Geotechnical Investigation and Design Report
Slope Stabilization at Station 13+500
Highway 102, Kaministiquia

GWP 6019-05-00
WP 6082-05-01

GEOCRES No. 52A-137

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June 26, 2009 Ref. No. 08-011

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Part A - **FOUNDATION INVESTIGATION REPORT**

1 Introduction

The Ministry of Transportation of Ontario (MTO) Northwestern Region retained TBT Engineering (TBTE) to provide foundation investigation services for two slope instability sites, located along Highway 102 approximately 33 km west of Thunder Bay. The slope sites are at approximate Stations 13+300 and 13+500, Township of Dawson Lots. This investigation is being carried out under Assignment No. 6007-E-0021, GWP 6019-05-00, WP 6082-05-01.

The original scope for this assignment involved field investigations, laboratory testing, installation and monitoring of geotechnical instrumentation, and provision of recommendations for remedial measures to mitigate slope instability. However, during this assignment the west slope instability at Station 13+300 progressed beyond acceptable safety limits and the MTO requested that TBTE complete the detailed design of remedial stabilization measures under a change of scope to this assignment. The west slope was located between Stations 13+274 and 13+312 with movements occurring along the east bound lane. The detailed design of remedial stabilization measures for the west slope at Station 13+300 has been reported under a separate cover by TBTE entitled "Geotechnical Investigation and Design Report, Slope Failure at Highway 102, Station 13+300, Kaministiquia (GWP 6019-05-00, WP 6081-05-01, GEOCRES No. 52A -133) February 9, 2009".

At the time of this investigation, the east slope movement was located between Stations 13+450 and 13+500 with slope movements occurring along the east bound lane.

This report (Part A) addresses the foundation investigation portion of this assignment, including the field investigations, laboratory testing, and installation and monitoring of geotechnical instrumentation. A subsequent section (Part B) discusses the analyses of various stabilization options and provides recommendations for remedial measures to mitigate the slope movements.

The MTO foundation section has assigned GEOCRES No. 52A-137 to this site.

2 Site Description

The site is located along Highway 102, approximately 33 km west of Thunder Bay and approximately 3 km east of the junction with Highway 11/17 at Sistonen's Corner. At this location, Highway 102 is aligned approximately east-west along the north slope of an easterly-draining tributary valley of the Kaministiquia River. The valley deepens to east, and its floor is greater than 15 m below the highway elevation at this site. The highway grade gradually descends the tributary valley slope to the east, into the main valley of the Kaministiquia River.

At the time of this investigation, the scarp of the slope movement area was evident as arc-shaped pavement cracking, between station 13+450 and station 13+500, extending across the east bound lane to the centre line. No toe bulge was observed down slope (likely obscured due to the presence of trees and shrubs).

Highway 102 near station 13+500 May 30, 2008 Cracking and Asphalt Patching - Looking West

Highway 102 near station 13+500 May 30, 2008 Right Side Slope - Looking West

A 34 m long 760 mm dia. centre line culvert also crosses the highway at this location with the inlet located along the north side ditch at station 13+480 and the outlet located at the south side embankment toe at station 13+495. The culvert outlet discharges into a ditch along the natural valley slope which directs the flow into the water course along the valley floor. There is evidence of ongoing erosion associated with this ditch.

2.1 Site History

Slope movements have been an ongoing issue at this site for many years. The asphalt surface has been repeatedly patched and embankment repairs undertaken on at least one occasion.

In 1996, a Foundation Investigation was carried out by MTO and is reported in “Engineering Materials Office, Foundation Design Section, WP 849-97-00, Dist 61, Hwy 102, Slope Stability Investigation at Hwy 102/Sistonen’s Corners from Sta. 13+450 to St. 13+500, GEOCRES 52A-

121, April 1997". At the time of the 1996 investigation, it was reported that "observations of the slope movement zone showed the slip surface did not extend along the whole slope but was located approximately 6 – 7 m down the crest, with tension cracks along the paved portion of the highway.". Three boreholes were advanced during this investigation which have been utilized for this report and identified as Boreholes 1-1996, 2-1996 and 3-1996.

In 1999, remedial measures were implemented at the site under Contract 99-208. It is understood that this work included an excavation starting at approximately 2.3 m north of centerline, cut at a 1.5:1V slope to an excavation depth of 3.5 m and backfill with granular fill. A subdrain was constructed in the north ditch draining the full length of the north back slope, extending between Stations 13+225 and 13+480.

In 2005, a slope inclinometer was installed at Station 13+478, 17 m Rt. This was installed to measure slope movements. The slope inclinometer was installed with a logged borehole which has been utilized for this investigation and has been identified as Borehole 1-2005. Slope inclinometer readings taken over a period from February 2005 to September 2005 indicated slope movements of up to 30 mm to a depth of 3 m below grade.

During the months of August and September of 2008, the adjacent slope failure at Station 13+300 was repaired.

On October 1, 2008, a 180 mm vertical drop was observed at this site along the right side shoulder rounding. A heaving rain fall event occurred several days prior to this movement. Environment Canada reported that 68.8 mm of rainfall occurred on September 26, 2008 as recorded at the Thunder Bay Burwood weather station located approximately 28 km southeast of the subject site. The short term remedial action plan developed at that time involved removal of excess granular material from the south crest (followed by sealing) to lessen the total mass on the slope movement area. In addition, the north side ditch was to be cleaned out above the subdrain to ensure positive drainage and a shallow layer of Granular "A" (50-100 mm) was to be placed with granular sealing to reduce potential surface water infiltration.

Highway 102 near station 13+500 October 1, 2008, a 180 mm vertical drop at the Rt. shoulder

3 Investigation Procedures

Field investigations under this assignment took place between May 28 and 30, 2008. TBTE drilled two boreholes in the area of slope movements (Boreholes 106 and 107). Additional borehole data at this general location consisted of three boreholes carried out in 1996 (identified as Boreholes 1-1996, 2-1996 and 3-1996) and one borehole advanced in 2005 (identified as Borehole 1-2005). The borehole locations are shown on Drawing 1 of Appendix C, and are summarized in Table 1.

Boreholes 106 and 107 were drilled to depths between 5.3 m and 18.7 m below existing surface grade, using a truck-mounted CME 55 drill rig. During the drilling operations, soil samples were obtained from the auger flights and using the techniques of the standard penetration test (SPT). This involves driving a 51 mm diameter thick-walled sampler into the soil under the energy of a 63.5 kg weight falling through 760 mm. The number of blows required to drive the sampler 0.3

m is known as the standard penetration blow count (N). Following completion of the test, representative soil samples were obtained from within the sampler. Where fine-grained soils were encountered during drilling, in-situ undrained shear strength was measured using a MTO style field vane and undisturbed samples were retrieved using a thin-walled 'Shelby' sampling tube. Borehole logs are included in Appendix A.

The boreholes were decommissioned by backfilling with a bentonite and/or grout mixture.

All borehole elevations were surveyed in the field, relative to a local benchmark provided by the MTO.

Soil samples were transported to TBT Engineering's laboratory in Thunder Bay for testing and analysis. Testing on selected samples included moisture content, grain size analysis, Atterberg limits, and a direct shear test. Results are outlined on the borehole logs (Appendix A) and detailed on laboratory data reports (Appendix B).

Table 1 – Borehole Locations

BH No.	BH Surface Elevation	BH Bottom Elevation	Station	Offset	Comments
1-2005	344.2	329.2	13+478.0	17.0 Rt	Slope inclinometer casing installed
3-1996	348.3	337.2	13+486.0	2.5 Rt	Near Rt shoulder
2-1996	348.7	336.1	13+476.5	2.1 Rt	East bound lane
1-1996	349.7	340.1	13+459.5	1.1 Rt	East bound lane
106	348.6	343.5	13+474.5	6.2 Lt	Advanced through embankment fill near Lt shoulder.
107	347.7	329.0	13+475.0	11.9 Lt	Lt side ditch, advanced to a depth at least 3 m into dense to very dense material.

4 Subsurface Conditions

4.1 Overview

Interpretation and descriptions of the subsurface conditions are based on the results of field investigations and laboratory testing. Details of the subsurface conditions are provided on borehole logs (Appendix A), and shown on the Borehole Location Plan and Soil Strata (Drawing 1, Appendix C).

The general subsurface stratigraphy consists of asphalt pavement overlying a granular fill embankment with a clay subgrade. Beyond the embankment fill, a surficial layer of top soil overlies the clay soils. The clay stratum overlies a silt deposit which was identified to the depths investigated in all boreholes except Borehole 107. At Borehole 107 (located within the north side ditch, up slope of the slope movement scarp), the silt stratum is underlain by silty sand.

4.2 Asphalt

Hot mix asphalt was encountered at the surface of Boreholes 1-1996, 2-1996, 3-1996 and 106. An asphalt thickness of 80 mm was measured at Borehole 106 which is located outside of (north of) the slope movement zone.

4.3 Embankment Fill

Embankment fill was encountered at all of the boreholes except BH 1-2005 which is located 17 m Rt. (Drawing 1, Appendix C). Fill material is variable and consists of sand, gravel and silt in variable proportions. Occasional cobbles and boulders were also noted. In general, the fill was found to be thicker (e.g. 3.2 m at BH's 3-1996, and 2-1996) on the south shoulder of the embankment than the north shoulder (e.g. 1.2 m thick at BH 106).

4.4 Clay

The native subgrade consists primarily of silty clay. The surface of the clay subgrade below the embankment slopes in a downward direction from Lt. to Rt. (north to south). Near the Lt. side of the embankment at Borehole 107 (13+475.0, 11.9 m Lt., Drawing 1, Appendix C) the clay surface is at an elevation 347.0 m. Near the right side toe of the embankment the clay is at about elevation 344.2 m as indicated at Borehole 1-2005 (13+478.0, 17 m Rt., Drawing 1, Appendix C). The base of the clay stratum is relatively consistent across the embankment at an elevation of about 340 m, approximately 8 m below centerline grade.

The clay is very stiff, with field vane and unconfined compression test shear strengths in excess of 100 kPa. One quick triaxial test carried out in 1996, indicates an undrained shear strength of 75 kPa. Atterberg limit testing carried out on selected samples indicates that the clay is of high plasticity with the natural moisture content was between the liquid and plastic limits.

As typical for stiff clays, the clays are considered to have strain softening characteristics. This was confirmed with laboratory testing at the adjacent 13+300 slope failure site.

4.5 Silt

A silt stratum underlies the above noted clay. This stratum extends to an elevation of 334.2 m (depth of 13.5 m) at Borehole 107 (13+475.0, 11.9 m Lt.) and to the extents (elevation 329.2 m, depth 15.0 m) of Borehole 1-2005 (13+478.0, 17 m Rt.). The base of this stratum appears to slope in a downward direction from Lt. to Rt. (north to south).

Grain size analyses show a variable composition in this unit, ranging from sandy silt to silt with some sand. The SPT “N” values vary from 16 to over 100 blows / 0.3 m (with most lying between 20 and 40 blows/ 0.3 m) indicating a compact to very dense condition.

A drained direct shear test was carried out on a sample of silt from Borehole 107 (13+475.0, 11.9 m Lt.) at a depth of 9.1 m. The test results (Appendix B) indicate a measured effective angle of internal friction of 35° with an effective cohesion intercept of 0 kPa.

4.6 Silty Sand

A silty sand stratum was identified below the silt at Borehole 107 (13+475.0, 11.9 m Lt.) from elevation 334.2 m (13.5 m depth) extending to the bottom of the borehole at elevation 329.0 (18.7 m depth). Grain size analyses show a variable composition in this unit, ranging from silty sand to sand and silt. Penetration resistance is very high in this unit (i.e. “N” greater than 40 blows per 0.3 m), indicating a dense to very dense condition.

4.7 Groundwater

Based on observations during drilling and trends with moisture content testing, the design groundwater level has been taken near the clay surface as indicated in Table 2.

Table 2 – Clay Surface and Design Groundwater Level (GWL) at Borehole Locations

BH No.	BH Surface Elevation	Clay Surface / GWL Elevation (m)	Depth (m) of Clay Surface / GWL
1-2005	344.2	344.2	0.0
3-1996	348.3	345.1	3.2
2-1996	348.7	345.5	3.2
1-1996	349.7	347.3	2.4
106	348.6	347.4	1.2
107	347.7	347.0	0.7

5 Instrumentation and Monitoring

5.1 General

Monitoring of the slope movement zone at station 13+500 was to consist of taking of two sets of readings at the single slope inclinometer installed in 2005. However, as additional instrumentation (survey control points and crack monitors) were installed to monitor ongoing and excessive movements at Station 13+300, some additional survey control points and crack monitors were also installed at Station 13+500 and monitored until June 9, 2008. The instrumentation utilized at Station 13+500 consists of the following:

- A 70 mm diameter inclinometer casing installed to a depth of 15.0 m below surface grade (329.2 m) at BH 1-2005 (Station 13+478, 17 m Rt.), near the south toe of the embankment fill.
- Survey control points were established along two cross-sections within the slope movement area, to facilitate on-going displacement measurements of the embankment and slope surface. These were established on May 28, 2008. Further control points were also established at selected guiderail locations to identify the extent of any vertical displacements along the highway shoulder.
- Crack monitors installed along existing cracks on the asphalt surface to allow for rapid visual indications of total movement of the slope movement area. Crack monitors consisted of steel concrete nails located across the crack to provided reference points for measurements of movements.
- Visual inspections to identify any significant changes at the site (i.e. additional deformations, cracking, seepage or sloughing) which could indicate further instability and/or the need to further restrict traffic.

The layout of the various monitoring sites is illustrated Appendix F.

5.2 Results of Monitoring

5.2.1 Visual Inspection

Visual inspections were carried out between May 30, 2008 and June 9, 2008, to monitor conditions at the site and identify any significant deterioration in the stability of the slope that could pose an unacceptable risk to the travelled portion of the highway. During that period, existing cracks of the pavement surface remained constant, both in length and width, in an arc-shaped pattern extending between the eastbound lane shoulder and the centerline. No patching work was done during this period.

On October 1, 2008, a 180 mm vertical drop was observed at this site along the right side shoulder rounding. A heaving rain fall event occurred several days prior to this movement. Environment Canada reported that 68.8 mm of rainfall occurred on September 26, 2008 as recorded at the Thunder Bay Burwood weather station located approximately 28 km southeast of the subject site. In addition several new cracks in the pavement were also noted. The short term remedial action plan developed at that time involved removal of excess granular material from the south shoulder (followed by granular sealing) to lessen the total mass on the slope movement area. In addition, the north side ditch was to be cleaned out above the subdrain to ensure positive drainage and a shallow layer of granular "A" (50-100 mm) was to be placed with a granular sealed to reduce potential surface water infiltration.

Copies of detailed TBTE site inspection reports are provided within Appendix F.

5.2.2 Slope Inclinometer

Initial slope inclinometer readings were taken between February 10, 2005 (initial reading) and September 22, 2005. Movements of up to 30 mm in the down slope direction were recorded over a depth of 3 m below ground surface. Subsequent readings were attempted March 26, 2008, at which time; the slope inclinometer casing developed a sharp bend at approximately 3 m below grade due to excessive soil movement. As a result, the slope inclinometer probe was unable to advance further and no subsequent readings were possible. A graphical summary of slope indicator monitoring (year 2005 data only) is provided in Appendix F.

5.2.3 Survey Control Points

Surveys of control points along the two slope cross sections indicated vertical movements were negligible (less than 5 mm) from May 28, 2008 to June 6, 2008.

Additional survey points established on the guide rails adjacent to the slope movement area indicated vertical movements (downward) along the guide rails of less than 5 mm.

A summary of the survey control monitoring data is provided in Appendix F.

5.2.4 Crack Monitors

During the interval from May 28 to June 9, 2008 negligible movements (maximum 2 mm horizontally) were measured. The crack monitors were destroyed (and not reinstated) during hot mix patching work carried out in October of 2008.

6 Miscellaneous

The field drilling services for this project were provided by TBT Engineering. Laboratory testing was carried out at the TBT Engineering laboratory in Thunder Bay. The drilling operations were supervised by H. Finke. This report was prepared by G. Maki, P.Eng. and W. Hurley, P.Eng.

Part B - **FOUNDATION DESIGN RECOMMENDATIONS**

7 Discussions and Slope Remediation Recommendations

7.1 Introduction

The Ministry of Transportation of Ontario (MTO) retained TBT Engineering (TBTE) to carry out a subsurface investigation, laboratory testing program, and monitoring program, and to provide analyses and recommendations for the remediation of a slope movement at Station 13+500 along Highway 102, approximately 3 km east of its junction with Highway 11/17.

Part A of this report described foundation investigations, laboratory testing, subsurface conditions and monitoring carried out at the subject site. The purpose of this section of the report (Part B) is to provide a discussion of remediation alternatives considered, results of stability analyses, options selection and recommendations for construction of the preferred option.

7.2 Slope Movement Remediation Considerations

The slope movement at Station 13+500 continues to cause settlements and cracking in the east bound lane over a 50 m long section of the highway. As evidenced by the movements observed in October of 2008 (180 mm drop at right side shoulder), it appears that the slope movements are escalating. As the slope continues to move, the shear strength of the clay foundation soils weaken due to the strain softening properties of the clay. In addition, the formation of tension cracking may have occurred which can lead to increased hydrostatic pressures and a reduction in strength which could promote further movement. With each slope movement, the degree and extent of settlement will likely increase. Remediation of the slope movement is recommended and should be carried out as soon as practical. Until remediation measures can be implemented, it is understood that MTO is visually inspecting the highway embankment for further movements and is prepared to close the east bound lane, if necessary.

The main cause of embankment movement at its current state is due to a loss of strength within the clay foundation soils. The loss of strength can be attributed to a low level of stability and the strain softening characteristics of the clay foundation soils. Prior to embankment movements, the embankment had a relatively low level of stability which was likely further reduced due to fluctuating and high groundwater conditions within the embankment. Groundwater levels within the embankment may have been influenced by groundwater infiltration from via the north side ditch. A subdrain system consisting of a 200 mm perforated pipe- with granular backfill was constructed approximately 0.5 to 1 m below the ditch invert and graded for positive drainage. It is understood that the granular fill used for the subdrain is directly connected to the granular fill of the embankment. In addition, no low permeable material was provided above the subdrain system along the ditch invert. It is understood that at times, the east side outlet of the subdrain becomes restricted and/or blocked which could lead to increased groundwater levels and seepage across the embankment. Once the level of stability is lowered sufficiently, overstresses zones within the clay foundation can lead to creep movements and strain softening of the clay soils. Strain softening of zones with the clay foundation further reduces the level of stability which leads to additional strain softening and further reduction in stability. This is evident by the increased level of movements over time. This cycle of propagating strain softening leads to failure of the embankment slope. Back analysis of the slope considering post peak effective strength parameters within the clay foundation and a high groundwater level confirms instability.

To arrest the slope movements, an improvement in stability is required. This may be achieved through realignment of the highway to outside of weakened soils, improvements in drainage and/or control of groundwater infiltration and seepage, replacement of weakened soils with higher strength soils, slope regrading, and/or the use of lightweight fills.

Considering these treatments, four remediation alternatives have been identified as being potentially feasible and practical and were considered for further development and analyses as follows:

- <u>Option 1 - Highway Realignment:</u>
  This option involves realignment of the highway towards the north (towards stable ground). In addition, the existing embankment fills would be cut (flattened) and graded with granular fill to improve stability.

- <u>Option 2 - Partial Excavation of Weakened Material and Replacement with Granular Fill:</u>
  In order to maintain at least 2 of the 3 existing lanes during construction, excavation of the weakened foundation soils would be started at a the existing right side (south side) edge of pavement (Option 2A and 2B), or would start at about the existing centre line (Options 2C and 2D). Option 2A involves excavation and fill replacement only. Option

2B includes a 2.4 m deep subdrain system located along the north side ditch to lower the groundwater level. Option 3A includes light weight fill and Option 4 also considers the inclusion of piles as slope reinforcement. With this approach, a significant portion of the weakened foundation soils will be left in place. Under all sub-options, a seepage cut-off blanket has been incorporated into the north side ditch to limit/reduce seepage into the existing sub-drain and granular embankment fills.

- <u>Option 3 – More Extensive Removal of Weakened Clay Foundation Soils:</u>
  This option involves a deep cut through the existing movement zone to remove a significant portion of the weakened/remolded clay which would then be replaced with higher strength and free draining compacted granular fill. The granular fill placed would also act to improve drainage conditions and act as a toe buttress. The excavation would start at a distance of 3.5 m to the left of the centre line leaving only one existing lane. As such, a temporary detour will be required. The excavation would be carried out at a slope of 1.5H:1V through the existing granular fills. Once the clay has been encountered, the excavation would be carried out at slopes of 2H:1V (Option 3A), 2.5H:1V (Option 3B), and 3.0H:1V (Option 3C). Under Option 3A, it is expected that all of the weakened clay would be removed. Under Option 3B and 3C, an increasing proportion of weakened clay would be left within the foundation. Option 3C also utilizes the inclusion of a 2.4 m deep subdrain installed within the north side ditch to improve stability by lowering the groundwater level within the foundation. Under all sub-options, a seepage cut-off is to be incorporated into the north side ditch to limit/reduce seepage into the granular embankment fills.

The use of sub-drains and/or piles alone or soil reinforcement systems were considered, but were determined to be either of limited long term reliability, not cost effective and/or provided insufficient improvement in stability and were not developed further.

7.3 Slope Stability Analyses - General

Stability analyses were carried out to assess the various remediation alternatives being considered. In addition, a back analyses of the existing conditions was completed to assess the accuracy of the subsurface model. Stability analyses were carried out using Slope/W software and limit equilibrium analyses using the Morgenstern-Price method.

Various soils properties were used during the analyses to model different loading conditions and stages for the proposed construction. Soil properties used for the analyses are provided in Table 2. Temporary excavations were modelled using undrained shear strength parameters within clay soils. The clay soils were modelled using both undrained shear strength parameters and effective strength parameters for fills and permanent slopes.

Table 2 Geotechnical Analyses - Soil Properties

Soil	Effective Strength Parameters		Undrained Shear Strength, Cu (kPa)	Unit Weight γ (kN/m^3)
	Effective Angle of Internal Friction ,φ' (degrees)	Effective Cohesion Intercept, C' (kPa)		
Granular Fill	35	0	N/A	21
Clay Within Movement Zone	21(for back analyses) 14 (for assessment of remediation options)	0	25	18
Clay Outside of Movement Zone	21	0	75	18
Silts and/or Sands	35	0	N/A	20
Lightweight Fill	N/A	0	15	1

Traffic loading was modeled with a distributed live load of 20 kPa.

Where lightweight fill was used, the use of expanded polystyrene was anticipated.

The minimum design factor of safety for all construction stages was selected as 1.3. The results of the various stability analyses have been included in Appendix D.

7.4 Results of Stability Analyses and Remediation Option Selection

Slope stability analyses were used to assess for the remediation options as defined in Section 7.2. In addition, a back analysis of the existing conditions was carried out to validate the stability model being used. The various options were reviewed and a preferred option selected based on the results of the stability analyses, economic considerations and physical layout of the project area.

The results of the analyses have been presented in Appendix D and have been summarized in Table 3.

Table 3: Results of Stability Analyses of Slope Movement Remediation Options

Case / Alternative	Calculated Factor of Safety (FoS)			Disadvantages / Requirements	Advantages	Comments
	Temporary Excavation	Final Configuration, Undrained Shear Strengths for Clays	Final Configuration, Effective Shear Strength Parameters			
Back Analyses (existing conditions)	N/A	1.19	1.00			The calculated FoS is close to unity which indicates a reliable stability model for an actively moving slope.
Option 1 Highway Realignment	N/A	1.69	1.30	A 15.5 m realignment to the north is required to obtain a suitable level of stability. In addition, a 1.4 m thick granular blanket is required over the south side slope. The alignment would occur over 1 km and involve major grading quantities, property acquisition, utility relocation and would significantly impact the adjacent structure. An assessment of structural options is required to accommodate this realignment. Construction will extend beyond the south side property line.	Could be constructed without detours required and minimal disruption to traffic.	This option is not considered feasible due to the impacts on the adjacent structure. Preliminary costs to complete the grading work alone has been estimated at $2.0 million..
Option 2 **2A)** Partial excavation of weakened soils leaving at least 2 lanes in-place. **2B)** with 2.4 m deep subdrain **2C)** includes subdrain and LWF **2D)** subdrain, LWF and piles	2A) 1.37	2A) 1.16	2A) 0.82 2B) 0.89 2C) 0.92 2D) 1.15	To partially remove the weakened material, a 2H:1V cut starting at the right side edge of pavement was considered. A deep (8.5 m) excavation is required to remove the weakened material beyond the toe of the embankment. To ensure stability during excavation, the excavation must be completed in progressive stages with a maximum excavation length of 3 m parallel to the highway. The inclusion of a 2.4 m deep subdrain, lightweight fill and piles (6 rows of piles at 1 m spacing) was also considered (additional excavation within the right side lane would be required). Construction will extend beyond the south side property line. Insufficient stability is realized due to a significant portion of weakened material remaining in the foundation.	No detour required. No alignment revision is required. Additional investigation would be required to assess pile installation.	Insufficient level of stability is achieved. Leaving a significant portion of the weakened clay (within the within the zone of slope movement) limits the level of stability even when considering a 2.4 m deep subdrain, lightweight fill and a significant amount of piles.
Option 3 Extensive removal of weakened foundation. **3A)** 2:1 cut in clay (full removal of weakened clay) **3B)** 2.5:1 cut in clay **3C)** 3:1 cut in clay and subdrain	3A) 1.35 3B) 1.35 3C) 1.35	3A) 1.59 3B) 1.45 3C) 1.31	3A) 1.49 3B) 1.43 3C) 1.30	As these options consider the excavation starting 3.5 m left of centre line, only one lane will remain. As such a temporary detour will be required. Option 3A provides the greatest level of stability as all of the weakened foundation soils are removed, but requires the most extensive excavation. Option 3B provides a reduction in excavation quantity while leaving a zone of weakened foundation soils in-place. Option 3C, further reduces the excavation quantities, but requires and relies on a costly subdrain constructed within the north side ditch. A seepage cut off system along the north side ditch has been incorporated into all of these options. Option 3C relies on the long term performance of the 2.4 m deep subdrain.	For single lane traffic during construction, no additional property or significant cut is required to the north. All sub-options provide an adequate level of stability.	Estimated costs for these options are as follows (these do not include property acquisition to the south): 3A – $1.21 million 3B – 1.17 million 3C – 1.51 million Given the above costs, Option 3B is recommended.

Option 3B is recommended. This involves removing a significant portion of the weakened foundation soils and replacement with granular fill. This option provides a sufficient level of stability and is considered to be the most cost effective.

7.5 Construction Recommendations

The recommended remediation concept will involve removal of the existing weakened soils within the movement zone and reconstruction of the embankment with higher strength compacted granular fill. Phased construction will be required to implement the proposed slope stabilization measures. The construction will require 4 phases as follows:

- Phase 1 – Detour
- Phase 2 – Excavation
- Phase 3 – Reconstruction of Embankment and Pavement Structure
- Phase 4 – Removal of Detour and Installation of North Ditch Seepage Barrier

Conceptual drawings illustrating the proposed slope remediation has been provided in Appendix E.

7.5.1 Phase 1 – Detour

Stage 1 will entail the construction of a single lane detour along the north side ditch. The detour will be accomplished by widening of the existing embankment over the existing north side ditch. Granular thicknesses up to 1.5 m thick will be required. The north side fore slope is to be constructed at a slope of 2H:1V. It is anticipated that the detour will need to be approximately 150 m long including tapers. In order to facilitate drainage around the detour, a temporary culvert may be considered along the existing north side ditch and/or ditching along the north side of the detour may be incorporated into the design of the detour. Some excavation and re-vegetation of the existing north side back slope may be required. If this is required, the existing back slope grading should be restored.

Traffic control lights will be required to facilitate traffic flow through a single lane. Although there will be enough width for two lanes, the lane closest to the treatment area (existing westbound passing lane) should be reserved for construction traffic and to provide separation between traffic and the crest of the excavation.

7.5.2 Phase 2 –Excavation

A relatively deep, temporary excavation is required to facilitate removal of weakened embankment materials within the slope movement area. The temporary excavation is to start at a distance of 3.5 m left of the existing centerline. The granular fills are to be cut at a slope of 1.5H:1V. The clay subgrade is to be cut to the base of clay stratum, exposing the surface of the silt stratum (to Elevation 340.1 m, at Borehole 2-1996, Station 13+476.5, 2.1 m Rt.). The excavation from the existing road grade will be approximately 8.5 m (+/-) deep. The base of the excavation is to be graded at a 3% downward slope to daylight on the existing natural valley slope (in a southerly direction). The length of the excavation base (parallel to the highway alignment) is to extend from Station 13+445 to 13+505 (60 m). At the west and east limits of the excavation the cut slope is to be graded to no steeper than 2H:1V; however a flatter slope may be considered to facilitate construction access and traffic issues.

Dewatering is not expected to be challenging and conventional sump and pump techniques (if required) should be adequate.

The excavated material will need to be removed from site. Should the MTO consider acquiring additional property (including the existing valley), wasting of the excavated materials within the valley may be considered. If this option is considered, a granular covering and/or vegetation would be required as the excavated materials will be highly erodible. Environmental impacts would need to be considered.

The excavation slopes should only be considered stable under temporary conditions as loss in effective strength of the clay will occur as negative porewater pressures dissipate. Visual inspection of the temporary excavation slopes should be carried out for signs of seepage, sloughing and/or other signs of distress. Where distress is observed, additional stabilization measures may be required, such as partial backfilling, the placement of granular sheeting, or other means of stabilization.

7.5.3 Phase 3 – Embankment Reconstruction to Pre-Slope Movement Profile Grade

Phase 3 will involve reconstruction the embankment to the original grades (including all three traffic lanes). The existing right (south) side embankment slope is currently constructed at a grade of approximately 2.3H:1V. Backfill to the underside of the pavement structure is to consist of OPSS Granular “B”, Type III. Vegetation over the reconstructed right (south) side fore slope should be used to limit future surface erosion. The existing culvert at Station 13+487 will also need to be reinstated.

7.5.4 Phase 4 – Removal of Detour and Installation of North Ditch Seepage Barrier

After the embankment has been reinstated, the north side detour can be removed. A seepage barrier is to be constructed along the entire length of the existing subdrain system between Stations 13+225 and 13+480. The seepage barrier should consist of the installation of a geosynthetic clay liner (GCL). Excavation below the existing ditch grade will be required to expose the native clay subgrade. The liner is to be keyed into the clay subgrade and extend across the existing subdrain system. The liner is also to extend along over the left side fore slope of the existing embankment. The GCL is to be installed at a slope of 2H:1V, or flatter and provided with granular cover. Some excavation into the existing left (north) side fore slope may be required to facilitate installation. Care must be exercised during excavation to avoid damage to the existing subdrain system. A conceptual drawing has been provided in Appendix E.

8 Additional Considerations

The natural valley slope, down grade of the existing south side property line and the existing highway embankment is subject to ongoing erosion and soughing. This is particularly evident downstream of the existing culvert located at Station 13+487 where a deep erosion gully has formed within the north side slope of the valley. At some time in the future, ongoing erosion may lead to new stability issues for the existing embankment. Should the acquisition of additional property be considered as part of this project, granular sheeting or vegetation treatment to the valley slope and base should be considered.

The existing centre culvert is expected to be replaced as a part of this project. Where the culvert is within the clay subgrade, a seepage collar should be placed near the upstream end of the culvert to prevent seepage into the subgrade through the granular pipe backfill.

9 Closure

We trust the above addresses your project requirements at this time. Should you have any questions or comments, please do not hesitate the contact us at your convenience.

Yours truly,
For TBT ENGINEERING

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