

Remediation of Slope Failure in the Application of Lightweight Fill and Timber Piles on Highway 17 in Ontario

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ABSTRACT

The failure site is located along Highway 17 approximately 2 km West of Hagar, Ontario. As a part of a frost heave repair, the existing embankment was widened to the north of Highway 17 to allow for temporary traffic flow. During construction, approximately 50 m of the newly widened detour embankment failed. The head scarp of the failure was located at the north edge of original pavement. Following the failure, a Geotechnical Site investigation was carried out to examine the conditions and provide remedial options. The shear key option was chosen as an effective method of dealing with the likely mode of failure. However, a second failure occurred following installation of the shear key, after reinstatement of the highway embankment to the final grade. This failure was assessed to be a deeper-seated slip surface just under the constructed shear key. Several factors may contribute to the second slope failure including: (i) Shear key was not installed deep enough, (ii) Previously weakened slip surface/zone was remolded and lubricated by infiltration of the surface water during the construction; and (iii) Placement of fill materials and heavy construction equipment vibration at 1 m near the slope crest and toe area exacerbating the conditions. Remediation of the second failure involved installation of timber piles driven to bedrock to intercept the failure zone and the use of EPS lightweight fill to reduce embankment loads. A toe berm was also placed to stabilize lower riverbank area. After the second remediation, the highway embankment was stabilized and is performing well with no signs of distress to date. This paper emphasized the importance of Geotechnical Site investigation prior to embankment widening on soft soil near river banks and demonstrates the effectiveness of ground improvement using timber piles and lightweight fills.

RÉSUMÉ

Le site de défaillance est situé le long de la route 17 à environ 2 km à l'ouest de Hagar, en Ontario. Dans le cadre de la réparation du soulèvement dû au gel, le remblai existant a été élargi au nord de la route 17 pour permettre une circulation temporaire. Pendant la construction, environ 50 m de la digue de détour nouvellement élargie ont échoué. La tête de l'effondrement était située au bord nord du trottoir d'origine. À la suite de la défaillance, une étude géotechnique du site a été menée pour examiner les conditions et proposer des solutions. L'option de clé de cisaillement a été choisie comme méthode efficace de gestion du mode de défaillance probable. Cependant, une deuxième défaillance est survenue après l'installation de la clé de cisaillement, après la remise en place du remblai de la route jusqu'au niveau final. Il a été déterminé que cette défaillance était une surface de glissement à assise plus profonde située juste sous la clé de cisaillement construite. Plusieurs facteurs peuvent contribuer à la rupture de la deuxième pente, notamment: (i) la clé de cisaillement n'a pas été installée assez profondément, (ii) la surface / zone de glissement précédemment affaiblie a été remoulée et lubrifiée par infiltration des eaux de surface pendant la construction; et (iii) Mise en place de matériaux de remblayage et de vibrations des équipements de construction lourds à 1 m près de la crête de la pente et de la zone des orverts, ce qui aggrave les conditions. La remise en état de la deuxième défaillance impliquait l'installation de pieux en bois entraînés jusqu'à la roche en place pour intercepter la zone de défaillance et l'utilisation d'un remblai léger en PSE afin de réduire les charges de remblai. Un talus de pied a également été placé pour stabiliser la partie inférieure de la berge. Après la deuxième remise en état, le remblai de la route était stabilisé et fonctionne bien sans aucun signe de détresse à ce jour. Ce document souligne l'importance de l'étude géotechnique du site avant l'élargissement des digues sur un sol meuble près des berges des rivières et démontre l'efficacité de l'amélioration des sols à l'aide de pieux en bois et de remblais légers.

1 INTRODUCTION

The Ministry of Transportation Ontario (MTO) implemented an extensive frost heave repair operation to improve the Highway 17 between North Bay and Sudbury in 2004 (Figure 1). As a part of a frost heave repair operation, a detour was constructed north of Highway 17, approximately 2 km west of Hagar, to allow for temporary traffic flow. The detour extended approximately 6 to 7m north of original highway embankment then sloped at

1H:1V to 2H:1V down to the original ground. The thickness of the fill was approximately 3m high.

Between the evening of October 19 and the morning of October 20, 2004, the detour embankment failed causing the newly placed fill to collapse. The failure extended along a length of highway approximately 50m. The head scarp of the failure was located at the north edge of pavement of the original highway and material moved into Veuve river as shown attached Photo 1.



Figure 1. Site location

Right after failure, a geotechnical site investigation was carried out and recommended remedial options. Upon discussions, the shear key was constructed as a remedial option. However, after reinstatement of the highway embankment to the final grade, a second failure occurred underneath the shear key. This failure was assessed to be a deep-seated slip surface just under the constructed shear key. Several factors may contribute to the second slope failure, probably due to the following reasons:

- Shear key was not installed deep enough,
- Previously weakened slip surface/zone was remolded and lubricated by infiltration of water of the heavy rainfall during the construction,
- Placement of fill materials and heavy construction equipment vibration at 1m near the slope crest and toe area causing remolding underneath soft soils.



Photo 1. Original failure

Remediation of the second failure involved installation and placement of timber piles driven to bedrock to intercept the failure zone and EPS lightweight fill to reduce embankment loads. Toe berm was also placed to stabilize lower riverbank area. After second remediation, the highway embankment was stabilized performing well with no signs of settlement and cracking up to date.

This paper describes the restoration works undertaken for the embankment failed because of improper construction of wider embankment without proper site investigation and embankment design. The paper also describes the geological conditions of the site, construction history of the road and details of failure and restoration.

2 GEOTECHNICAL SITE INVESTIGATION AND SUBSURFACE CONDITION

Following the original failure, the Geotechnical Consultant was invited and reviewed the site condition and performed a geotechnical site investigation. Five boreholes (BH1 to BH5) were drilled within the failure area on October 2004. After second failure, one additional borehole and four rock probes were advanced on February 2005. The location of all boreholes and rock probes are shown on plan on Figure 2.

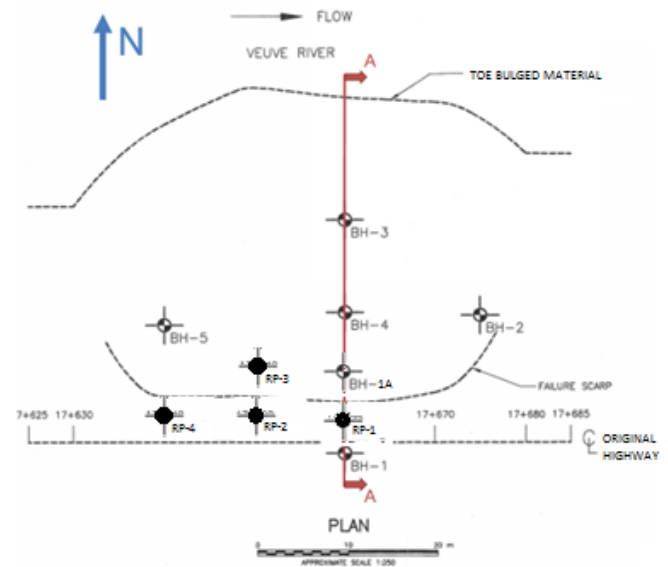


Figure 2. Location of boreholes

The stratigraphy of the failure area generally consisted of granular and roadway fill overlying a thin layer of clayey silt and silty clay in the upper bank area and clayey silt at the lower bank area. The assumed bedrock geometry sloped down from the roadway to the north and also sloped down from the east to west across the limits of the failure. The thickness of the silty clay material ranged from 3 to 4 m near the east limit of the failure, 10 to 12 m near the centre of the failure and was approximately 15 m thick near the west limit of the failure. The intact undrained shear strength of the silty clay as measured by in situ field vane

testing, generally ranged from 16 to 48 kPa. Figure 3 shows the typical stratigraphy of section along A-A.

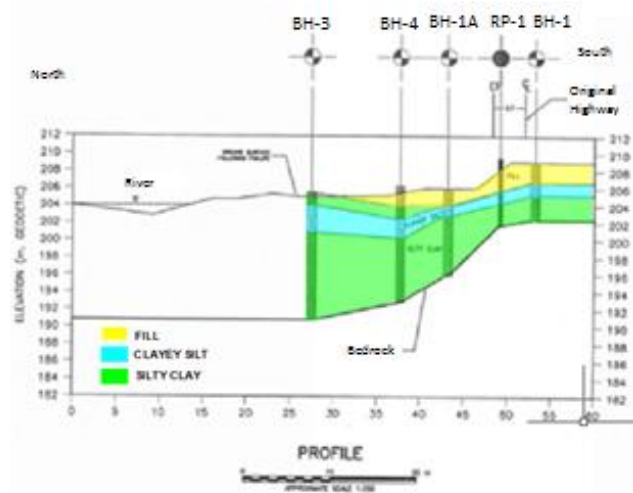


Figure 3. Section along A-A

The undrained shear strength profile for the intact shear strengths from all boreholes is shown as Figure 4. In general, there is a decrease in shear strength from ground surface to approximate Elev. 200 to 199 m and then a general marginal increase in the undrained shear strength with depth. This reflects the typical crust of higher strength and subsequent strength reduction followed by an increase with depth. From this figure, as idealized shear strength profile with depth has been assumed and consists of a decrease in shear strength with depth from ground surface to Elev. 199 m ($C_u = 12$ kPa at Elev. 199 m) and a general increase in undrained shear strength thereafter.

The moisture content generally increases near Elev. 199 m and then a subsequently decrease with depth. The maximum moisture content value of 117% was measured with BH-3 at Elev. 199m as shown in Figure 4.

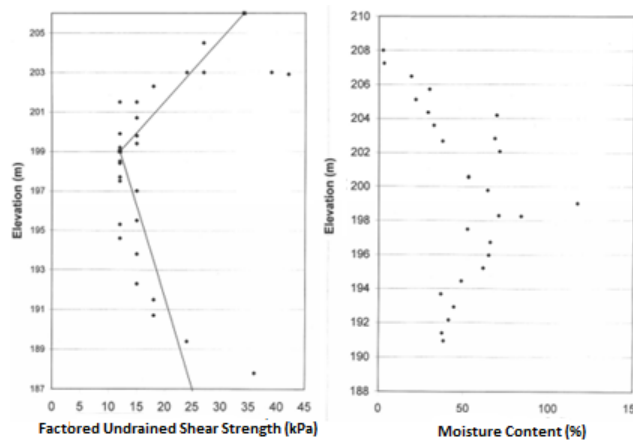


Figure 4. Profile of factored undrained shear strength and moisture content

3 SLOPE STABILITY ASSESSMENT AND REMEDIATION FOR ORIGINAL FAILURE

3.1 Original Failure Assessment

The idealized cross-section for the slope stability was chosen. Using the interpreted strength profile and the moisture content profile, back analyses were carried out to find the critical strength of soils.

3.2 Back Analyses of Original Failure

With the original detour fill in place, a shallow critical block slip surface at the location of the head scarp had an estimated safety factor (FS) of 1.01 assuming an undrained shear strength within the silty clay of 12 kPa above Elev. 199 m and 16 kPa below Elev. 199 m. A potential deep-seated circular slip surface had an estimated FS of 1.17 for the same undrained shear strengths. The shallow critical block slip surface had an estimated FS of 1.03 assuming effective strength parameters of $\phi' = 15^\circ$ and $c' = 3$ kPa. Based on the above, the shallow critical block slip surface matched the actual failure as viewed in the field as to the location of the head scarp and the displacement of the toe material out into the river channel.

3.3 Slope Stability Analysis of Original Remedial Works

With the rock-filled shear key placed and the highway embankment reconstructed to its original design (i.e. the removal of the fill for the detour to the north), the critical upper block slip surface had an estimated FS of 1.41, assuming an undrained shear strength within the native silty clay of 12 kPa above Elev. 199 m and 16 kPa below. Using the effective strength parameters of $\phi' = 15^\circ$ and $c' = 3$ kPa, a potential circular slip surface extending below the shear key had an estimated FS of 1.23. This was assuming full bank saturation conditions. For the effective stress case and the groundwater level at Elev. 204 m (a more realistic assumption), the estimated safety factor of the critical slip surface extending below the shear key was 1.57.

The shear key option as the original remediation works was therefore chosen as an effective method of dealing with the likely mode of failure. The shear key was considered to be relatively simple to construct and was relatively inexpensive.

As part of the exercise, assessments of potential failure below the shear key were discussed and assessed. The total stress analyses using the geometry and stratigraphy yielded a FS of 1.24 for a slip surface at the location of the original failure. It was concluded that using the strength profile interpreted and assuming good construction practice it should be possible to reconstruct the slope and having it remain stable in the undrained condition. In the longer term using effective strength parameters the factors of safety increased as presented above.

3.4 Remediation of Original Failure

Remediation of Original failure involved excavation and construction of rock-filled shear key (5m deep and 5m wide) to intercept the subsurface shear plane/ shear zone

to stabilize the slope, as shown in Photo 2. Rockfill was well-graded angular material with median diameter of 0.3 to 0.5m and maximum 1m. All debris was also excavated in the river channel.



Photo 2. Shear key construction

3.4 Slope Stability Assessment of Failure Following Shear Key Installation

Following the installation of the rock-filled shear key and the reinstatement of the highway embankment, a second failure of the highway embankment occurred on December 10, 2004 following final grading activities. Photo 3 shows the second failure. This failure was obviously deep seated and likely extended below the shear key. The reason of the failure relative to the original slope stability assessment was postulated and the following observations was drawn:

1. The steep cuts required for the shear key installation (even though the excavation was done in short sections) might cause local instability and movement of the silty clay material below Elev.199 m thereby reducing the shear strength as a result of remodeling.
2. Water infiltration by heavy rain along the previous sliding surface during the shear key installation process might act to be remolded and lubricated along disturb zones. This might cause movement of any pre-existing localized soft zones within the silty clay material resulting in instability at deeper levels.
3. During construction there might be unusual loading in terms of fill materials at the upper slope and near the toe, including equipment, etc. These loads and associated vibrations caused by this construction activity might cause movement within the silty clay material beneath the shear key resulting in remodeling and reduction in strength.
4. There might be a non-conservative interpretation of shear strength between Elev. 199 and 194 m in the initial assessment, and/or variations (lower) in the shear strength within the failure are not reflected by the borehole data.
5. There could be some over excavation during removal of material in the river deposited during the initial failure thereby reducing the resisting forces for potential deeper failure.

6. A combination of all the above elements could incrementally participate the second failure.



Photo 3. Second failure

4 SLOPE STABILITY ASSESSMENT AND REMEDIATION FOR SECOND FAILURE

4.1 Additional Geotechnical Investigation

One additional borehole and four rock probes were advanced within the failure area during the week of February 14 to 18, 2005. This new geotechnical information augmented the five boreholes drilled on October 2004. The location of all boreholes and rock probes is shown in plan on Figure 5. Within the failure area, one additional borehole and four rock probes were performed. The results of this new borehole (BH-1A) were similar to the results of the previous boreholes, i.e. granular fill overlying clayey silt and soft to firm silty clay as shown on Figure 3.

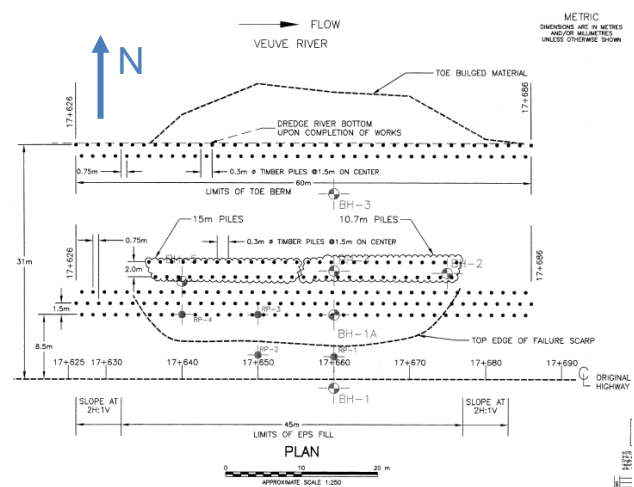


Figure 5. BH/RP locations with timber piles

From the results of the borehole and rock probes, the suspected bedrock surface through the center of the failure

zone (Sta. 17+660) is relatively flat beneath the existing roadway at approximate Elev. 202 m. From approximately 3m north of the centerline of the original highway, the bedrock geometry slopes down at 1H:1V between RP-1 and BH-1A, at 2H:1V between BH-1A and BH-4, and approximately 5H:1V between BH-4 and BH-3, as shown on Figure 3. A similar bedrock geometry was observed at Sta. 17+650, with the bedrock surface sloping at 1H:1V between RP-2 and RP-3. At Sta. 17+640, the bedrock surface slopes at approximately 4H:1V between RP-4 and BH-5. All rock probes that are shown on Table 1.

Table 1. Summary of rock probes

Rock Probe #	Surface Elevation	Depth (m)	Bedrock Elevation (m)
RP-1	209.05	7.32	201.73
RP-2	208.95	7.16	201.76
RP-3	206.23	10.06	196.17
RP-4	206.20	18.0	188.21

4.2 Recommended Remedial Works for the Replacement of the Highway

To allow for the replacement of the highway to its original pre-failure location, the following potential additional works were evaluated.

- Reconstruction of the highway embankment using Expanded Polystyrene (EPS)
- Installation of lower toe berm
- Installation of driven timber piles

A variety of combinations and geometries of the identified potential remedial works have been evaluated to achieve the required design criteria for a factor of safety following all construction of 1.3 and a long-term factor of safety of 1.5. It has been assumed that these factors of safety would need to be achieved following the removal of the failed material within the river channel.

4.3 Slope Stability Assessment

A detailed slope stability assessment has been performed utilizing the computer model SLOPE/W. Utilizing this model, the combination of potential works was evaluated to achieve the required design criteria, while maintaining optimal cost and practicability. Based upon the results of the slope stability analysis, the following works were proposed to achieve the design criteria and control cost:

- Upper bank offloading consisting of EPS fill to reconstruct the highway embankment.
- A 1.0 m thick lower toe berm consisting of a free draining rockfill material.
- The 257 timber piles to be installed.

The limits of all required works is shown on Figures 5, 6 and 7.

Stability analyses were carried out on the overall critical slip surface with all recommended remedial works in place. For the undrained shear strength of $C_u = 12$ kPa for the in-situ soft silty clay material and full bank saturation, the estimated factor of safety of the overall critical slip surface was 1.33. This estimated factor of safety increased to 1.35

assuming a groundwater level at the base of the lightweight fill.

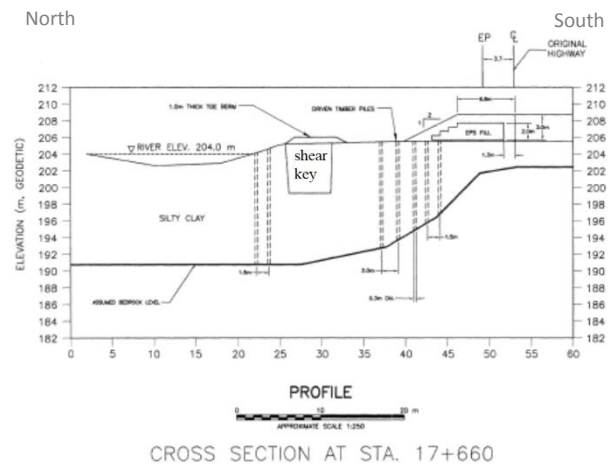


Figure 6. Timber piles, shear key, and EPS embankment

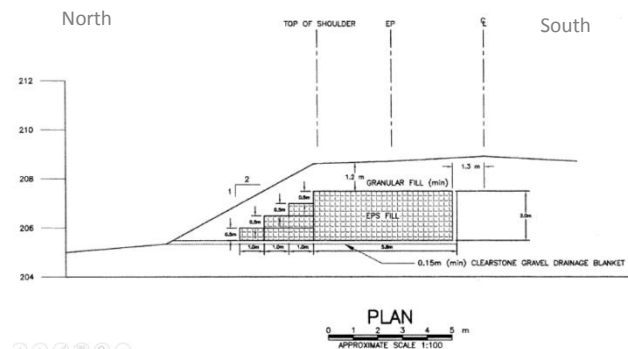


Figure 7. EPS fill embankment

For the long-term conditions of effective strength parameters within the soft silty clay of $\phi' = 12^\circ$ and $c' = 3$ kPa, the overall critical slip surface had an estimated factor of safety of 1.52 for the full bank saturation conditions. The estimated factory of safety of the overall critical slip surface increased to 1.75 for the condition of the groundwater level at the base of the lightweight fill.

5 REMEDIAL CONSTRUCTION WORKS

The following was the construction works for the various stages.

Timber Piles

- All pile had a minimum top diameter of 0.3 m
- The plan location and anticipated length of the 257 timber piles are shown on Figure 5. All piles were driven to refusal (Photo 4).

Lower Toe Berm

- The lower toe berm was constructed using a free draining rockfill material. The material was sized such

that the anticipated maximum water velocity will not cause movement of the rock particles.

- The toe berm had a top Elev. of 206 m and be constructed to a minimum of 5 m wide, as shown in Figure 6.



Photo 4. Timber pile installation

Highway Embankment Construction

- The subgrade soils at the base of the EPS fill placement was prepared and proof rolled to a minimum of 95% of the SPMD Subgrade was sloped to the north a minimum of 2% to promote drainage.
- A minimum of 0.15 m of clear stone gravel (OPSS 1004) or HL-8 course aggregate was placed beneath the EPS. A non-woven geotextile separator (Terrafix 270R or equal) was placed between the subgrade soils and the clear stone gravel.
- The plan limits of the EPS fill are shown on Figure 7 and Photo 5.
- The lightweight fill was installed to the approximate dimensions shown on Figure 7 following all manufacturers' specifications.



Photo 5. EPS fill placement

- The Contractor was to follow all requirements of the MTO Expanded Polystyrene Embankment Special Provisions.

River Bottom Dredging

- After all remedial works were constructed, the failed material within the river channel was removed.
- All material removed from the river channel immediately upon excavation.
- After the river channel was reinstated to its original pre-failure condition, a layer of rockfill riprap was installed. The rockfill rip rap have a mean diameter of 0.3 m, and a maximum diameter of 0.45 m, and be placed to approximately 0.6 m thick, the rip rap extended out into the river channel 3 m, and extended up to the bank to Elev. 206 m.



Photo 6. Rip-rap erosion protection in the river channel

Other Considerations

- Construction activities and traffic patterns were conducted such that there was minimum impact on the embankment slope stability.

6 CONCLUSIONS AND LESSONS

After reconstruction of embankment implemented along recommendations, the new highway embankment was stabilized and it has been performing satisfactory with no sign of distress to date.

The following are conclusions and lessons drawn from this project:

- i. A site investigation is essential to be performed before any construction near creeks or rivers.
- ii. Stability analyses must be carried out before any construction of embankment over soft soils.
- iii. The shear keys option is not enough remedial option where deep soft soil exists under embankment.
- iv. Timber piles driven to bedrock are effective to intercept shear zones.
- v. EPS fills are very effective to reduce embankment load.

- vi. Construction of Lower Toe Berm increases overall stability of new construction.

7 ACKNOWLEDGEMENT

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