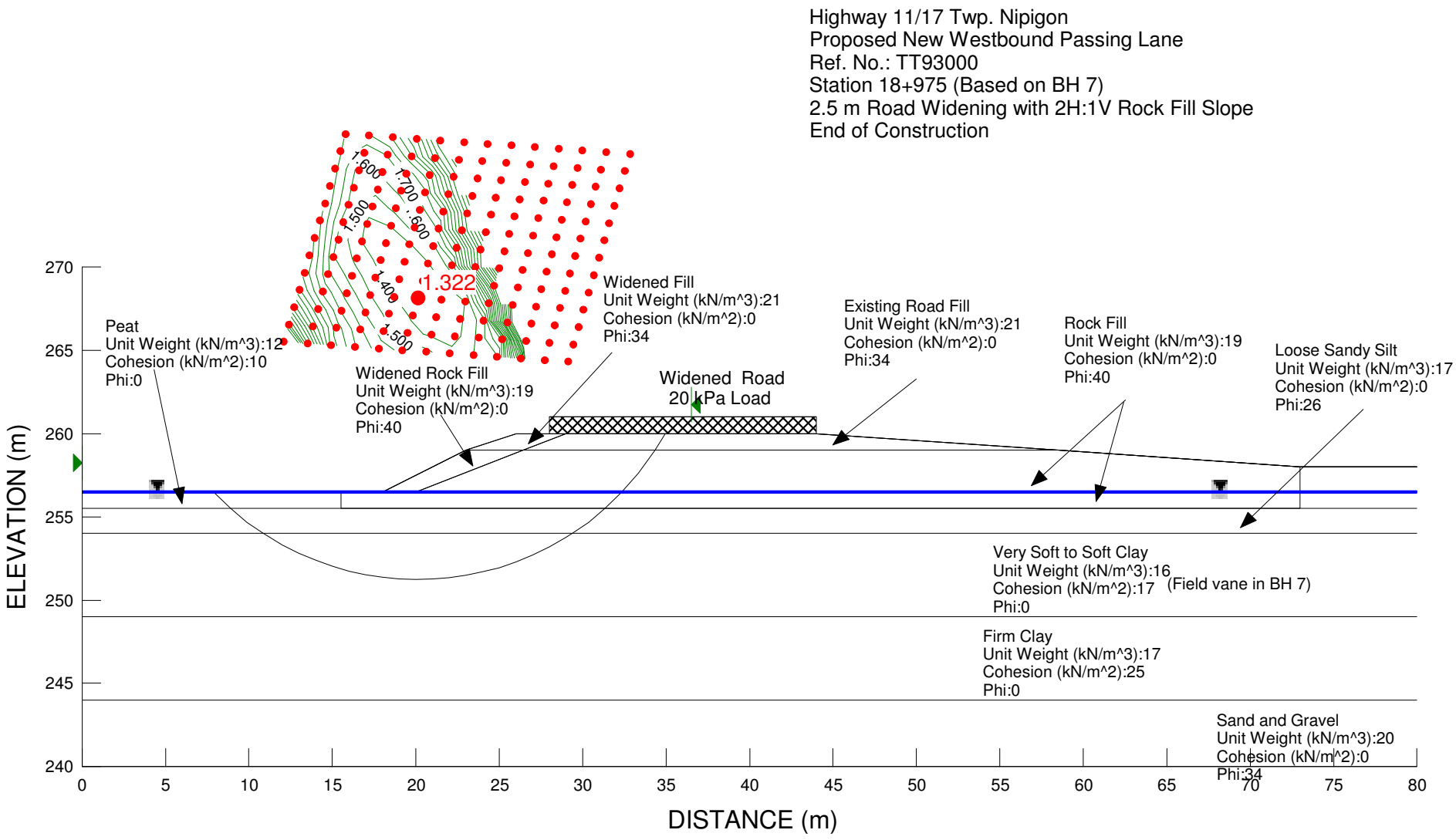


Figure X3.4 Slope Stability Analysis of Section at Station 18+975 - 2.5 m Road Widening (End of Construction - Slip Surface within Westbound Lane)

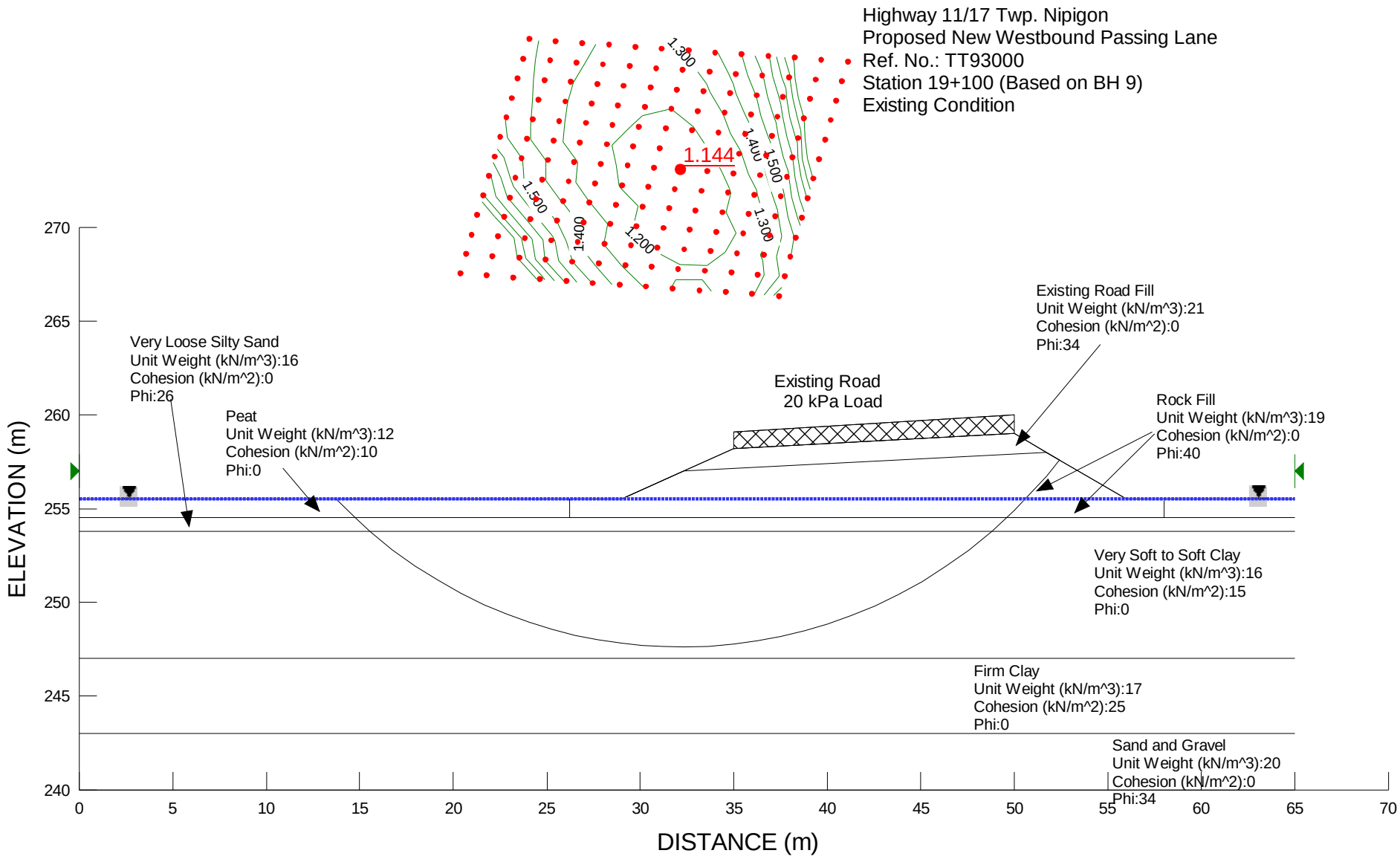


Figure X4.1a Slope Stability Analysis of Section at Station 19+100
(Existing Condition - Critical Slip Surface)

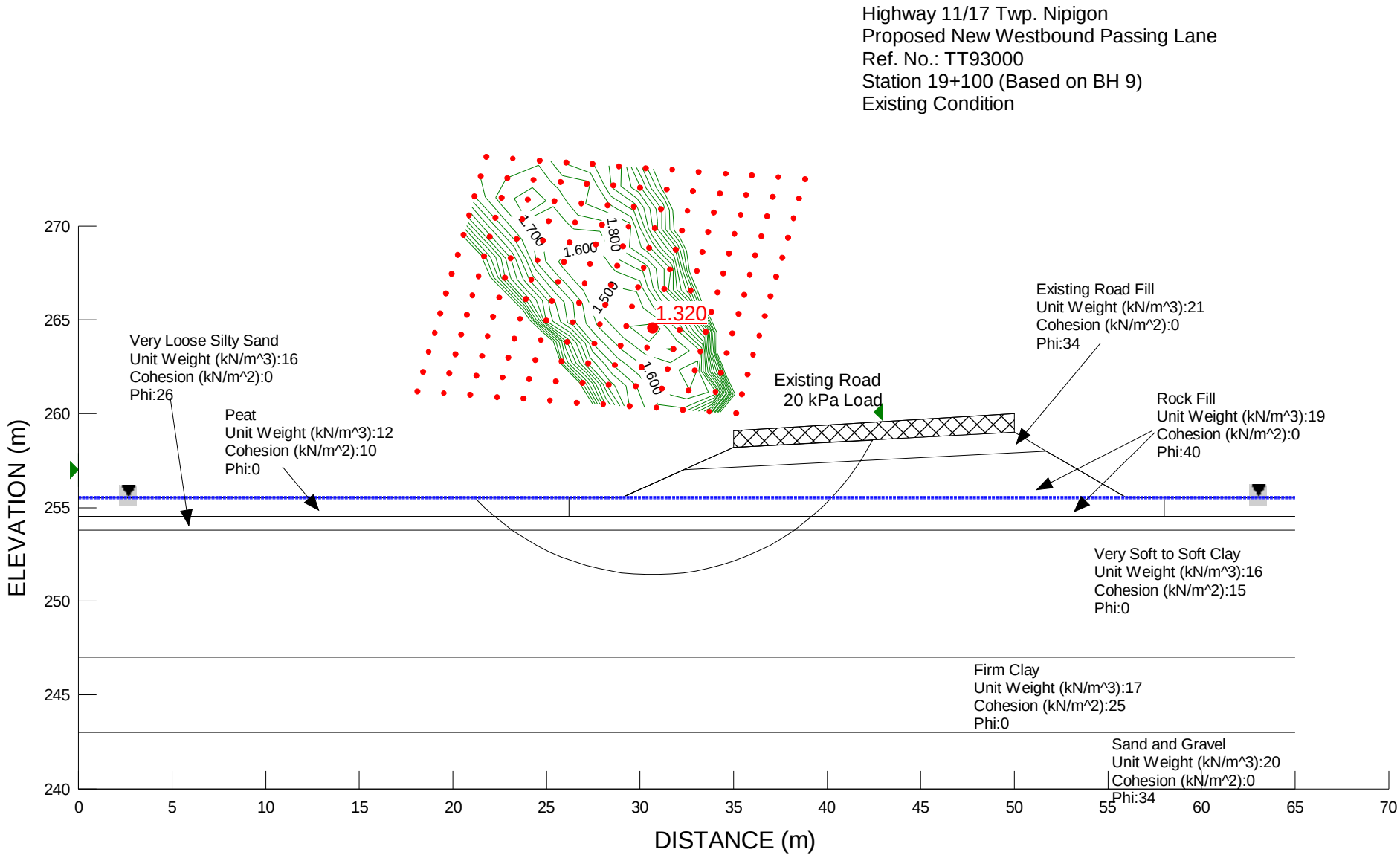


Figure X4.1b Slope Stability Analysis of Section at Station 19+100 (Existing Condition - Slip Surface within Westbound Lane)

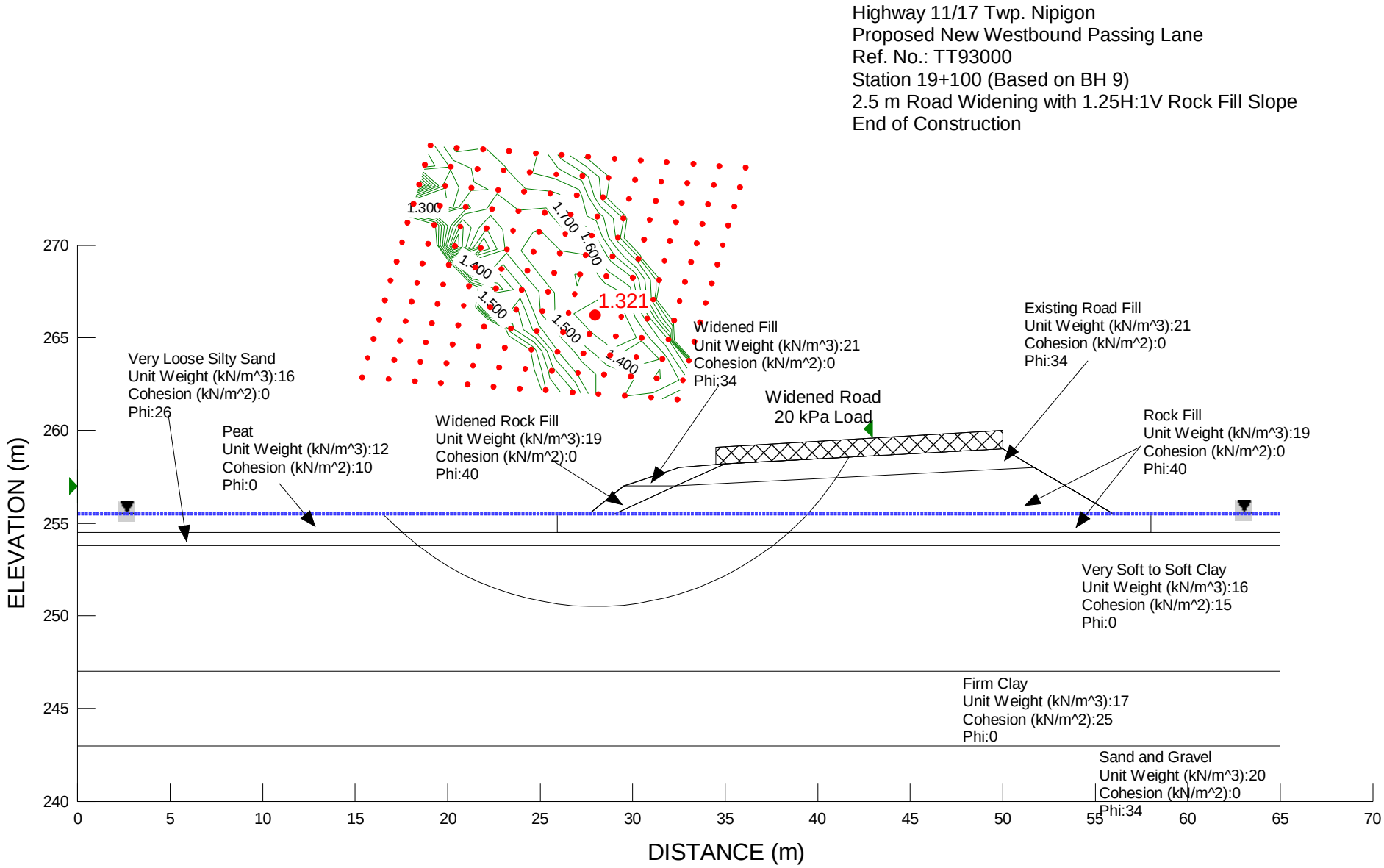


Figure X4.2 Slope Stability Analysis of Section at Station 19+100 with 2.5 m Road Widening (End of Construction - Slip Surface within Westbound Lane)

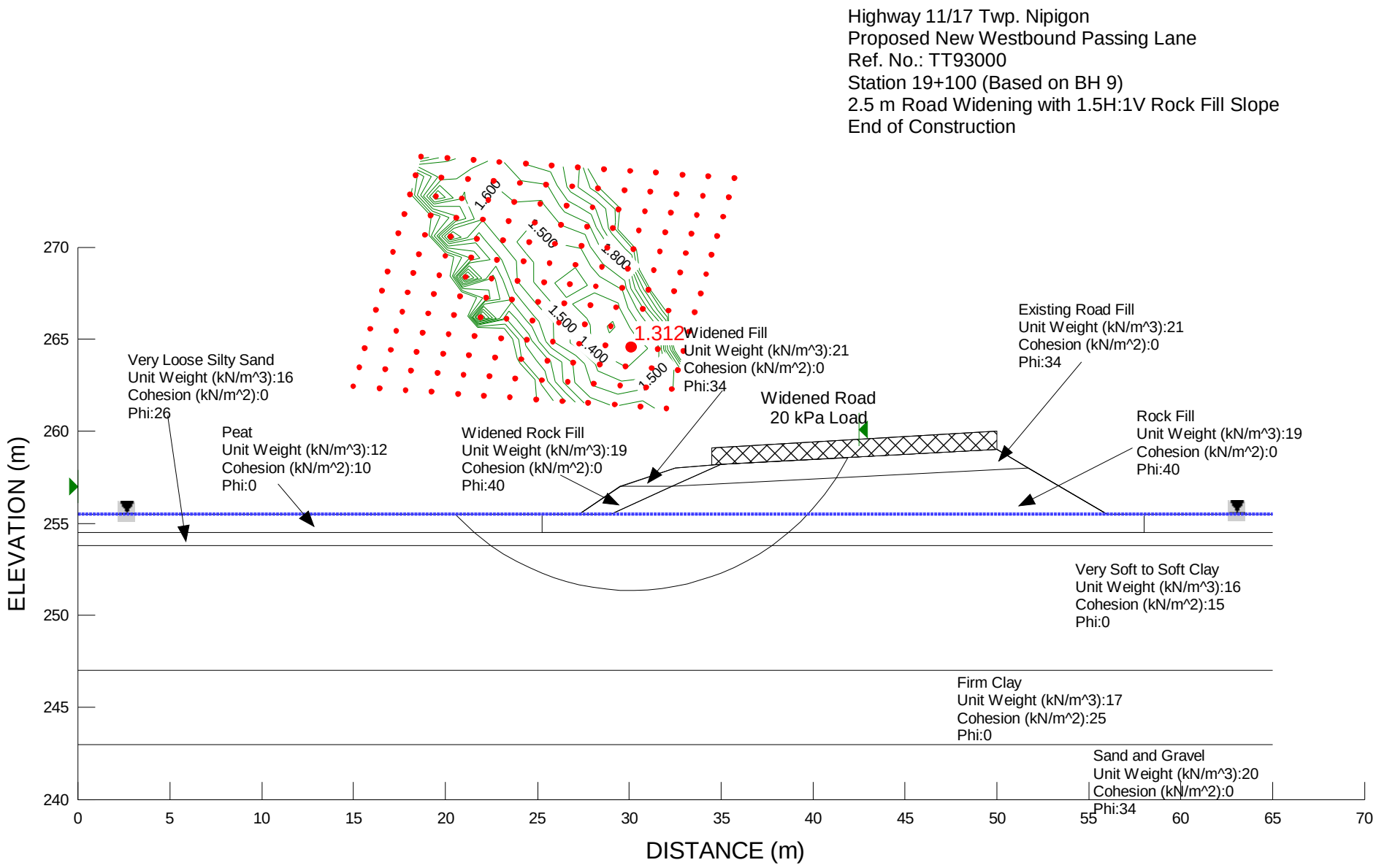


Figure X4.3 Slope Stability Analysis of Section at Station 19+100 with 2.5 m Road Widening (End of Construction - Slip Surface within Westbound Lane)

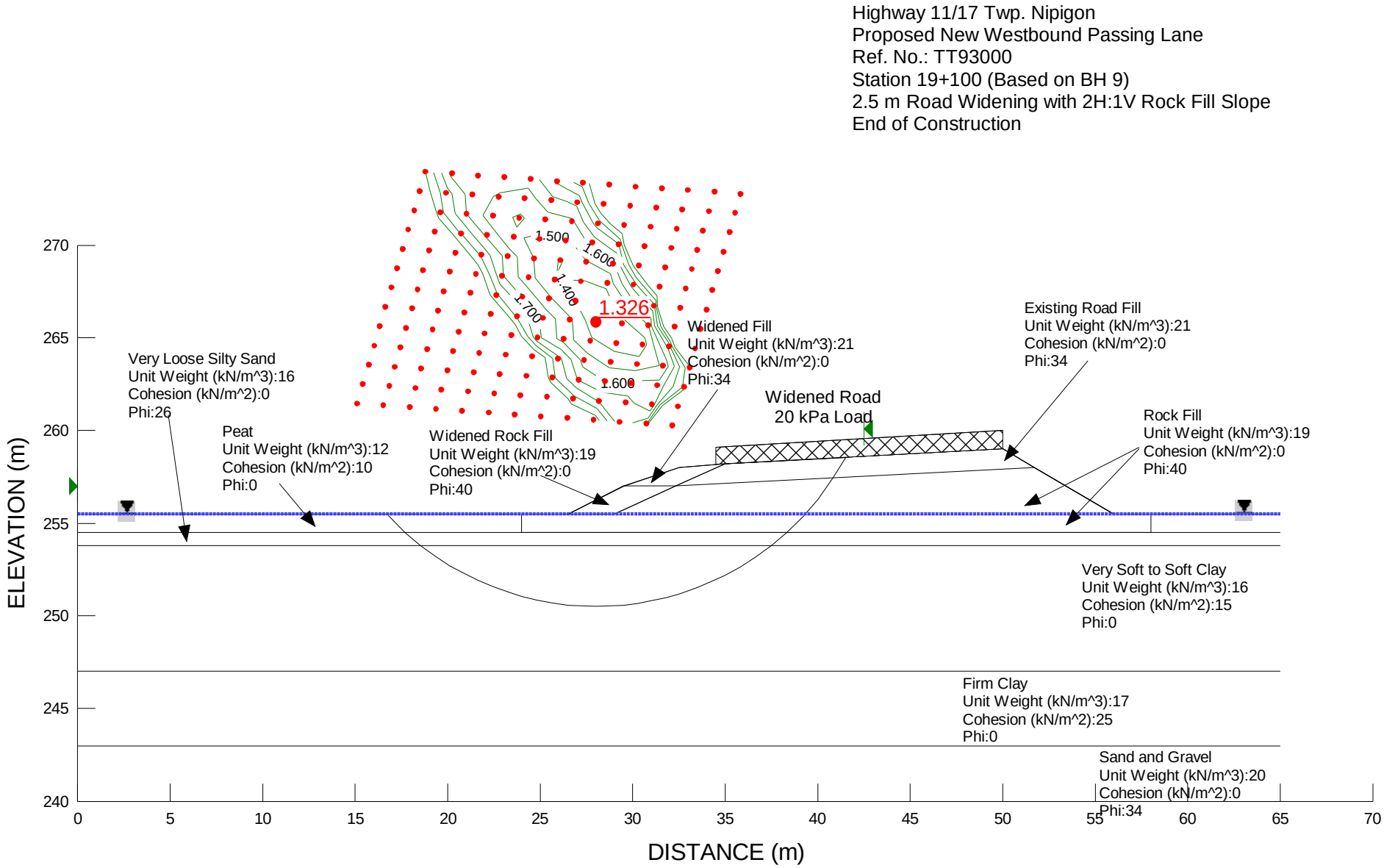


Figure X4.4 Slope Stability Analysis of Section at Station 19+100 with 2.5 m Road Widening (End of Construction - Slip Surface within Westbound Lane)

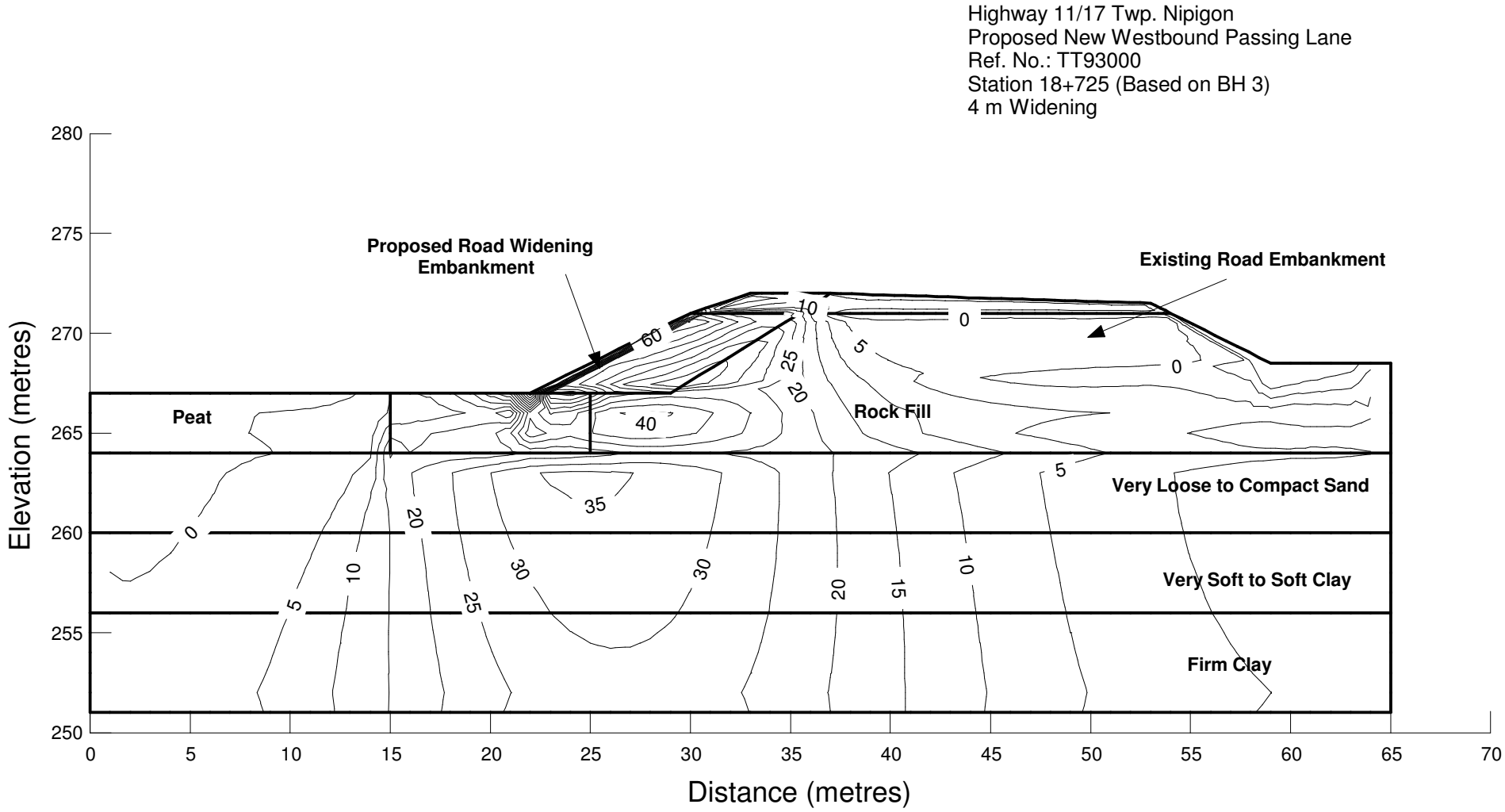


Figure X5.1 Vertical Stress Increment Contour (kPa) - 4 m Road Widening at Station 18+725

Highway 11/17 Twp. Nipigon
Proposed New Westbound Passing Lane
Ref. No.: TT93000
Station 18+875 (Based on BH 4)
3.5 m Widening

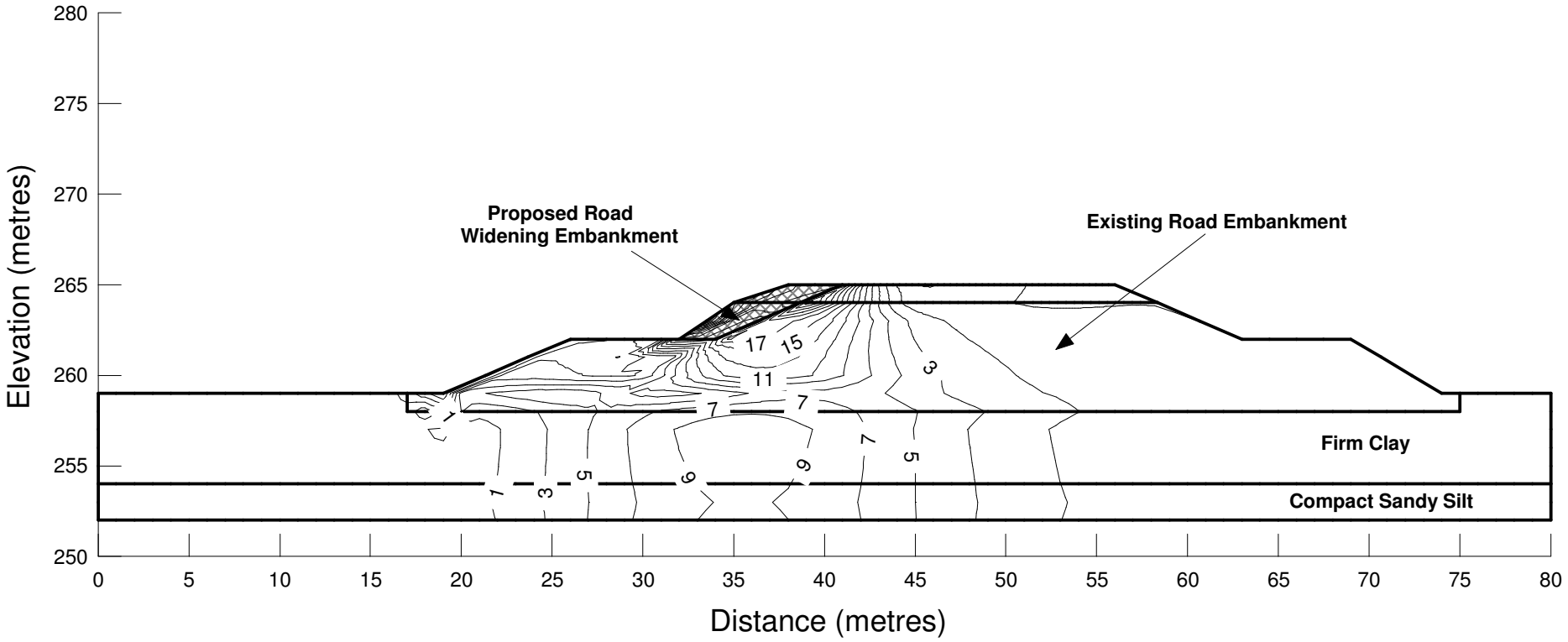


Figure X5.2 Vertical Stress Increment Contour (kPa) - 3.5 m Road Widening at Station 18+875

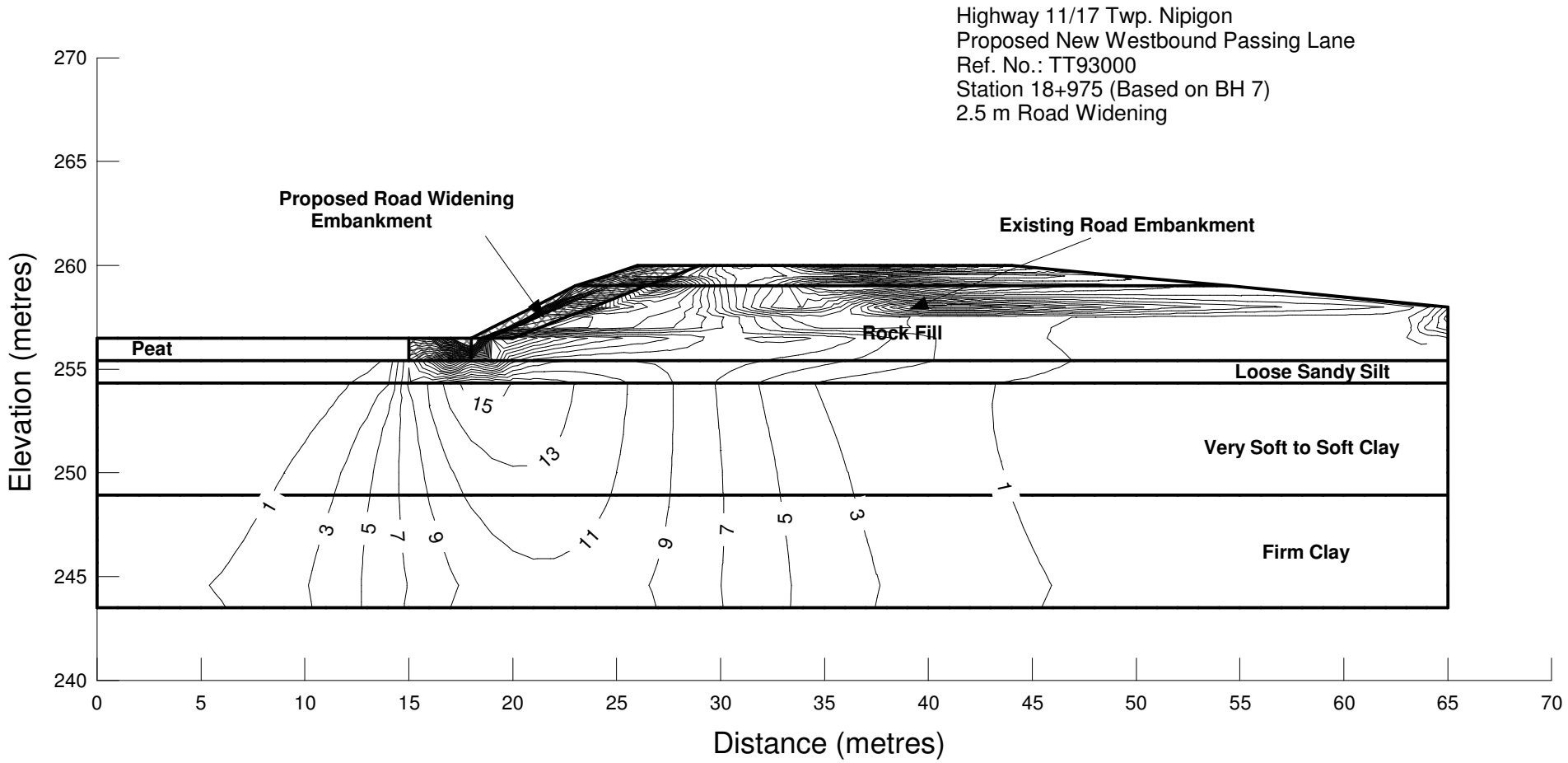


Figure X5.3 Vertical Stress Increment Contour (kPa) - 2.5 m Road Widening at Station 18+975

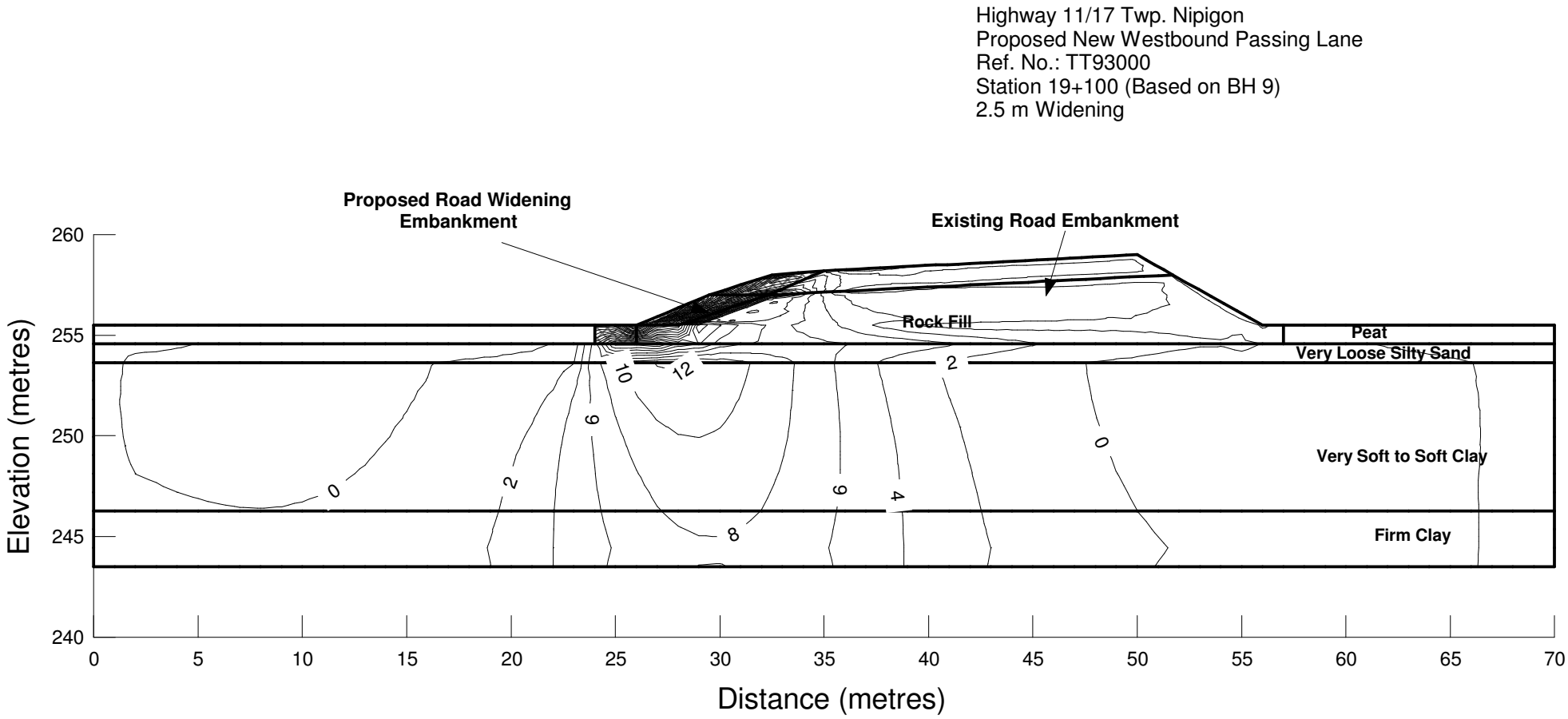


Figure X5.4 Vertical Stress Increment Contour (kPa) - 2.5 m Road Widening at Station 19+100

Soil - clay NCC

C_c	C_s	e_o	e_f	c_v (m^2/day)	time (months)	c_α	t_{90} (months)
0.29	0.04	1.01	0.76	0.02	132	0.011	113

Loc	H m	p_0 kPa	Δp kPa	p_f kPa	Total Consol. mm
C	9	112	30	142	134

Time (months)	H (m)	p_0 kPa	Δp kPa	p_f kPa	Tv	%U	settlement (mm)	
0	9	112	30	142	0	0	0	
3	9	112	30	142	0.023	17	23	
6	9	112	30	142	0.045	24	32	
9	9	112	30	142	0.068	29	39	
12	9	112	30	142	0.090	34	**45**	
24	9	112	30	142	0.180	48	64	
36	9	112	30	142	0.270	58	78	
48	9	112	30	142	0.360	66	89	
60	9	112	30	142	0.451	73	98	
72	9	112	30	142	0.541	79	105	
84	9	112	30	142	0.631	83	111	
120	9	112	30	142	0.901	91	122	
132	9	112	30	142	0.991	93	**124**	**79**

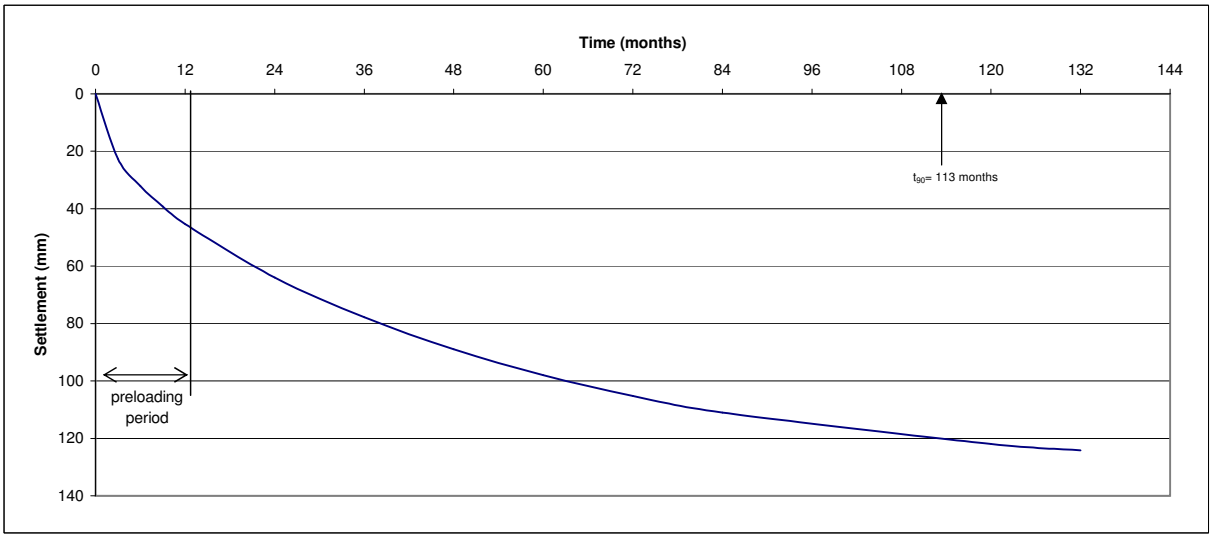


Figure X6.1 Calculation of Consolidation Settlement at Station 18+725

Soil - silt NCC

C_c	C_s	e_o	e_f	c_v (m^2/day)	time (months)	c_α	t_{90} (months)
0.18	0.02	0.77	0.55	0.02	132	0.004	22

Loc	H m	p_0 kPa	Δp kPa	p_f kPa	Total Consol. mm
C	4	115	9	124	13

Time (months)	H (m)	p_0 kPa	Δp kPa	p_f kPa	Tv	%U	settlement (mm)	
0	4	115	9	124	0	0	0	
3	4	115	9	124	0.114	38	5	
6	4	115	9	124	0.228	54	7	
9	4	115	9	124	0.342	65	9	
12	4	115	9	124	0.456	74	**10**	
24	4	115	9	124	0.913	91	12	
36	4	115	9	124	1.369	97	13	
48	4	115	9	124	1.825	98	13	
60	4	115	9	124	2.281	99	13	
72	4	115	9	124	2.738	99	13	
84	4	115	9	124	3.194	99	13	
120	4	115	9	124	4.563	100	13	
132	4	115	9	124	5.019	100	**13**	**3**

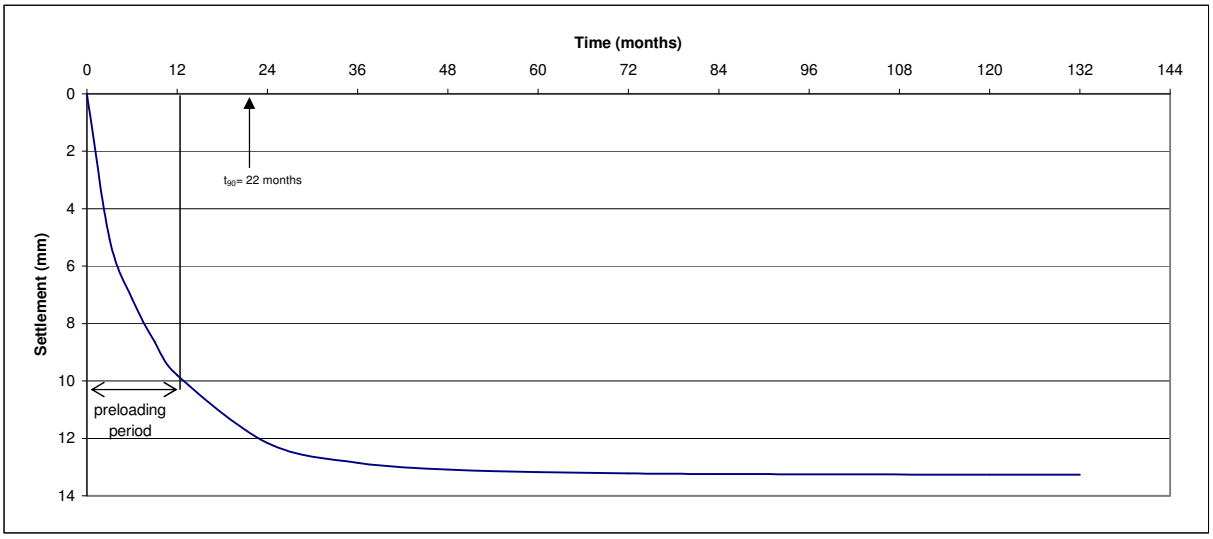


Figure X6.2 Calculation of Consolidation Settlement at Station 18+875

Soil - clay NCC

C_c	C_s	e_o	e_f	c_v (m^2/day)	time (months)	c_α	t_{90} (months)
0.29	0.04	1.01	0.76	0.02	132	0.011	139

Loc	H m	p_0 kPa	Δp kPa	p_f kPa	Total Consol. mm
C	10	90	10	100	66

Time (months)	H (m)	p_0 kPa	Δp kPa	p_f kPa	Tv	%U	settlement (mm)	
0	10	90	10	100	0	0	0	
3	10	90	10	100	0.018	15	10	
6	10	90	10	100	0.037	22	14	
9	10	90	10	100	0.055	26	17	
12	10	90	10	100	0.073	30	**20**	
24	10	90	10	100	0.146	43	28	
36	10	90	10	100	0.219	53	35	
48	10	90	10	100	0.292	60	40	
60	10	90	10	100	0.365	67	44	
72	10	90	10	100	0.438	72	48	
84	10	90	10	100	0.511	77	51	
120	10	90	10	100	0.730	87	57	
132	10	90	10	100	0.803	89	**59**	**39**

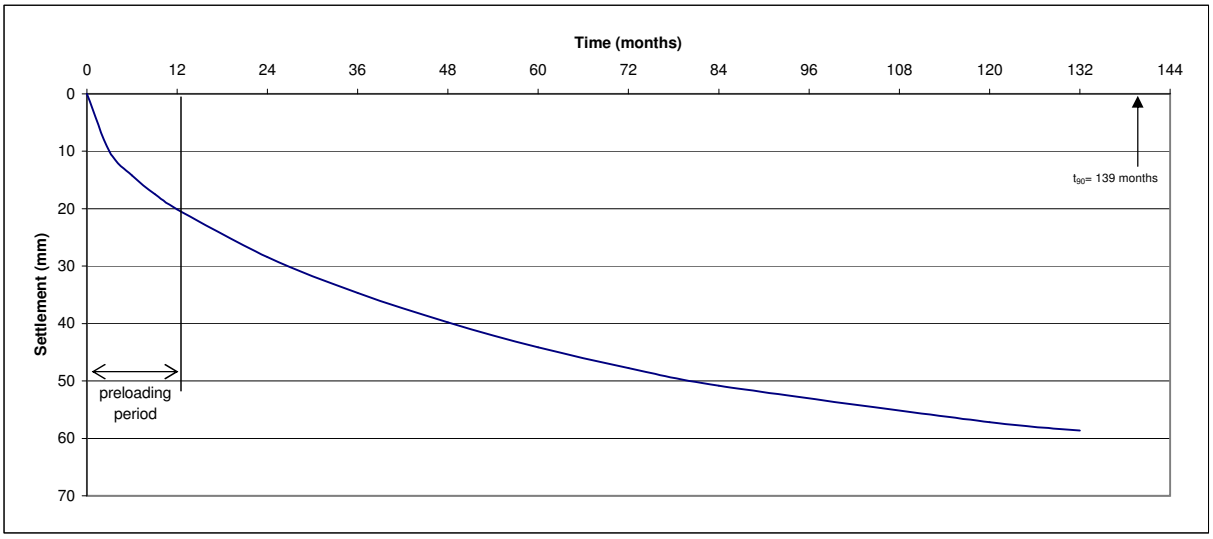


Figure X6.3 Calculation of Consolidation Settlement at Station 18+975

Soil - clay NCC

C_c	C_s	e_o	e_f	c_v (m^2/day)	time (months)	c_α	t_{90} (months)
0.29	0.04	1.01	0.76	0.02	132	0.011	139

Loc	H m	p_0 kPa	Δp kPa	p_f kPa	Total Consol. mm
C	10	68	8	76	70

Time (months)	H (m)	p_0 kPa	Δp kPa	p_f kPa	Tv	%U	settlement (mm)	
0	10	68	8	76	0	0	0	
3	10	68	8	76	0.018	15	11	
6	10	68	8	76	0.037	22	15	
9	10	68	8	76	0.055	26	18	
12	10	68	8	76	0.073	30	**21**	
24	10	68	8	76	0.146	43	30	
36	10	68	8	76	0.219	53	37	
48	10	68	8	76	0.292	60	42	
60	10	68	8	76	0.365	67	47	
72	10	68	8	76	0.438	72	50	
84	10	68	8	76	0.511	77	54	
120	10	68	8	76	0.730	87	60	
132	10	68	8	76	0.803	89	**62**	**41**

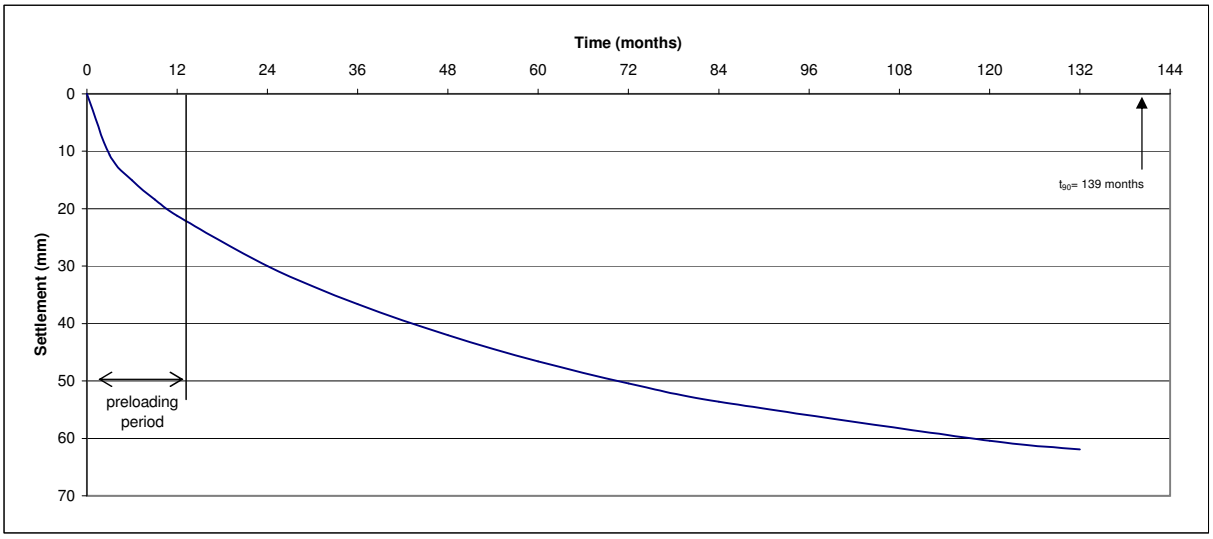


Figure X6.4 Calculation of Consolidation Settlement at Station 19+100