Slope failure in clay shale in western Manitoba: a case study

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ABSTRACT
Slope instabilities have affected Provincial Trunk Highway 5 near Dauphin Manitoba for over 50 years. In 2011, Manitoba Infrastructure and Transportation began investigating possible failure mechanisms and remedial alternatives. Slope inclinometers and piezometers were monitored over two years. Laboratory testing compared direct shear and torsional ring shear tests from underlying Cretaceous clay shale. Coupled finite element modeling examined excess pore-water pressures and slope stability during construction of a stabilizing berm. Modeling results will be used to compare with the measured pore-water pressure response following construction.

RÉSUMÉ
Instabilités de pente ont affecté la route provinciale 5, près de Dauphin au Manitoba depuis plus de 50 ans. En 2011, Infrastructure et Transports Manitoba a commencé à enquêter sur les mécanismes de défaillance possibles et solutions correctives. Inclinomètres et piézomètres de pente ont été suivis sur deux ans. Les tests de laboratoire par rapport cisaillement direct et les essais de cisaillement de torsion du sous-jacent schiste argileux du Crétacé. Modélisation par éléments finis couplé examiné pressions interstitielles en excès et la stabilité de la pente lors de la construction d'une berme de stabilisation. Résultats de la modélisation seront utilisés pour comparer avec la réponse de la pression de l'eau interstitielle mesurée après la construction.

1 INTRODUCTION

With an increasing number of slope failures affecting its infrastructure, Manitoba Infrastructure and Transportation (MIT) began using a Geohazard Management System to better prioritize geohazard sites and manage associated risks to the provincial highway system. A Geographical Information System (GIS) model for managing the risks was presented to MIT in 2007 (Baldwin et al. 2007). It is based on successful risk management systems used in Alberta and Saskatchewan. While MIT has yet to fully adopt the GIS system, it has implemented a simpler risk management system modified from those used by Manitoba's western neighbours.

The MIT currently has approximately 60 active geohazard sites. While these include issues such as culvert damage or highway pavement frost boils, the majority are slope failures or landslides. MIT's Geohazard Management System assesses risk at a particular site by assigning qualitative measures of probability of failure (PF) and consequence of failure (CF) based on experience and engineering judgment. An additional factor called the User Impact Rating (UIR) is determined from an assessment of such factors as annual average daily traffic (AADT), potential length of detour required, population affected, and the potential impact on emergency services. The UIR and PF are added, and then multiplied by the CF to assign a level of risk with respect to stability. This measure of risk assigns a numerical value that allows a particular site to be ranked, and therefore prioritized in relation to other geohazard sites in a particular region of the province.

The site chosen for this case study has been ranked among those with the highest priority in northwestern Manitoba. The study is still in progress, with site investigation and laboratory testing completed and instrumentation installed. Analysis has examined possible remedial solutions and allowed safe construction of a stabilizing berm. The paper describes this stage of the project. Further analysis remains for the future in comparing measurements from the field instruments with ongoing time-dependent analysis, as well as exploring failure mechanisms.

1.1 Project Background

The study site is approximately 15 km west of Dauphin, MB (Figure 1), where Provincial Trunk Highway (PTH) 5 crosses the Mineral Creek valley. In the late fall of 2007, a grade slope failure was reported near Ashville Junction at the intersection of PTH 5 with PTH 10. The failure consisted of two separate slumps at the road shoulder. The slumps were re-graded and the pavement was patched intermittently when slope movements continued. By late 2010, the area affected by slope movements had grown larger, with cracks developing between the two slump locations; into the east bound travel lane (EBL); and into the southbound turning lane. In spring 2011, after extended periods of rainfall and subsequent flooding of the valley, the failure expanded into the EBL. It resulted in partial lane closures of PTH 5 at the Ashville Junction (Figure 2). Progress of the failure meant that re-grading and patching was no longer adequate. Manitoba Infrastructure and Transportation initiated a geotechnical investigation to better understand the failure. The site was instrumented later that year with four slope inclinometers, five vibrating wire piezometers and one standpipe piezometer.

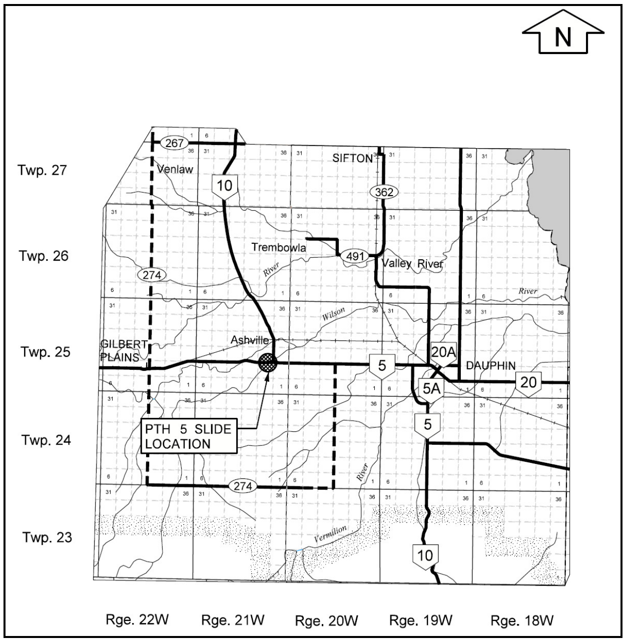

Figure 1. Project Site Location

Figure 2. East on PTH 5 - Cracked Pavement Surface

1.2 Site Conditions

The grade slope in the study area is heavily vegetated with trees, shrubs and grasses. Boulders left following post-glacial erosion of till occur intermittently along the slope. Where the highway crosses Mineral Creek, it cuts into the side slope of the valley, exposing weathered clay shale where the cut is most pronounced. The back slopes required re-grading following two failures in 1988. At its lowest point, the highway is approximately 9 m below natural prairie level. The north side of the highway is ditched, sloping west to east toward Mineral Creek. The south side slopes directly into the valley.

Multiple scarp lines exist on the grade slope, suggesting retrogressive movement back up the slope toward the highway over time. The scarp lines indicate that the width of the failure along the highway is approximately 100 m. The shoulder slumps mentioned earlier affect the pavement at the east-most extent of the failure. Significant cracking and slumping of the EBL has occurred near the centre of the failure. The total extent of the failure affecting the pavement is approximately 70 m. Cracking was also observed near the centerline in the west bound lane (WBL). It is uncertain if the cracks are directly due to slope movements: no significant change has occurred since the initial site investigation in 2011.

Provincial Trunk Highway 10 ends at its junction with PTH5, but a gravel municipal road (Mile Road 121W) continues south from the junction. The gravel road is subject to washouts during significant flood events on Mineral Creek. Local residents claim that washouts have increased in frequency and magnitude since 2009. The gravel road crosses Mineral Creek approximately 200 m south of the Ashville Junction above an under-sized corrugated steel pipe culvert. The pipe once had a weir at the inlet but this was washed out in June, 2013. During flood events, the culvert is overwhelmed, and water backs up, washing out the road approximately 30 m south of the Ashville Junction. During normal stream flow, the water surface elevation of Mineral Creek is typically 337.0 m. Flood events raise the water surface elevation to approximately 340.9 m, flooding much of the study area. Most of the substantial slope movements appear to follow these flooding events.

2 METHODOLOGY

The study proceeded as follows:

- Review the site history, including previous site investigations and analyses;
- Analyze data collected through current site investigations and laboratory tests;
- Use limit equilibrium (LE) to back analyze the failure at the critical section to determine shear strength properties;
- Apply a coupled finite element model (FEM) to predict the pore-water pressure response, and therefore stability, during construction of stabilization measures;

3 SITE HISTORY

A review of the history of the site showed that the Ashville Junction slide has affected PTH 5 for over 50 years. In 1964, four test holes were drilled when regional staff reported slope movements. At that time, a saturated layer of blue-grey bentonite was believed to have led to the instabilities. The location of the test holes is unclear, as there is no available reference point. Following the site investigation, several recommendations were made for improving the stability of the slope. In 1965, surface drainage works were completed in an attempt to divert runoff from the site into Mineral Creek a few hundred metres west of the failure. Recommendations were also made to bench cut and re-compact disturbed soils, construct a toe berm, and install subsurface drainage. No records could be found that indicate any of these options were completed.

The failure was active again in 1974. A survey revealed multiple scarp lines, consistent with retrogressive failure, along the grade slope from the south shoulder of PTH 5 down to the toe. Small cracks were observed in the highway pavement surface. Three more test holes were drilled at the site and again revealed a blue-grey layer of bentonite in the shale. The bentonite was once more identified as the cause of failure and was shown to occur at the same orientation as a proposed failure surface. (The site was not instrumented). Several recommendations were made to improve stability, although in the end, only minor re-grading of the site was undertaken. In 1993, the intersection was widened to improve sight lines and add an east-bound through lane. In doing so, the highway was raised by up to 0.75 m to meet then-current MIT highway design standards. This produced additional loading at the crest of the side slope. The toe of the slope at the most critical section of the failure is now approximately 9 m below the existing pavement elevation. The grade slope varies between 5H:1V and 7H:1V. Movements continued following reconstruction of the intersection in 1993. It became clear in the fall of 2007 that the pavement was being significantly affected and remedial work was needed.

4 SITE GEOLOGY

4.1 Surficial Geology

The Mineral Creek valley was formed following the retreat of glacial Lake Agassiz. While not as pronounced as many of the larger post-glacial valleys in the region, the valley is fairly substantial in its depth and breadth, with relief of approximately 15 m in the study area. The base of the valley is alluvial sediments. Following flooding and subsequent washout of Mile Road 121W in the spring of 2013, these sediments were exposed by the flood waters. The exposed soils consisted of layered sands and gravels, stratified silts, and fine sands. These were deposited by river action from behind the Manitoba escarpment following lake retreat, and from beach deposits formed to the west when glacial lake waters were still prevalent (Matile et al. 2004).

The valley walls are comprised of till-like colluvium, deposited following erosion of the valley. The colluvium is clay-rich and contains variable quantities of silt, sand and gravel. Selenite (hydrous calcium sulphate) crystals are common in the soil matrix, and are seen at sizes up to approximately 15 mm in diameter. Large boulders of various origins deposited during glaciation are evident where erosion and weathering of the valley walls has left them exposed.

4.2 Bedrock Geology

Approximately 2 km northwest of the Ashville Junction is an outcrop of Cretaceous clay shale, exposed through erosion of the Wilson River Valley. The Manitoba Geological Survey (Bamburak et al. 2004) identifies this outcrop as the Upper Ashville Formation. The Mineral Creek valley cuts through this deposit in the study area. The Ashville Formation was deposited approximately 100 million years ago, and has a maximum depositional thickness of approximately 80 m. The deposit is carbonaceous shale, dark grey to black in colour, and is generally non-calcareous (not containing $CaCO_3$), although some minor bands of calcareous material have been observed. The formation also contains various relatively thin layers of bentonite. The bentonite is generally blue-grey in colour and weathers to yellow-orange (Bamburak et al. 2004). The layers of bentonite exposed at the Wilson River outcrop are consistent with this description, and are well defined at their contacts with subsequent soil layers. The Upper Ashville Formation, also called the Belle Fourche Formation, weathers into flat, chip-like pieces; may turn a brownish-grey colour when exposed; and selenite crystals are common. The small exposure on the north side of PTH 5 in the study area has weathered into small fragments, with relatively large selenite crystals occurring frequently.

5 ANALYSIS

5.1 Steady State Seepage FEM Calibration

A steady state seepage model was created to generate a pore-water pressure distribution for the original cross section (Figure 3) using Seep/W from Geo-Slope International[1]. The model was calibrated using groundwater data collected from logged vibrating wire piezometers corrected for barometric effects with an on-site barometric piezometer. Total head boundary conditions were applied to the left (H_L) and right (H_R) boundaries of the domain, with a no flow (du = 0) condition applied at the lower boundary. The right boundary was extended to a distance of approximately 400 m from the highway centerline to the divide between the Wilson River and Mineral Creek watersheds. This distance was estimated from topographic maps of the area. A constant head boundary was applied to the creek (H_C) to represent normal flow conditions at the site.

Hydraulic conductivity values for the shale (Cummings et al. 2012) and alluvium (Pliakas et al. 2011) are assumed within acceptable values as determined from the literature. Hydraulic conductivity values for the colluvium were determined from flexible wall permeameter tests and back calculation from oedometer tests. The mean saturated vertical hydraulic conductivity (k_v) as determined through these tests was 6.3×10^{-9} m/s for the applicable stress range. This value was subsequently adjusted during calibration of the model.

In order to better reflect the groundwater conditions

[1] GeoStudio 2007, GeoSlope International, Calgary, AB

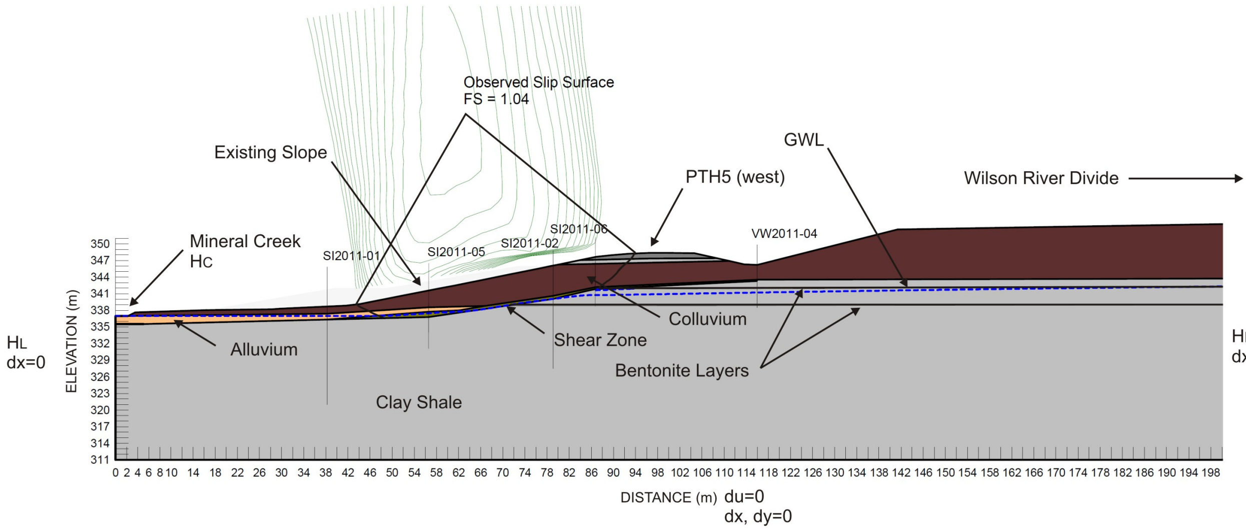

Figure 3. Analyzed Section

observed by the instrumentation, a hydraulic conductivity ratio of 0.001 was initially applied to the colluvium. This is likely a physically unrealistic representation of the hydraulic properties of the colluvium, as a ratio of 0.01 implies almost completely horizontal flow (Seep/W, 2010). A closer examination of the subsurface conditions was needed to determine a more realistic representation of the observed groundwater flow regime.

The 2011 site investigation revealed two nearly horizontal layers of bentonite. Shelby tube samples also showed considerable fracturing, weathering and gravel-sized limestone at the interface of the shale bedrock and the Quaternary soils above, suggesting that a localized bed of relatively higher permeable soil may exist at this location. These conditions were simulated by adding lines in the model at the location of each material layer, and extending the lines across the domain (Figure 3). Interface elements can then be applied to these lines, effectively creating a new region, with assumed hydraulic properties assigned to each respective material. Table 1 summarizes the vertical and horizontal hydraulic conductivities determined for each material:

Table 1. Modeled Hydraulic Conductivity

Soil	k_v (m/s)	k_h (m/s)	k_v/k_h
Colluvium	1 e^{-8}	1 e^{-07}	0.1
Alluvium	5 e^{-06}	5 e^{-05}	0.1
Shale	5 e^{-11}	5 e^{-10}	0.1
Bentonite	1e^{-12}	1e^{-11}	0.1
Fractured Zone	1e^{-5}	1e^{-7}	0.01

In other Cretaceous formations, head drops have been observed to occur across layers of bentonite[2]. While the mineralogy of the bentonite is such that it has an affinity for water storage (reflected in laboratory tests of moisture content), it does not have the same affinity for water flow. It can therefore be less permeable than the surrounding shale, particularly if the shale has a fractured macrostructure as observed in core samples taken during the 2011 site investigation. Total head drops across the two layers of bentonite are seen in the model (Figure 4). Of note in the steady state model is the development of a perched water table near vibrating wire piezometer, VW2011-06. The observed piezometric elevation at this location was abnormally high with respect to the other instruments, suggesting that a perched water table may exist at this location.

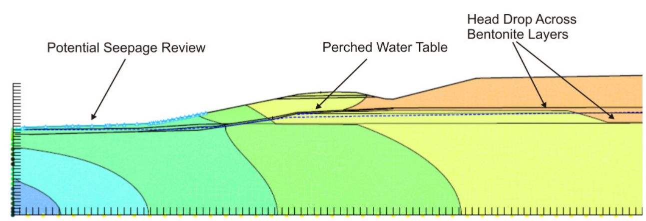

Figure 4. Total Head Contours (H = 334.0 m – 342.0 m)

5.2 Back Analysis using Limit Equilibrium

Using Slope/W from Geo-Studio International, limit equilibrium was used to back analyze the section and evaluate shear strength parameters for the soils. Figure 3 shows the cross-section, the piezometric surface, and the observed slip surface used in the analysis. The Morgenstern-Price method of slices was used, with shear strength parameters being adjusted until a factor of safety (FS) near unity was reached. A composite slip surface was analyzed, with the shale being designated as impenetrable bedrock and the remaining materials modeled using Mohr-Coulomb (c, ϕ) strengths. A shear zone of weakened shale was applied at the interface of the colluvium and shale bedrock where the failure surface was observed to occur. Figure 3 also shows the location of two bentonite seams encountered during the 2011 investigation. The previous investigations did not identify separate layers, likely due to the limited number of test holes drilled at that time. The relatively horizontal orientation of these layers with respect to the observed

2 Personal communication, G. van der Kamp, 2014

failure surface suggests that they are not the primary cause of failure.

A series of laboratory tests was completed to help verify the shear strength parameters identified in the LE back-analysis. Direct shear testing is the oldest test method used for determining shear strength parameters, and is especially suitable for testing stiff clays and shales (Terzaghi, 1987). However, determination of residual shear strength parameters requires large strains which, in the direct shear test, require multiple reversals of the shearing apparatus and may not truly represent the displacements along a failure plane.

The torsional ring shear test has the advantage of continuous, unidirectional straining of a specimen, although obtaining an intact specimen to fit the ring apparatus is difficult. Stark and Eid (1994) successfully used the torsional ring shear apparatus with reconstituted specimens for the determination of residual shear strength values. This method was therefore used to test reconstituted specimens of clay shale collected from the Ashville Junction site.

Consolidated, undrained triaxial (CIŪ) tests (with pore water pressure measurement) were completed on samples of colluvium from the valley wall for the determination of soil strength parameters to be used in the analysis. Table 2 compares the back- analyzed and mean laboratory shear strength values.

Table 2. Shear Strength Parameters

Soil	ϕ'_r (DS[1])	ϕ'_r (RS[2])	ϕ'_r (BA[3])	ϕ'_{cs} (CIŪ)	c'(kPa)
Colluvium	-	-	25°	27°	0
Shale	10.1°	9.0°	8.0°	-	0

[1] direct shear
[2] ring shear
[3] back analysis

5.3 Coupled Stress-PWP FEM

Several stabilization options were considered for the site. Soil nailing was first considered. However, to keep the project within budgetary constraints, only local stability at the edge of road could be addressed using three rows of nails. It was determined that over time, continued movements of the slope below the stabilized area might eventually undermine the nails and again damage the pavement surface.

A shear key was then considered, but there was concern that trenching in the marginally stable slope might induce further movement and cause significant damage to the pavement surface during construction. To avoid trenching, rock fill columns were then considered. Locally, rock fill columns are a commonly used method for riverbank stabilization (Thiessen et al. 2011). However, the equipment required is relatively large and there was concern that the earthworks required for preparing the site for the equipment, as well as the load applied by the equipment itself, might further destabilize the slope during installation.

Construction of a stabilization berm with free draining, granular fill was considered the most suitable option for both its simplicity and economy, and would not require significant excavation of the slope during construction. A coupled stress-pore water pressure FEM model was created with Sigma/W from GeoSlope International to assess the excess pore-water pressure generated by loading from the fill, and determine if staged loading would be required to maintain stability during construction.

The model was created with a mesh of quadrilateral and triangular elements, with a maximum global element size of 1.5 m. A mesh-size ratio of three was applied to the global element size to decrease the size of the mesh at selected areas of the domain where high gradients or stress concentrations might occur, primarily between the foundation soils and the granular fill. The left and right boundaries of the domain were fixed in the x (dx = 0) direction, with the lower boundary being fixed in both the x and y (dy = 0) directions (Figure 3). Loading of the embankment was modeled so that uniform lifts of granular fill were placed over a period of 16 days. An elastic-plastic constitutive model was selected for the colluvium, while a linear elastic model was selected for both the alluvium and shale.

Young's modulus (E) for the alluvium was assumed within acceptable values for the material type as determined from literature (Budhu, 2007). Young's moduli for the colluvium and shale were determined from laboratory and in-situ testing, respectively. The initial tangent modulus was taken from plots of deviator stress versus strain for CIŪ tests completed on samples of colluvium at different depths. The data were plotted as E versus vertical effective stress (σ'_v), and a best-fit function of the data was imported into the coupled model. Figure 5 is a plot of the elastic modulus function used for the colluvium. It shows an increase in stiffness with depth.

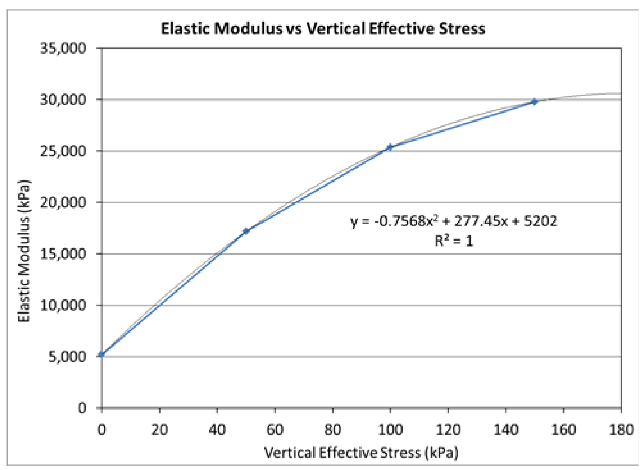

Figure 5. E vs σ'_v for Colluvium

Anochikwa et al. (2012) describe a method for determining elastic and hydraulic properties of a saturated, isotropic and linearly elastic aquitard. Only elastic properties have been examined in this case study, as the Ashville Junction site was not instrumented to collect the data required to determine k_v. Because they are affected by changes in barometric pressure, observed pore-water pressure responses may generally appear irregular. An on-site barometric piezometer can be used to apply a correction to these data to eliminate barometric effects. Changes in barometric pressure can be multiplied by this correction factor, which may be adjusted until barometric effects have been eliminated from a plot of the data. As described in this method, the applied correction factor represents the loading efficiency, commonly referred to as B-bar (here written as B) in geotechnical engineering. Changes in pore-water pressure in response to an applied surface load are described by:

$$B = \Delta u/(\Delta \sigma_B) \qquad [1]$$

The B term describes the loading efficiency, u is the pore-water pressure and σ_B is the barometric pressure. If the soil is considered a saturated, linearly elastic and isotropic material, B can also be described by:

$$B = (1/E_c)/(1/E_c + n/E_w) \qquad [2]$$

Here, E_c is the constrained elastic modulus, E_w is the bulk modulus of elasticity of water (~2.2×10^9 Pa), and n is the soil porosity. Assuming that the measured barometric response is undrained, E_c can be shown as:

$$E_c = (E_w - BE_w)/(Bn) \qquad [3]$$

Young's modulus E, can be determined using the relationship between E, Poisson's ratio, v, and E_c:

$$E = E_c(1+v)(1-2v)/(1-v) \qquad [4]$$

It is common to assume v = 1/3. As such, E can then be defined as:

$$E = 2/3\ E_c \qquad [5]$$

Figures 6 (a) and (b) show plots of the corrected data for VW piezometers installed at different depths in the shale. The removal of barometric effects is reflected as a "smoothing" of the data with the B correction applied. A value of B = 0.9 was determined for the shale using a "trial and error" approach. This means that under loading, a pore-water pressure response of 90% of the applied load will be induced. The value of B can then be used as an input parameter for coupled modeling. The load response ratio in Sigma/W is equivalent to the coefficient B, as determined above (Sigma/W, 2010). Using the above relationships between B, v, n and E, Young's modulus for the shale was then determined to be 388 MPa. This value was used in the coupled model. Table 3 shows a summary of the elastic properties used for each material in the model.

5.4 FEM Slope Stability

Because LE slope stability calculations are based only on statics, strains induced by loading of the embankment are not accounted for. Stresses computed by finite element can be imported into a limit equilibrium analysis, eliminating the need to make assumptions about the interslice forces (Krahn, 2003), although assumptions must now be made with respect to elastic properties. The FEM slope stability analyses were completed after every

Table 3. Modeled Elastic Properties

Soil	E (kPa)	Unit Wt (kN/m^3)	v
Colluvium	varies[1]	19.0	0.35
Shale	388,000	18.5	0.34
Weakened Shale	30,000[2]	18.5	0.34
Alluvium	40,000	20.0	0.30
Granular Fill	60,000[3]	19.0	0.30

[1]function shown in Figure 5
[2]applied to shear zone (Figure 3)
[3]dry, gravelly sand, track compacted in winter

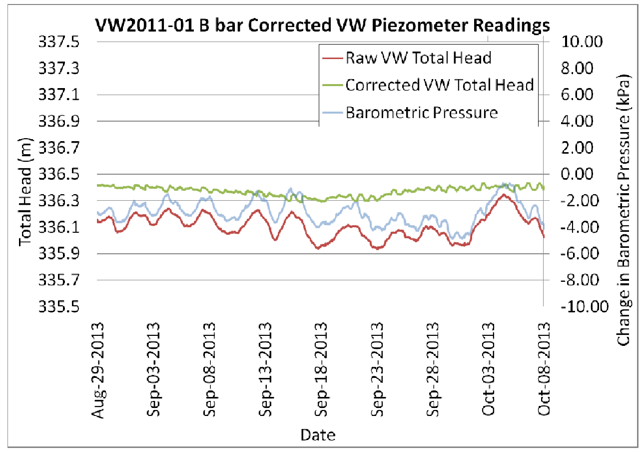

Figure 6 (a). VW Piezometer, VW2011-01, B-bar Corrected Data (tip depth of 17.4 m)

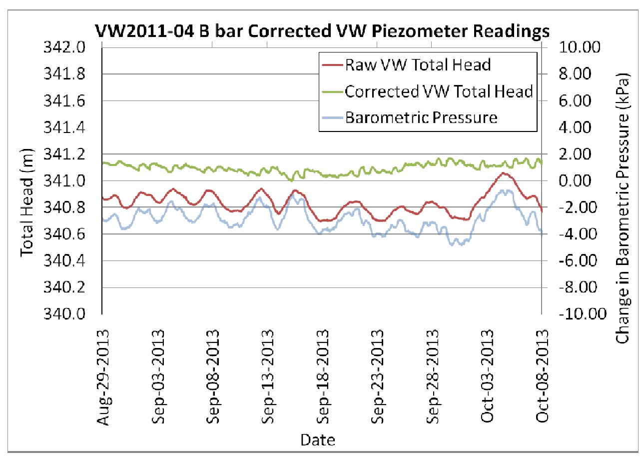

Figure 6 (b). VW Piezometer, VW2011-04 B-bar Corrected Data (tip depth of 8.0 m)

four lifts applied to the model. As the stresses change with applied loading, the stability model uses the re-calculated stresses from the parent analysis at each stage of stability analysis. A total of four analyses were completed during loading, with a fifth being completed following dissipation of excess pore-water pressures. This allows an assessment of stability at various stages of construction, as well as under long-term drained conditions. Figure 7 shows the constructed berm and associated factor of safety following dissipation of excess pore water pressures over a one-year period.

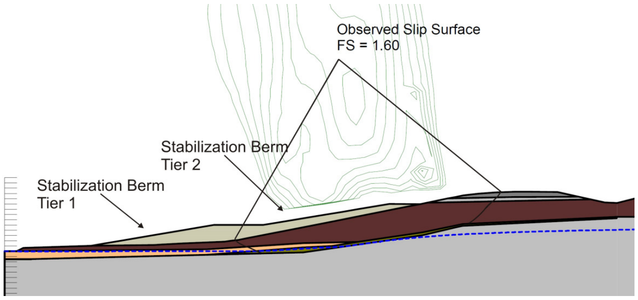

Figure 7. Modeled Stabilization Berm

6 RESULTS AND DISCUSSION

History matching appears to produce better correlation between the shear strength parameters measured in the torsional ring shear tests and back analyzed values from LE analysis than those measured from direct shear tests. Of course, increasing the sample size may offer better insight into the validity of these results, but only limited intact samples were available for direct shear testing. Ring shear testing is ongoing and results will be included in future publications.

The coupled FEM allows for the prediction of excess pore-water pressure generation, which may lead to instability during construction, as well as settlements of the foundation soils. The prediction of settlements can be useful in determining an appropriate timeline for addressing the damaged pavement surface. Replacing the surface can be costly, and construction prior to settlements being within serviceability tolerances may lead to additional costs of pavement repair. Figure 8 shows a plot of excess pore-water pressure (plotted as total head) versus time during fill placement and pore-water pressure dissipation in the model. These data will be compared with the actual response in future work. Figure 9 shows a plot of foundation soil displacement at different stages of loading in the model. Figure 9 shows that predicted displacements are minimal. As expected, the maximum displacement occurs beneath the location of greatest fill placement with a magnitude of approximately 6 mm. This is considered reasonable given the relatively thin layer of natural soils that occur between the granular fill and the underlying clay shale, but will be compared to actual settlements following an additional survey of the site.

Figure 10 shows a plot of FS versus time for both the critical slip surface and the observed slip surface, with each day corresponding to one applied lift of granular fill. Following the eighth lift, the FS for the critical slip surface is around 1.3, as the failure surface moved up the slope, above the first tier of fill (Figure 7). A second tier was then added to ensure that the minimum required FS could be maintained. The application of the second tier is reflected in a slight dip in FS following lift 12, but allows the minimum FS of 1.5 to be maintained for both the lower berm and upper berm failure mechanisms.

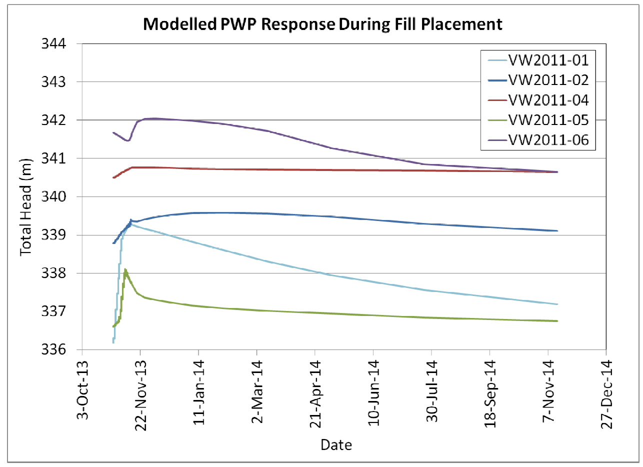

Figure 8. Total Head vs Time During Fill Placement

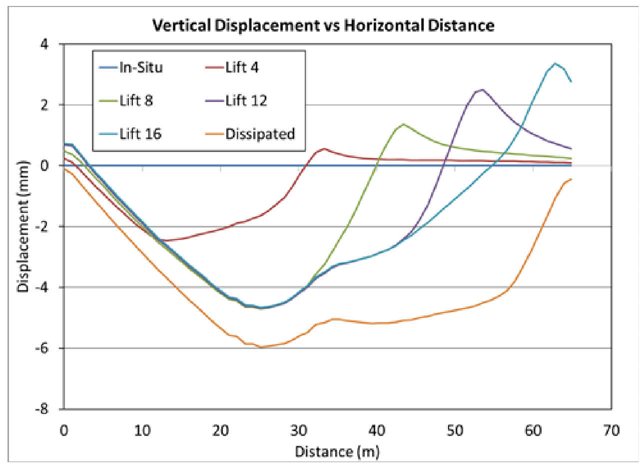

Figure 9. Displacement vs Distance for Foundation Soils

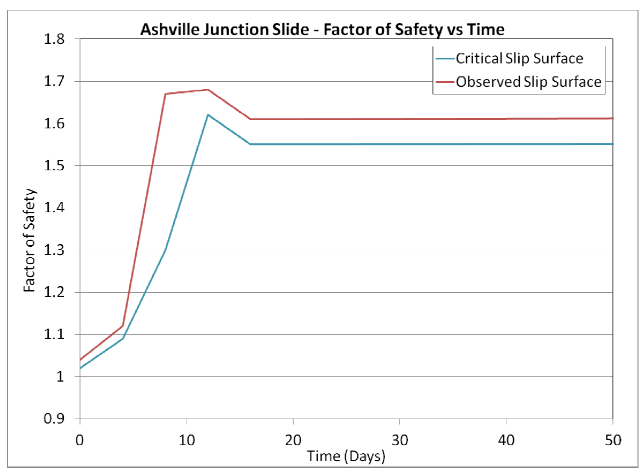

Figure 10. Factor of Safety Versus Loading Stage

7 CONCLUSIONS

This paper presents the methodology used to address one of the geohazards of highest priority affecting Manitoba highway infrastructure. It highlights the importance of sound field work and in-situ testing, laboratory testing, data collection, and the review of historical data in the analysis of geohazards. With reliable data, numerical modeling can be an excellent tool in predicting soil behaviour during geohazard site remediation.

The shear strength parameters determined from LE back analysis agree reasonably well with laboratory determined values, and can be applied confidently to the analysis of this site. The calibrated steady state seepage model shows good correlation between predicted and observed values of pore-water pressure, given the relative geologic complexity of the site. The pore-water pressure distribution generated with the coupled FEM model, and the results of finite element stability analyses show that staged loading would not be required to maintain stability during construction.

Construction of the stabilization berm was completed in late 2013, and data collection is ongoing. Additional instrumentation is to be installed to monitor post-construction slope movements in late spring 2014. Once the logged vibrating wire piezometer data is compiled, the modeled construction response will be calibrated against the collected data. Following the spring thaw in 2014, an additional survey is to be completed to compare predicted and actual settlements of the foundation soils. Validation of the model with these data will be presented in future.

Additional analyses are also being completed to determine the failure mechanisms associated with this geohazard. In particular, the Mineral Creek valley is prone to cyclic flash flooding and additional numerical modeling is being completed to determine the effect of these flood events on slope instability.

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