Case Study of a Landslide on Highway 20 North of Craven, SK

Harpreet Panesar[1], Hung Vu[2], Laurie Stang[2], Jody Schafer[1], Hamid Soleymani[1]
[1]*Ministry of Highways and Infrastructure, Regina, Saskatchewan*
[2]*Golder Associates Ltd., Saskatoon, Saskatchewan, Canada*

ABSTRACT

Saskatchewan's climate is classified as semi-arid; however, the occurrence of rainfall events and the intensity of rainfall have increased significantly in recent years. An increased frequency and intensity of rainfall events results in more water infiltration and a subsequent increase in groundwater table elevations and reduced soil suctions on the side slopes of highways. Higher groundwater tables and reduced soil suctions are contributing to the destabilization of marginally stable natural slopes and manmade fills across many valleys throughout Southern Saskatchewan.

Two such failures were observed on Highway 20 north of Craven, SK. Highway 20 has a number of high fills across ravines that feed into the valley of Last Mountain Lake just a few kilometers north of Craven where the highway starts to climb out of the valley. The fill at the location of the failure is more than 20 m high. Severe cracking along the shoulder was observed in August 2014.

This paper documents the results of a geohazard risk assessment, terrain analysis, site investigation, instrumentation monitoring, slope stability analysis and an evaluation of remediation options.

RÉSUMÉ

Le climat de la Saskatchewan est classé comme une zone semi-aride. Cependant, une augmentation du nombre de jours de pluie, ainsi que de l'intensité des précipitations a été observée ces dernières années. Une augmentation de la fréquence et de l'intensité des précipitations conduit à une plus grande infiltration d'eau et, par conséquent, à un relèvement de la nappe phréatique, ainsi qu'à une réduction de la succion des sols dans les talus latéraux des autoroutes. Ce qui contribue donc à la déstabilisation des pentes naturelles marginalement stables et des pentes de zones en remblai qui traversent de nombreuses vallées dans tout le sud de la Saskatchewan.

Deux ruptures de ce type ont été observées sur l'autoroute 20 au nord de Craven, SK. Cette autoroute possède de nombreux remblais importants traversant des ravins qui conduisent à la vallée du lac Last Mountain, à quelques kilomètres au nord de Craven, où l'autoroute commence à sortir de la vallée. Le remblai à l'emplacement de la rupture a une hauteur de plus de 20 m. Des fissures importantes ont été observées sur l'accotement de l'autoroute en août 2014.

Cet article présente les résultats de l'évaluation des géorisques, de l'analyse de terrain, des observations sur le site, de l'instrumentation de mesures, de l'analyse numérique de la stabilité des pentes, ainsi qu'une évaluation des options de correction.

1 INTRODUCTION

Landslide activity is a common occurrence along valley walls in southern Saskatchewan. The construction of highways in south Saskatchewan valleys presents a geotechnical challenge due to the inherent landslide risks associated with these susceptible areas. In recent years, the combination of high frequency rainfall events and intense rainfall events has led to landslide activity as a result of increased pore-water pressure and reduced soil suction in highway embankments. The landslide activity occurred north of Craven, SK near Valeport on Provincial Highway 20 (Control Section (CS) 20-01 kilometer 14.7)(Figure 1). Highway 20 connects Highway 11 in the south at Lumsden to Highway 3 in the north near Birch Hills, and is used locally by many seeking to view the Qu'Appelle Valley and Last Mountain Lake.

The steep embankment fill on CS 20-01 is constructed of a till material and intersects natural drainage paths that carry surface water towards Last Mountain Lake (Figure 2). The embankment fill is approximately 150 m long and is more than 20 m in height with 2.3(H):1(V) side slope. Cracks in the pavement have been noted in this location as early as 1991 and have been monitored historically with visual inspections. Site inspection in August 2014 identified more pronounced vertical displacement and cracking in the pavement in both southbound and northbound lanes, leading to an elevated response level (Golder 2015).

2 PHYSICAL SETTING

2.1 Geomorphology

The portion of Saskatchewan to the south of the Canadian Shield is a part of the interior plains. This physiographic region is most heavily influenced at surface from the last deglaciation which occurred approximately 12,000 years ago, where the valleys were carved as glacial meltwater flowed from Glacial Last Mountain Lake through the Qu'Appelle and Assiniboine River Systems into Glacial Lake Agassiz (Christiansen, 1979). The topography of the natural valley slopes is the result of instability which probably began when the spillway bottom was eroded to about elevation 515 m and persisted until the thalweg of the present valley was reached. The area of interest is located along the Last Mountain Lake valley wall; the uplands are composed of a ridged morainal till plain featuring kettle depressions throughout.

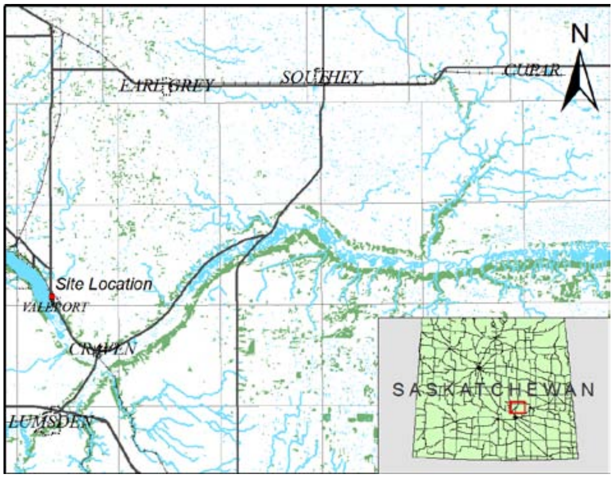


Figure 1. General site location plan

Figure 2. Site location plan showing high fill extent and cracks on pavement

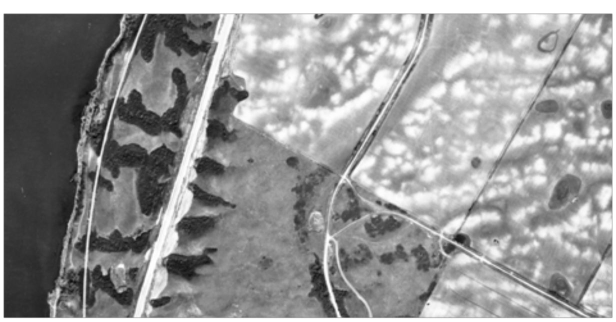


Figure 3. Aerial photograph of the site from 1969

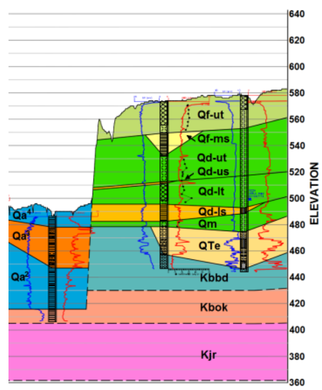
Aerial photographs covering the site area from 1969 onwards were evaluated. Aerial photographs indicate that CS 20-01 was constructed before 1969, and that sections of the highway embankment were constructed over vegetated areas. The photographs indicate that the high embankment fill, at the location of both slides, traverses depression areas of thick natural vegetation.

The old failure scarp on the east ditch back slope shown in Figure 2 is also noted on aerial photographs dating back to 1969. Aerial photographs also reveal that the residential property at the crest of the back slope, approximately 75 m east of the north slide, was constructed sometime after 2001.

2.2 Geology

The Qu'Appelle Valley is a glacial meltwater spillway eroded through 48 m to 65 m of till into clay shale of the Bearpaw Formation. The total depth of erosion is about 109 m below the valley crest at elevation 559 m (thalweg at elevation 450 m). The Bearpaw Formation lies in conformable contact with the Judith River Formation which occurs at about elevation 410 m. (Figure 4)

The contact between the till and the clay shale would likely be represented by a gouge zone in the shale create by glacial shear. Gouge zones are typically in a remolded condition. The effective friction angle of gouge in clay shale of the Bearpaw Formation is most likely at residual state.



Stratigraphy			Lithology	Symbol
Group	Formation	Unit		
Saskatoon	Surficial Stratified Deposits	Alluvium	Clay, Silt, Sand	Qa^4
		Haultain	Silt, Sand, Gravel	Qa^3
			Clay, Silt, Sand	Qa^2
	Floral	Upper	Till	Qf-ut
		Riddell	Gravel, Sand	Qf-ms
Sutherland	Dundurn	Upper	Till	Qd-ut
			Gravel, Sand, Silt, Clay	Qd-us
		Lower	Till	Qd-lt
			Gravel, Sand, Silt, Clay	Qd-ls
	Mennon		Till	Qm
Empress			Gravel, Sand, Silt, Clay	QTe
Montana	Bearpaw	Broderick	Silt and Clay	Kbbd
		Outlook	Sand and Silt	Kbok
	Judith River		Sand and Silt	Kjr

Figure 4. Regional geology of Qu'Appelle valey area (modified cross-section from Water Security Agency)

2.3 Hydrogeology

The valley itself contains Last Mountain Lake, where most regional surface water drains. The boundary of Last Mountain Lake in the south transitions to a wetland, and Last Mountain Creek flows through Craven to the southeast (Figure 5).

2.4 Precipitation Data

In general, moisture conditions and groundwater levels in the soils vary in response to the amount of water available at the ground surface and the amount of discharge or recharge potential of the soil profile, both of which are dependent of the variation of precipitation. Prolonged precipitation over the course of a few days or weeks may result in elevated groundwater levels in the soils that affect slope stability conditions; while a single precipitation event that would typically result in large runoff may cause erosion problems.

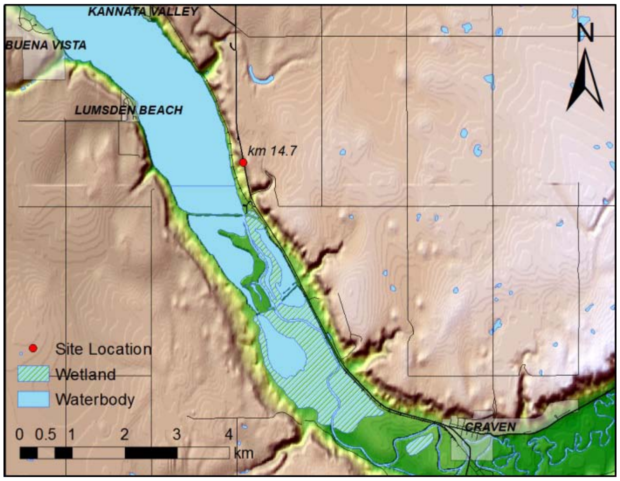


Figure 5. Digital elevation model showing hydrology of the area

A 58 year daily precipitation record for the Cupar area was analyzed to determine the climatic conditions that may have influenced slope stability at the site. The record was based on observations from the Environment Canada Reference Climate Station at Cupar (Station ID 4080) for the years between 1956 and 2014. Cupar is located approximately 50 km northeast of the site. Figure 6 presents the total yearly, maximum, minimum and average values of precipitation for the period from 1956 to 2014.

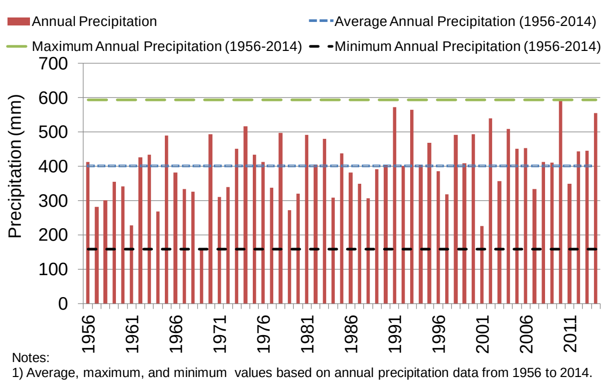


Figure 6. Cupar area annual precipitation (1956 to 2014)

The precipitation data reveals high annual precipitation for the majority of the past decade; recorded annual precipitation during the past decade was above the average value for 1956 to 2014 (401 mm/year), with the exception of 2007 and 2011 (Figure 6). The highest annual precipitation recorded between 1956 and 2014 occurred in 2010, when 595 mm of precipitation was measured; followed by a year of slightly below average annual precipitation in 2011 (349 mm). Annual precipitation measure in 2012 and 2013 was slightly above average, while 2014 was the fourth wettest year on record (555 mm).

Sustained daily precipitation between June 26 and 30, 2014 amounted to 65 mm, with 52 mm concentrated on June 28 and 29, 2014 (Figure 7). Total precipitation recorded for June 2014 was 122 mm.

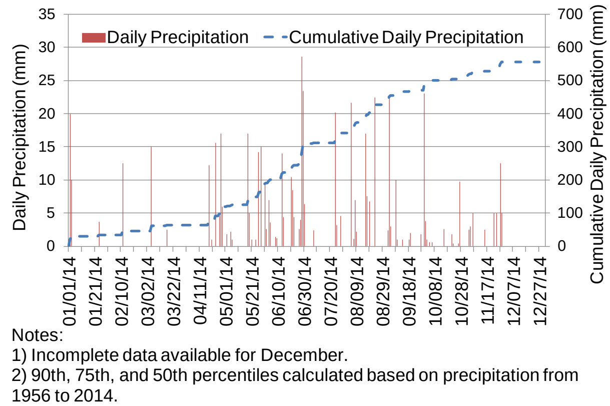


Figure 7. Cupar area daily and cumulative precipitation

3 SITE INVESTIGATION

3.1 Site Reconnaissance

Visual site inspection has been undertaken since 1991. The Ministry's Geohazard Risk Management Program (GRMP) was used for the site inspection since 2011. The GRMP considers the past history, instrumentation input (movement and piezometer data), erosion, seepage and structural distress (pavement and culverts).

Risk was evaluated by defining the likelihood of landslide occurrence or probability factor (PF) and consequences of a landslide or consequence factor (CF). The resultant of the two factors provided a numerical assessment of risk which could be ranked and categorized for response levels and management approach (FIM 2015).

Figure 8 provides the record of historic geohazard risk rating for the site. The site was first included in the GRMP in 2011. A CF equal to 8 was assigned due to the fact that closure of the road would be a direct and unavoidable result of a slide occurrence and additional hazards such as fill height more than 20 m and third party concerns were present (such as house upslope). Cracking was observed throughout the fill section of approximately 150 m, primarily in the southbound lane and extending into the northbound lane in some locations. The landslide risk rating for the east abutment was equal to 78 in year 2011. In fall of 2014, horizontal and vertical displacements of the road surface were more severe (especially in the north slide) than those observed in previous years, results in an elevated risk level of 120.

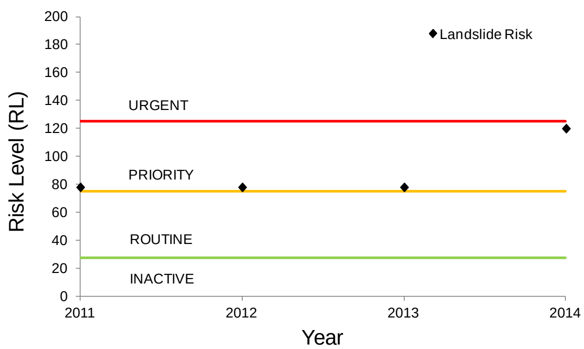


Figure 8. Geohazard Risk Rating for the site

Figure 9. 2014 site inspection photos of the north slide (looking in the south direction)

3.2 Topographic Survey

The topographic survey was completed using GPS survey equipment. Horizontal datum was referenced to the NAD 83 Transverse Mercator coordinate system; surface feature elevations were referenced to geodetic datum CGVD28. Figure 10 shows the plan view of the survey area, contours and slope stability cross-sections. Digital Terrain Model (DTM) data was used to extend the contour plan beyond the limits of the topographic survey. Cross-sections A-A' and B-B' are also shown on Figure 10.

3.3 Field Investigation

Field investigation involved auger drilling up to a maximum depth of 32.8 meters below ground surface (mbgs) along the highway and east ditch (in October 2014) and test pitting up to 5.0 mbgs at the toe of the embankment (February 2015). Borehole and test pit locations are shown in Figure 13.

The general stratigraphy encountered in the failure area includes embankment fill (silty clayey till) overlying native till (silty clayey till). Till at the till/fill contact has trace of organic material. Bedrock shale was not encountered to the depth of the investigation.

3.4 Laboratory Testing

Laboratory tests conducted on representative soil samples included visual classification, water content, Atterberg limits, unit weight, specific gravity, grain size analysis, direct shear tests, and soil-water characteristic curve (SWCC) test.

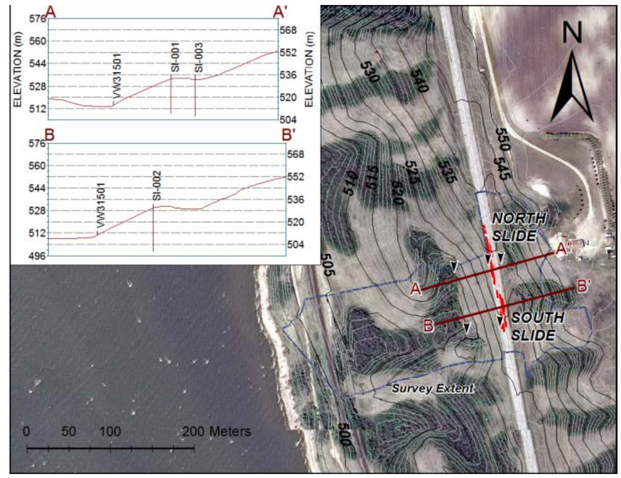


Figure 10. Site contour and location plan

The liquid limit for the silty clay till (CL) varied from 29% to 38% with plasticity index values ranging from 17 to 26. Natural moisture content in samples obtained from near ground surface at the toe of the embankment were higher than the liquid limit, indicating trafficability might be a potential problem during the construction at the toe.

Direct shear test on till fill sample did not exhibit strain softening (i.e., the till fill sample showed only slight decrease in resistance following its peak strength at large strain). The direct shear test results suggest that the risk of sudden catastrophic failure at this embankment is low and the expected failure would be gradual unless there is significant change in groundwater conditions.

Figure 11 shows the measured SWCC's for the till fill sample from this site. The air-entry value (AEV) determined from the measured SWCC for this sample was approximately 200 kPa and there was no distinct residual value; however it appeared to be around 10,000 kPa.

The shear strength of an unsaturated soil is described using the following equation (Fredlund et al. 1978):

$$\tau_f = c' + (\sigma_n - u_a)\tan\phi' + (u_a - u_w)\tan\phi^b \quad [1]$$

Where φ^b is the angle indicating the rate of increase in shear strength relative to the matric suction. The suction strength is expressed as a linear equation:

$$s = (u_a - u_w)\tan\phi^b \quad [2]$$

In slope stability analysis the contribution to the shear strength by suction is simulated by the input of either φ^b or the "apparent cohesion" (i.e., the strength arising from suction). Based on the estimated unsaturated shear strength envelope shown in Figure 12 the angle φ^b for till is approximately 15 degrees.

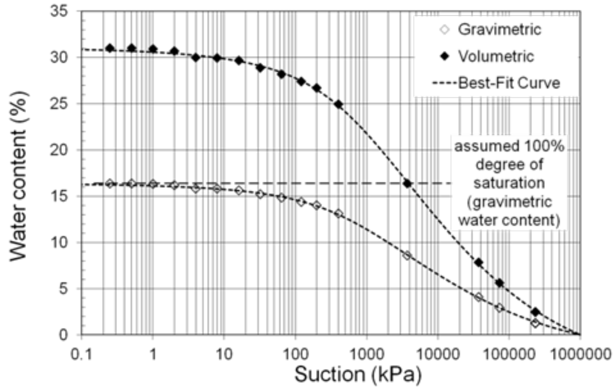


Figure 11. Measured SWCC for till fill

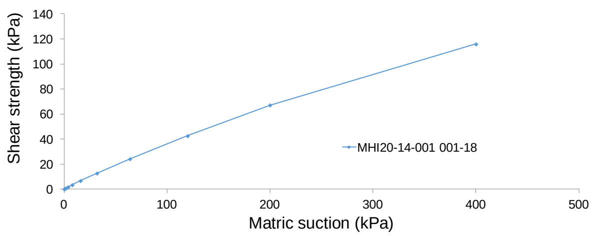


Figure 12. Estimated unsaturated shear strength envelope for till fill

3.5 Instrumentation Monitoring

Instrumentation installed at this site includes slope inclinometers (to measure slope movement), survey pins (to monitor movement at ground surface), and vibrating wire piezometers and stand pipe piezometers (to monitor pore water pressures). The location of the instrumentation is shown in Figure 13.

Shear displacement vectors at SI001 (approximately 5.5 mbgs/ 520 masl) and SI002 (approximately 6 mbgs/ 526 masl) are shown in Figure 13. SI003 showed a slight movement at approximately 19 mbgs/513 masl, indicating a potential multiple zone of movement at the site..

The piezometer nest along the embankment crests indicated a downward flow gradient of up to 1.0. The piezometer nest at the east ditch indicated a hydrostatic condition. Groundwater elevations measured at the toe of the embankment were less than 1.0 mbgs. Groundwater elevations in general are expected to be low during winter, then increase during the spring and summer months.

4 SLOPE STABILITY ANALYSIS

4.1 Methodology

The slope stability analysis was performed using the computer software SLOPE/W, marketed by Geo-Slope International Ltd. (2007). Two-dimensional analyses were conducted using the Morgenstern-Price limit equilibrium method. Both deterministic slope analysis and probabilistic slope analysis were conducted.

Figure 13. Instrumentation Location Plan

4.1.1 Deterministic slope stability analyses

Determination of a minimum acceptable factor of safety for a slope stability model depends on several factors, including the reliability of the input parameters and the consequence of failure. Stratigraphy, soil properties and piezometric conditions were inferred based on available information.

The analysis methodology included first back analyzing the existing failure conditions of the north failure to a factor of safety (F_S) of 1.0, using deterministic slope analysis, to determine the mobilized shear strength parameters at the failure location (Figure 15). Cross sections for the stability analysis were obtained from the topographic contour plan of the site (Figure 13). Cross section A-A' was selected for the back analysis, being most unstable section of embankment.

Typically, for a factor of safety increase of 10% (i.e., F_S =1.1), some deformation and creeping is expected. An increase in factor of safety of 50% (i.e., F_S = 1.5) typically means that only limited long term slope monitoring and maintenance may be required. For the current project, the slope remediation measure was set to achieve at least 30% increase in factor of safety (i.e., F_S =1.3) with some future monitoring of the slope after the remediation.

4.1.2 Probabilistic slope stability analyses

The factor of safety is an index indicating the relative stability of a slope. Factor of safety alone does not imply the actual risk level associated with a slope, since some of the input parameters have variability. A higher factor of safety may not necessarily correspond to a lower probability of failure because the probability of failure is dependent on the degree of uncertainty of the parameters and the accuracy of the analysis model.

A probabilistic analysis allows the engineer to assess the likelihood of failure in addition to the factor of safety associated with mean values. The probability of failure and the reliability index are two useful indices that can be used to quantify the stability conditions or the likelihood of failure of a slope.

Instead of calculating a single factor of safety for a slope, a distribution of factor of safety is calculated from repeated iterations of the stability model using different combinations of input parameters selected from credible ranges using Monte Carlo sampling techniques. The probability of failure, P_f is then computed as the percentage of analyses performed where the factor of safety was less than 1.0 as follows:

$$P_f = P[F_S \leq 1.0] \quad [3]$$

The reliability index, β describes safety by the number of standard deviations (i.e., the amount of uncertainty in the calculated value of factor of safety) separating the best estimate of factor of safety from its defined value of 1.0. The reliability index is defined in terms of the mean (μ_F) and the standard deviation (σ_F) as follows:

$$\beta = \frac{\mu_F - 1.0}{\sigma_F} \quad [4]$$

The probability of failure and the reliability index provide a measure of the uncertainty involved in the results of the analysis and thus in the probability of stability state of the slope. For example, a slope with a factor of safety of 1.5 and a standard of deviation of 0.5 (e.g., reliability index of 1.0) may have much higher probability of failure than a slope with factor of safety of 1.2 and a standard deviation of 0.1 (e.g., reliability index of 2.0).

Reliability concepts can be applied to provide a logical framework for choosing factors of safety that are appropriate for the degree of uncertainty and the consequences of failure involved (Duncan 2000).

Table 1 adapted from U.S. Army Corps of Engineers 1999 provides example values for the reliability index along with the probability of failure and an expected performance level for levees.

Table 1. Target reliability indices for levees (adapted from U.S. Army Corps of Engineers 1999)

Reliability Index	Probability of Failure	Expected Performance Level
5.0	2.817×10^{-7}	High
4.0	3.169×10^{-5}	Good
3.0	0.00135	Above average
2.5	0.00621	Below average
2.0	0.02275	Poor
1.5	0.06681	Unsatisfactory
1.0	0.15866	Hazardous

Similar to the minimum factor of safety concept, different reliability indices or probability of failure are required for different conditions, failure modes and consequences. Unlike levees, most highway embankments are not designed to act as flood control structures. Currently, within geotechnical industry there is no acceptable probability of failure and reliability index values for highway embankments. There is also no direct relationship between the deterministic factor of safety and the probability of failure or the reliability index. Based on the current project requirements, a desired reliability index was set at 2.0 or higher.

4.2 Potential Slope Failure Modes at the Site

Based on site topography, stratigraphy, hydrogeology and slope instability conditions observed along the Qu'Appelle River valley, the failure mechanisms under consideration for the current site are outlined in Table 2 and graphically shown in Figure 14.

Table 2. Potential failure mechanisms

ID	Potential Failure Mechanism
FM1	Circular slip surface through the till fill and foundation till
FM2	Composite slip surface through the contact between the till fill and the foundation till
FM3	Circular slip surface in the upper slope
FM4	Circular, deep seated slip surface in overall slope
FM5	Composite slip surface in glacial till and underlying shale

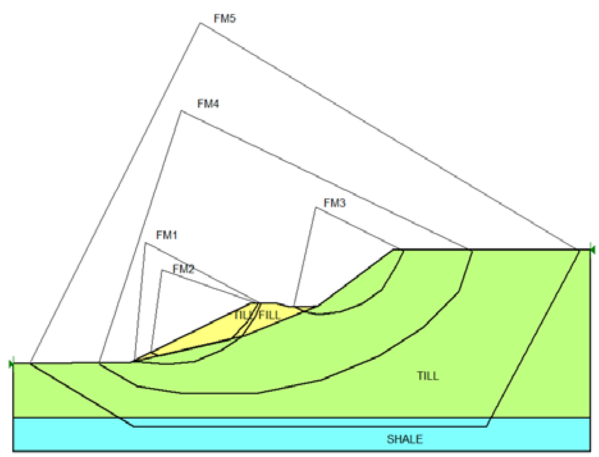


Figure 14. Illustration of potential slope failure mechanisms

The observed failures at the north slide area and south slide areas are associated with failure mechanisms FM1 or FM2. The old scarp from a shallow failure on the east ditch back slope south of the two slides (see Figure 2) was associated with failure mode FM3. This study addresses the failure through the till fill (failure mechanisms FM1 and/or FM2). Because shale was not countered to the depth of investigation, failure mode FM5 was not considered in this study. The soil investigation and instrumentation program were conducted to provide geotechnical information for the remediation of the embankment slope. Any proposed remediation option should not have negative impact on failure mechanisms FM3 (stability condition of upper slope) and FM4 (stability condition of overall slope).

Seepage analysis was not conducted to evaluate ground water conditions of the site because of inadequate site specific geotechnical information. One piezometric line was assumed based on available piezometer data to represent the shallow groundwater conditions in the embankment till fill and till under the embankment. There is significant uncertainty associated with assumed groundwater levels for failure mechanisms FM3 and FM4.

4.3 Material Properties

Soil properties required for the stability analysis include the unit weight, effective friction angle, the effective cohesion and angle ϕ^b (i.e., rate of increase in shear strength relative to matric suction).

Material properties for the slope stability analysis were selected based on laboratory test results, field test results, back-analysed values, and typical values reported in the literature. Table 3 shows the range of shear strength properties used for the slope stability analysis.

Table 3. Model shear strength parameters

Material	Unit Weight γ_w (kN/m^3)	Cohesion c (kPa)	Effective Friction Angle ϕ' (˚)	Angle ϕ^b (˚)
Till Fill[1]	20 (19-22)	4 (0-15)	25 (20-30)	15 (10-20)
Till	21 (18-23)	5 (0-20)	27 (20-26)	15 (10-20)
New Till Fill	20 (19-22)	10 (0-15)	23 (20-26)	15 (10-20)
Granular Fill	20	0	35	0

[1]Numbers in parentheses indicate the range considered in the probabilistic analysis

4.4 Slope Stability Analysis Results

A parametric analysis was completed to assess the sensitivities of the calculated factor of safety to variability in material properties. The results of the parametric study indicated that the effective friction angle and effective cohesion of till are the factors that have the greatest effect on the calculated slope F_S. However, the effective cohesion and angle ϕ^b of till fill appear to control the shape of the slip surface.

Several remediation options were evaluated, namely finger drains at the toe of the embankment, drainage trench (10 m deep) installed along the east ditch, combination of finger drains and drainage trench (5 m and 10 m deep), steel H piles installed along the embankment crest and a toe berm constructed at the toe of the slope. Table 4 provides the results of deterministic and probabilistic analyses. Table 5 provides a summary of F_S increase, along with advantages and disadvantages of various remediation options. Based on the results, the toe berm was selected as the preferred remediation option for this site. Because there is uncertainty in soil conditions at the toe location, field monitoring during the construction (e.g., visual observation and instrumentation monitoring) is recommended.

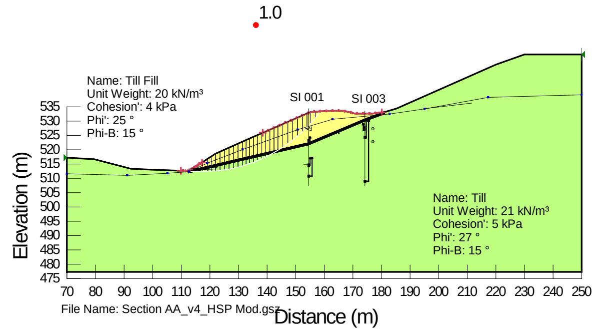


Figure 15. Factor of safety map with F_S increment of 0.01 for the back analysis of cross-section A-A’

5 RECOMMENDATIONS

Currently, within geotechnical industry there is no acceptable probability of failure and reliability index values for highway embankments. There is also no direct relationship between the deterministic factor of safety and the probability of failure or the reliability index. It would be valuable to perform probabilistic analyses of historic and future slope failures and develop an acceptable framework.

The results of this study demonstrate the limitations of the deterministic slope stability analysis approach alone. The use of probabilistic and deterministic slope analyses would provide a more efficient framework for the investigation and design of remedial measures for slope stability.

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Table 4. Results of deterministic and probabilistic analyses

Cross Section	Failure Mechanism	Description	Deterministic F_S	Probabilistic Analysis					
				Mean F_S	Standard Deviation	Min F_S	Max F_S	Probability of Failure (%)	Reliability Index
A-A' North Slide	FM1	Back Analysis	1.00	1.05	0.10	0.74	1.40	33.16	0.46
	FM2	Back Analysis, Till Impenetrable	1.02	1.09	0.13	0.74	1.49	25.84	0.68
	FM3	Upper Slope	1.49	1.36	0.15	0.95	1.80	0.10	2.46
	FM4	Overall Slope	1.34	1.53	0.16	1.01	2.03	0.00	3.26
	FM1	Winter conditions	1.23	1.27	0.12	0.90	1.69	0.94	2.17
	FM1	Finger Drain	1.18	1.21	0.12	0.87	1.61	3.42	1.75
	FM1	Drainage Trench 10 m deep	1.18	1.28	0.15	0.85	1.84	1.82	1.87
	FM1	Finger drain and drainage trench 10 m deep	1.37	1.40	0.14	0.99	1.83	0.01	2.88
	FM1	Finger drain and drainage trench 5 m deep	1.28	1.31	0.13	0.94	1.71	0.16	2.48
	FM1	H Steel Pile	1.01	1.07	0.11	0.74	1.43	29.04	0.59
	FM1	Toe Berm	1.31	1.38	0.14	0.98	1.86	0.06	2.67
B-B' South Slide	FM1	Back Analysis	1.06	1.10	0.11	0.75	1.46	19.22	0.88
	FM2	Back Analysis, Till Impenetrable	1.11	1.19	0.14	0.82	1.61	7.96	1.38
	FM3	Upper Slope	1.34	1.35	0.14	0.97	1.78	0.34	2.48
	FM4	Overall Slope	1.35	1.38	0.15	0.95	1.85	0.38	2.51
	FM1	Winter conditions	1.21	1.25	0.13	0.85	1.65	3.02	1.88
	FM1	Toe Berm	1.31	1.39	0.16	0.97	1.88	0.09	2.56

Table 5. Summary of various remediation options

Option	Advantages	Disadvantages
Finger Drains	✓ Low remediation cost ✓ Constructible during winter ✓ Highway closure not required ✓ Relatively short construction time (likely 2-4 weeks) ✓ Specialized equipment not required	✓ Minimal increase in F_S ✓ Maintenance required to make sure drains are performing adequately ✓ Susceptible to damage/ sedimentation
Drainage Trench (10 m depth)	✓ Relatively short construction time (likely 2-4 weeks) ✓ May only require closure of one lane of traffic	✓ Minimal increase in F_S ✓ High remediation cost ✓ Maintenance required to make sure drains are performing ✓ Susceptible to damage/ sedimentation ✓ Specialized equipment required for deep trench excavation ✓ Partial/ full highway closure required ✓ Does not address failure at the toe of the slope ✓ Not constructible during winter
Drainage Trench (10 m depth) & Finger Drains	✓ Suitable F_S increase for short term condition ✓ May only require closure of one lane of traffic ✓ May be constructed simultaneously or in stages	✓ Minimal increase in F_S ✓ High remediation cost ✓ Maintenance required to make sure drains are performing ✓ Susceptible to damage/ sedimentation ✓ Specialized equipment required for deep trench excavation ✓ Partial/ full highway closure ✓ Not constructible during winter
Drainage Trench (10 m depth) & Finger Drains	✓ Moderate increase in F_S ✓ May only require closure of one lane of traffic ✓ May be constructed simultaneously or in stages	✓ High remediation cost ✓ Maintenance required to make sure drains are performing adequately ✓ Susceptible to damage/ sedimentation ✓ Partial/ full highway closure required
Steel Piles	✓ Minimal construction footprint (within right of way) ✓ Relatively short construction time (likely 2-4 weeks)	✓ Marginal increase in F_S ✓ High remediation cost ✓ Specialized construction ✓ High risk solution, may be difficult to drive piles through the till ✓ Full highway closure required ✓ Does not address failure at the toe of the slope
Toe Berm	✓ Toe berm can be sized to achieve required F_S ✓ Borrow material available close to the site ✓ Specialized equipment not required ✓ Highway closure not required ✓ F_S increase for overall till slope	✓ High remediation cost ✓ Large disturbance area ✓ Construction would extend outside of right of way.