

Alberta Highway 858:02 embankment stabilization—geotechnical investigation, design, and construction

X. Wang, H. El-Ramly & D. Proudfoot
Thurber Engineering Ltd., Edmonton, Alberta, Canada

R. Skirrow
Alberta Transportation, Edmonton, Alberta, Canada

ABSTRACT: An embankment failure on Highway 858:02 north of Lac La Biche, Alberta was investigated through field drilling and monitoring programs. The embankment is founded on peat and soft clay deposits. The results of field monitoring indicated that the embankment failure is characterized by translational failure mode. The failure is attributed to high groundwater table in the area and poor foundation soils. Discrete steel piles have been selected to improve the slope stability by providing horizontal restraint to the unstable embankment mass. Slope stability analyses were carried out to determine the additional resisting force required to increase the factor of safety to the desired level. The maximum shear force and bending moment that each pile has to resist were evaluated using lateral pile analyses. The pile type and number of piles were selected based on the results of the lateral pile analyses. During construction, excess pore water pressures generated by pile driving were monitored. The sequence of pile driving was controlled to minimize the risks of triggering further slope movement during construction.

1 INTRODUCTION

Discrete piles have been widely used to improve the stability of slopes which have either failed or are showing incipient signs of failure. The successful use of this method often provides a cost effective remedial measure to stabilize slopes, particularly where limited land availability prohibits an earthwork solution (e.g. re-grading, toe berm, etc.). The analysis and design of slope-stabilizing piles have been described by several investigators (e.g. Poulos 1995, Smethurst & Powrie 2007, Ellis et al. 2010).

This paper presents a case history of using discrete driven steel piles for the stabilization of an embankment slope on Highway 858:02 north of Lac La Biche, Alberta. Investigation of the highway embankment slope failure is first described. Repair design strategies, and measurements of slope movements and pore water pressures during pile installation are then reported.

2 EMBANKMENT SLOPE FAILURE INVESTIGATION

2.1 Slide descriptions

The site of the slope failure is located on the south sideslope of the roadway embankment of Highway 858:02 km 45.85, north of Lac La Biche, Alberta (Fig. 1). This section of Highway 858:02 was

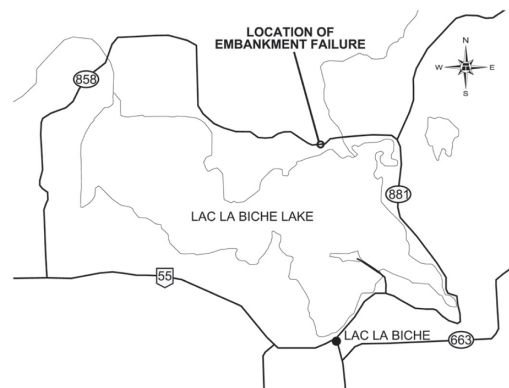


Figure 1. Site plan showing embankment failure location.

initially constructed as a gravel road, but was subsequently paved in the summer of 2005. Distress and cracking of the asphalt pavement on the eastbound lane was first observed in 2006. Because of persistent slope movement and opening of the pavement cracks, patching of the pavement of the affected traffic lane was carried out every year since 2006.

As part of the geotechnical investigation, site reconnaissance and survey of the slide area were carried out in 2010. The results of field observations and survey are presented on Figure 2 and summarized in following paragraphs.

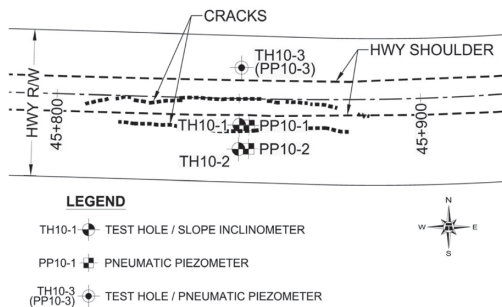


Figure 2. Pavement cracks and test hole locations.

At the slide location, the highway embankment is about 2 m in height and the inclination of the embankment side slope is approximately 11° to the horizontal. The slide area is about 70 m long and 25 m wide in a plan view. The highway east-bound traffic lane experienced extensive cracking and an overall arc shaped pattern of tension cracks was observed in the pavement surface during the site reconnaissance. Relatively continuous cracking was noted along the centerline of the highway and extended for a distance of about 30 m. The crack widths ranged from less than 5 mm on the west limit of the slide to about 50 mm near the centerline of highway and on the east edge of the slide area.

In addition to the cracks on the highway pavement, ground cracks and a shallow graben running parallel to the highway were observed on the south embankment side slope. The cracks on the sideslope were located at a distance of about 3 to 4 m from the edge of the pavement. The crack widths ranged from 5 mm to 75 mm.

The area to the north and south of the highway embankment consisted of muskeg terrain. Muskeg ditches were present on both sides of the highway at the toes of the sideslopes. The south muskeg ditch was about 1.5 to 2 m wide and 0.5 to 0.8 m deep. Accumulation of stagnant water in the muskeg ditch was about 0.3 to 0.5 m deep. There was no distinct bulge observed at or near the toe of the failed slope.

2.2 Subsurface and groundwater conditions

A geotechnical drilling and instrumentation program was carried out in March 2010 to investigate the slide mechanism. Two test holes (TH10-1 and TH10-2) were drilled to depths ranging from 15 m to 16 m below ground surface on the south sideslope within the slide area. One test hole (TH10-3) was drilled to a depth of 15 m on the north sideslope. Two slope inclinometers were installed in test holes TH10-1 and TH10-2 to monitor slope movement. Pneumatic piezometers PP10-1 and PP10-2 were installed at locations adjacent to test holes TH10-1

and TH10-2, and PP10-3 was installed in test hole TH10-3 for monitoring of groundwater condition in the slide area. The locations of test holes and the installed instruments are shown on Figure 2.

The general subsurface conditions encountered at the test hole locations consisted of clay fill, over peat and organic clay, over clay and sand, overlying clay shale. The general soil conditions are summarized on the stratigraphic cross-section presented on Figure 3. Brief descriptions of the soil units encountered at the site are presented below.

Clay fill was encountered in all three test holes and extended to depths ranging from 0.6 m to 5.3 m below ground surface. The clay fill was typically brown to grey, silty, sandy and contained some gravel. The natural moisture contents of the clay fill ranged from 10 percent to 25 percent. One Atterberg limits test carried out on a clay fill sample indicated liquid limit of 46 percent and plastic limit of 18 percent. The SPT blow counts recorded in the clay fill ranged from 5 to 15 blows per 300 mm of spoon penetration, indicating firm to stiff consistency.

Peat was encountered in all three test holes underlying the clay fill. The thickness of peat was 0.4 m in TH10-1 and approximately 0.8 m in TH10-2 and TH10-3. The peat was dark brown, and contained trace amount of silt and some shell fragments. The natural moisture contents of the peat ranged from 117 percent to 271 percent. The SPT blow counts recorded in the peat ranged from 2 to 4 blows per 300 mm of spoon penetration, indicating a soft consistency.

Organic clay was encountered in test holes TH10-2 and TH10-3 underlying the peat. The thickness of the organic clay was 2.3 m in TH10-2 and about 0.4 m in TH10-3. The natural moisture contents of the organic clay ranged from 115 percent to 314 percent. One Atterberg limits test carried out on an organic clay sample indicated liquid

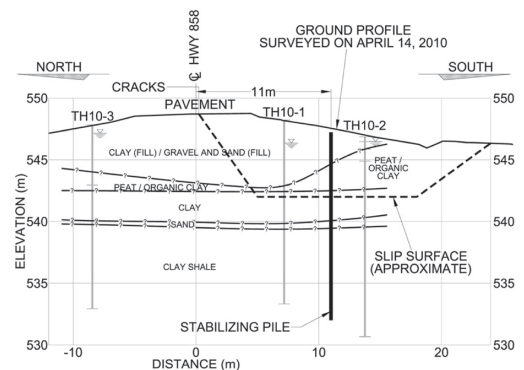


Figure 3. Geotechnical profile of embankment slope and slip surface.

limit of 125 percent and plastic limit of 73 percent. The SPT blow counts recorded in the organic clay were 4 blows per 300 mm of spoon penetration, indicating a soft to firm consistency.

The peat and organic clay were underlain by a clay layer that extended to a depth of approximately 8 m below ground surface in test holes TH10-1 and TH10-3, and to a depth of about 6.5 m in TH10-2. The clay was grey to dark grey, silty and contained some organics in the upper zone. The natural moisture content of the clay ranged from 30 to 60 percent. Atterberg limits test carried out on clay samples indicated liquid limits ranging from 35 to 75 percent and plastic limits ranging from 17 to 33 percent. The SPT blow counts recorded in the clay layer varied from 2 to 8 blows per 300 mm of spoon penetration, indicating a soft to firm consistency. One unconfined compression test carried out on a clay sample at a depth of 6.8 m from test hole TH10-1 yielded an undrained shear strength of 28 kPa.

A sand layer, 0.3 m to 0.6 m in thickness, was encountered in all three test holes underlying the clay. The sand was grey, silty, and fine grained. The natural moisture contents of sand sample ranged from 20 to 23 percent. SPT blow counts recorded in the sand layer ranged from 12 to 14 blows per 300 mm of spoon penetration, indicating a compact relative density.

Clay shale was encountered underlying the sand in all three test holes and extended to termination depths of test holes. The clay shale was grey, high plastic, and contained thin silt laminations. The natural moisture contents of the clay shale ranged from 15 to 25 percent. The SPT blow counts recorded in the clay shale ranged from 15 to 68 blows per 300 mm of spoon penetration, indicating a very stiff to very hard consistency.

Groundwater conditions in the slide area were monitored using three pneumatic piezometers, PP10-1 through PP10-3. The monitoring results indicated that the groundwater table in the slide area was relatively shallow and varied from 0.6 m to 2 m below the ground surface. The groundwater tables in PP10-1 and PP10-2 were lower than the water table in PP10-3, indicating a north to south groundwater flow regime through the embankment fill and foundation soils towards Lake Lac La Biche.

2.3 Slide mechanism

Slope movements were monitored using two slope inclinometers, SI10-1 and SI10-2, installed during the geotechnical investigation. The results of slope monitoring are presented on Figure 4, and show a maximum deflection up to 50 mm along the slip surface over a period of approximately 3 month

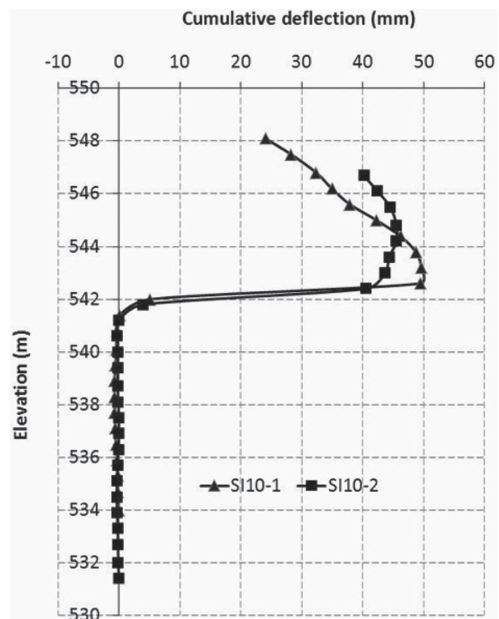


Figure 4. Cumulative deflection measured from slope inclinometers between March 15 and June 16, 2010 (positive values denote downslope movements).

(March to June 2010). In August 2010, both slope inclinometers were sheared off.

Figure 4 indicates a near horizontal slip surface located at elevations 542 to 542.5 m, suggesting a translational failure mode. Based on the field observations and slope monitoring results, an approximate location of the slip surface is shown on Figure 3.

As noted on the borehole logs, the roadway embankment is founded on buried peat and organic clays, overlying a soft clay deposit. The base of the observed slip surface (approximate elevation of 542 m) coincides with the top of the soft clay layer. The embankment failure was likely triggered by elevated groundwater table combined with poor foundation soils characterized by very low shear strength.

The continued patching of the highway surface since 2006 would have resulted in additional load being placed at the slope crest which would have contributed to the on-going slope movement.

3 DESIGN OF STABILIZATION MEASURES

Preliminary slope stabilization designs were first carried out for three remedial options: discrete piles, excavation and reconstruction of highway

embankment, and construction of a toe berm. These options were then compared based on a number of factors including constructability, impacts of construction on roadway traffic, cost-effectiveness, and reliability. The comparative assessment favored the use of driven steel piles to stabilize the embankment slope.

3.1 Design procedure for slope stabilizing piles

A general procedure for the design of slope stabilizing piles consists of following three main steps (Viggiani 1981, Poulos 1995): (1) evaluating the total resisting force required to increase the factor of safety of the slope to the target value; (2) estimating the maximum resisting force that each pile can provide to resist the movement of the unstable mass of the slope; and (3) analyzing and selecting the type and number of piles, and the optimal location of the piles.

Slope stability analyses are generally carried out in step 1. Soil strength parameters used in the stability analyses are typically determined from the results of the geotechnical investigations and back analyses of the existing slope. For an actively moving slope, the factor of safety of the slope is close to unity. The required resisting force is then estimated by the stability analyses to improve the factor of the slope stability from unity to a target value (e.g. 1.3 for roadway embankment side slopes).

Steps 2 and 3 involve laterally loaded pile analyses to determine deflections, shear forces, and bending moments of a pile subjected to sliding soil movement. Poulos (1995) pointed out that effective slope stabilizing piles normally have the following characteristics: (1) have relatively large dimension and stiffness to generate large resisting force without causing failure of the pile itself; (2) extend well below the critical sliding surface to avoid causing deeper slope failure; (3) located in the vicinity of the midpoint of the slope to effectively improve the overall slope stability; and (4) minimize the risk of localized slope failures developing above or below the pile locations.

Regarding pile spacing and optimal location, previous studies demonstrated that the center-to-center spacings required for the discrete piles typically range between 2 and 4 pile diameters (Poulos 1995). Recent studies on discrete piles for slope stabilization by Ellis et al. (2010) have indicated that piles installed at the midslope are most effective in stabilizing the slope.

3.2 Slope stability analyses

Slope stability analyses of the embankment slope were carried out using Slope/W, a program based on limit equilibrium approach. In the stability

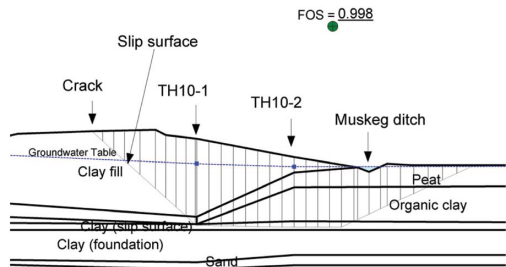
analyses, a transitional slope failure mechanism was assumed and the slip surface was estimated based on field observations and slope movement monitoring results. The soil strength parameters used in the stability analyses are presented in Table 1.

Figure 5 presents the stability analyses of the embankment slope without and with the stabilizing piles. The result of the stability analyses indicate that a resisting force of 65 kN per linear meter along the highway is required to improve the long term factor of safety of the embankment slope from near unity to approximately 1.5. The factor of safety used in the design is greater than 1.3 to take account

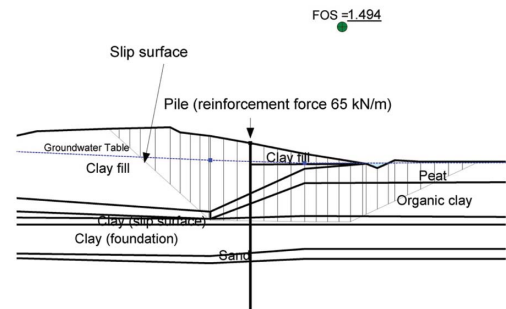
Table 1. Soil parameters used in stability analyses.

Soil type	γ kN/m ³	ϕ' Degrees	c' kPa
Clay fill	20	18	0
Peat	15	20	0
Organic clay	16	10	0
Clay (slip surface)	18	8	0
Clay (foundation)	18	15	5

* γ - unit weight, ϕ' - effective friction angle, c' - effective cohesion.



(a) Slope without stabilizing pile



(b) Slope with stabilizing pile

Figure 5. Slope stability analyses for the design of stabilizing pile.

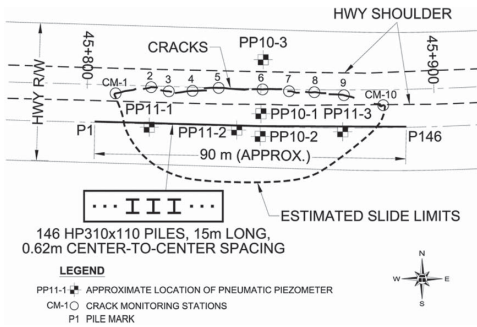


Figure 6. Sketch of the stabilizing pile design.

of future deterioration of the slope. The actual mobilized resistance required to ensure the stability of the embankment slope is expected to be less than the above value specified. (i.e. 65 kN/m).

3.3 Lateral pile analyses

Lateral pile analyses were carried out using the computer program LPILE to estimate the required pile length and spacing, maximum bending moment, and deflection. The resultant force determined from the slope stability analysis was applied as a triangular pressure distribution acting over the section of the pile within the sliding mass.

The results of the lateral pile analyses indicate that discrete steel piles consisting of HP310 × 110 at a center-to-center spacing of 0.62 m (2 times the pile width) will be adequate to stabilize the sliding mass. The pile tip elevation is 532 m (15 m below the ground surface at the pile location), approximately 8 m below the slip surface.

The calculated pile lateral deflection at the ground surface and maximum bending moment are 75 mm and 160 kN · m, respectively. In order for piles to function effectively, the piles are located approximately 11 m from the centerline of the highway, near the midslope location. A total of 146 piles are required to stabilize the failed slope. An illustration of the layout of the stabilizing piles is shown on Figure 6.

4 CONSTRUCTION MONITORING

Prior to and during the early stage of pile construction, the embankment slope was marginally stable. Construction activities and ground vibrations during pile installation can generate excess pore water pressure and may trigger further slope movement. Thus, the stabilizing piles were installed in a prescribed sequence (every seventh pile) to reduce the cumulative impacts of ground vibrations and excess pore water pressure induced

by the piling operation. In addition, an observational approach was implemented to monitor and control impacts of the construction on the slope stability.

4.1 Slope movement monitoring

The slide conditions, including the extent and width of the pavement cracks and settlement of roadway surface, were documented prior to the start of construction. The slide development was continuously monitored during pile installation. The locations where pavement cracks were monitored during construction are shown as CM-1 to CM-10 on Figure 6. The monitoring results were used to assess the construction induced ground movement.

Pile installation was carried out between June 9 and 14, 2011. All the piles were driven from a bench cut on the south embankment sideslope using a Junttan PM20 pile driving rig with a Junttan HHK 5A hydraulic hammer.

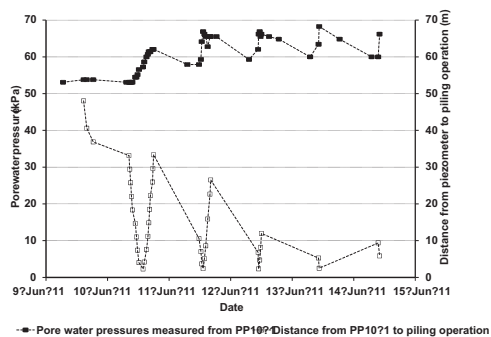
Crack development during the piling operation was estimated by comparing the crack dimensions measured prior to and immediately after the pile installation. The maximum increases in the crack widths and vertical drops of the pavement were approximately 4 mm at CM-8 and 10 mm at CM-6, respectively. The results of crack monitoring indicated that adopted methodology for pile installation minimized the adverse impacts of construction activities on the stability of the marginally stable slope.

4.2 Pore water pressure monitoring

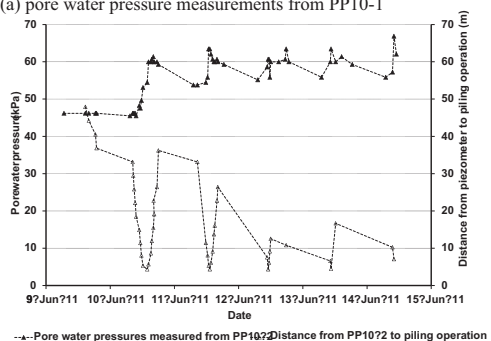
In addition to the three pneumatic piezometers installed during the 2010 geotechnical investigation program, three new pneumatic piezometers (PP11-1 to -3) were installed adjacent to the pile wall to monitor the construction induced pore water pressures. The locations of the six pneumatic piezometers (PP10-1 to 3 and PP11-1 to 3) are shown on Figure 6.

The development of excess pore water pressures at three piezometers located near the center of the pile wall is presented in Figure 7. The plots show the variation of the measured pore water pressures with time and distance to the location of piling operation. Examination of these plots indicates the following observations:

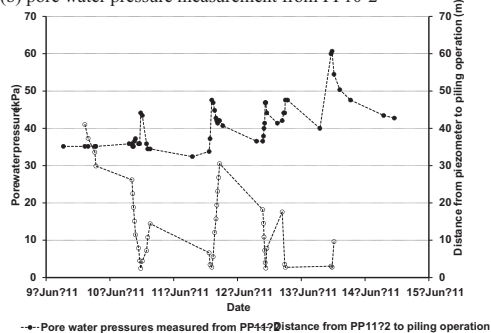
1. Excess pore water pressures typically increased as the distance to the piling operation decreased;
2. A cumulative increase of excess pore water pressure was observed over time as the number of installed piles increased;



(a) pore water pressure measurements from PP10-1



(b) pore water pressure measurement from PP10-2



(c) pore water pressure measurements from PP11-2

Figure 7. Pore water pressure monitoring during pile installation.

3. An increase in pore water pressure was measured as piles were installed within a distance up to 30 m (approximately 100 pile widths) from the piezometer location.
4. The excess pore water pressures generated during the pile installation dissipated relatively rapidly. A significant portion of the construction induced excess pore water pressures dissipated within 24 hours. The presence of the peat layer immediately above the sliding zone was likely responsible for the rapid pore water pressure dissipation.

5 CONCLUSIONS

Geotechnical investigation and slope movement monitoring indicated that the Highway 858:02 embankment failure was the result of a combination of high groundwater table and low strength of the foundation soils. The design of stabilizing pile was performed based on results of slope stability and lateral pile analyses. During pile installation, slide conditions and pore water pressures were continuously monitored to assess the impacts of piling operation on the slope stability. The monitoring results were used successfully to control and modify the sequence of pile installation to minimize the adverse impact of construction on the stability of the slope.

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