Design and Construction of Rockfill Columns in Deep Clay Deposits for Riverbank Stabilization in St. Jean Baptiste, Manitoba

James R. Bartz
Department of Civil Engineering - University of Manitoba, Winnipeg, Manitoba, Canada
Rob Kenyon, Bruno Pierre Arpin, David Flynn
KGS Group, Winnipeg, Manitoba, Canada
Alena James
Manitoba Infrastructure, Winnipeg, Manitoba, Canada

ABSTRACT
The integrity of flood protection dikes for the town of St. Jean Baptiste in southern Manitoba were at risk due to slope failures along the Red River through Lake Agassiz clay deposits. Manitoba Infrastructure completed stabilization works to protect the dikes adjacent to over 100 m of riverbank with the construction of rockfill columns. This is a common landslide stabilization technique in the region, however the depth of the clay deposit and artesian groundwater conditions in the underlying aquifer provided unique design and constructability challenges. This paper presents the design including deformation predictions, construction details, and a review of the performance to date.

L'intégrité des digues protectrices contre les inondations dans la ville de Saint-Jean-Baptiste, dans la région sud du Manitoba, était menacée par des talus défaillants le long de la rivière Rouge à cause des dépôts d'argile du lac Agassiz. Infrastructure Manitoba a terminé les travaux de stabilisation sur plus de 100 mètres de rive afin de protéger les digues adjacentes avec la construction de colonnes d'enrochement. Il s'agit d'une technique courante de stabilisation des glissements de terrain dans la région, mais la profondeur des dépôts d'argile et la condition des eaux souterraines artésiennes dans l'aquifère sous-jacent a posé des problèmes de conception et de constructibilité uniques. Cet article présente la conception, y compris les prédictions de déformation, les détails de construction, et une évaluation des résultats à ce jour.

1 INTRODUCTION

Riverbank failures along the Red River are common along outside bends where the river is adjacent to and cutting into Lake Agassiz sediments (James, 2009). The landslides are commonly triggered by fluctuating river elevations and groundwater elevations (Tutkaluk, 2000). Although the typical rate of landslide displacement can be described as very slow to extremely slow with the Cruden and Varnes (1996) classification, the ongoing displacements are a concern for structures and infrastructure along the crest of the slope. Such was the case in St. Jean Baptiste, Manitoba where riverbank failures along the Red River threatened the integrity of a ring dike protecting the town.

Riverbank stabilization works consisted of rockfill columns along the mid and upper bank, a riprap toe berm along the mid and lower bank, and rockfill drainage channels. The depth of clay and artesian groundwater levels provided challenges for design and construction of the works. The design incorporated unique components including the deepest rockfill columns constructed by Manitoba Infrastructure (M.I.) to date as well as passive drainage elements to reduce groundwater pressures in the underlying till aquifer.

This paper describes the background and history of riverbank instability in the community, the design of the stabilization works, the construction methods used, the ongoing performance of the bank, and lessons learned from the project.

2 BACKGROUND

St. Jean Baptiste, Manitoba is a predominantly francophone farming community located approximately 60 km south of Winnipeg as shown on Figure 1. The town was founded in the 1870’s by Métis families from St. Norbert that were joined by families from Québec and the eastern United States. The community is located along the west bank of the Red River and is protected from flood events by a ring dike. The dike is comprised of a combination of clay dike, sheet pile wall, and a bin wall dike. The dike was first constructed in the 1960s and was subsequently raised in height in response to the 1979 and 1997 flood events. The current crest elevation ranges from approximately 239.7 m to 240.2 m above sea level.

The portion of riverbank that runs parallel to the dike has a history of instability (Penner, 2007). In 2009, routine inspections by M.I. revealed multiple active bank failures along a 125 m long section of riverbank with headscarps that had propagated to the toe of the dike. The slope failures had compromised the integrity of the dike and threatened the town with the risk of flooding.

M.I. conducted detailed geotechnical investigations to better quantify the groundwater conditions and engineering properties of the soil. Concurrently, M.I. conducted a test program to investigate whether deep rockfill columns were feasible to construct as a potential stabilization method.

2.1 Geotechnical Investigations

Detailed investigations were conducted in stages between 2007 and 2014. This included a drilling and sampling program, laboratory testing, electric piezocone testing (CPTu), and installation of slope inclinometers (S.I.), piezometers and monitoring wells. A site plan is shown in Figure 2 and a general cross section is shown in Figure 3.

Based on the geotechnical investigations, the stratigraphy was identified as consisting of approximately 30 m of high plasticity clay of glaciolacustrine origin overlying granular till. The general stratigraphy of the site is shown in Figure 3.

Monitoring of the piezometers and groundwater monitoring wells identified that artesian groundwater conditions were present at the site. Groundwater levels measured in the till aquifer were as high as an elevation of 236 m. Groundwater levels measured in August, 2014 are shown on Figure 3.

Six (6) slope inclinometers (S.I.) were installed during the 2007/2008 drilling program. By 2012, each of the S.I. casings had deformed beyond their functional limits. The approximate slide plane is shown in Figure 3 and labelled SS1.

2.2 Test Rockfill Column Program

Several bank stabilization techniques were considered in the early design stages including a rockfill toe berm, bank regrading, and rockfill columns. Locally, rockfill columns are typically extended through lacustrine clay and into the dense underlying till. The clay is deep at this site and beyond the depth to which rockfill columns have successfully been installed in Manitoba. Therefore, M.I. conducted a test program in 2010 to determine the feasibility and challenges associated with installing deep columns.

Two (2) test rockfill columns were successfully installed on the river side of the dike at prairie level where the depth to till was 33.5 to 36.5 m, below ground surface. The diameters of the caissons were 1.6 m and 1.8 m, respectively, and were sleeved with a steel casing to prevent collapse of the open drilled shaft. The shaft was backfilled with crushed limestone which is typical for the Winnipeg region (Thiessen et al., 2011; Abdul Razaq, 2007). The backfill was then compacted using a Vibrolance®. Based on the findings of this test program, it was deemed feasible to construct large diameter columns to the required depth.

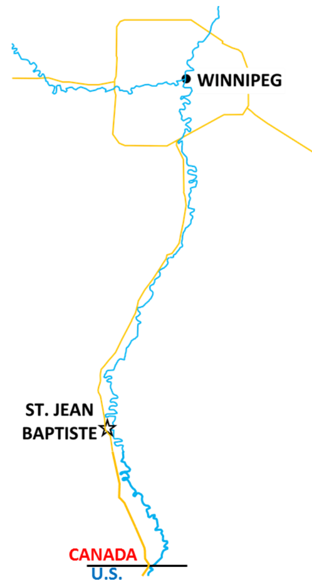

Figure 1: Location of project site relative to Winnipeg, MB.

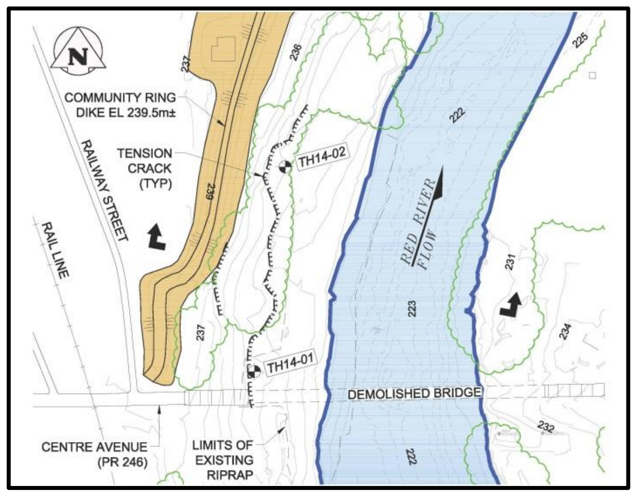

Figure 2: Topographic site plan prior to stabilization works.

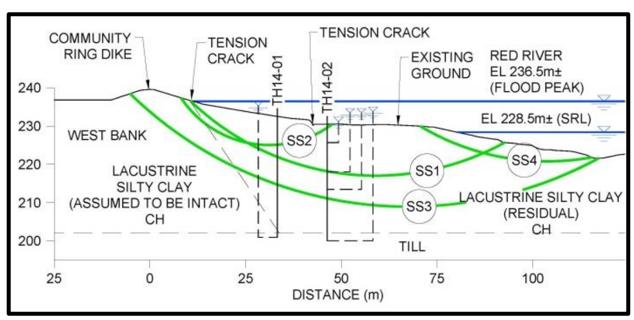

Figure 3: Section A – General cross section.

3 DESIGN

The final design of riverbank stabilization works included riprap erosion protection, toe berms, rockfill columns and rockfill drainage channels. A site plan of the works is shown in Figure 4 and a general cross section of the works is shown in Figure 5.

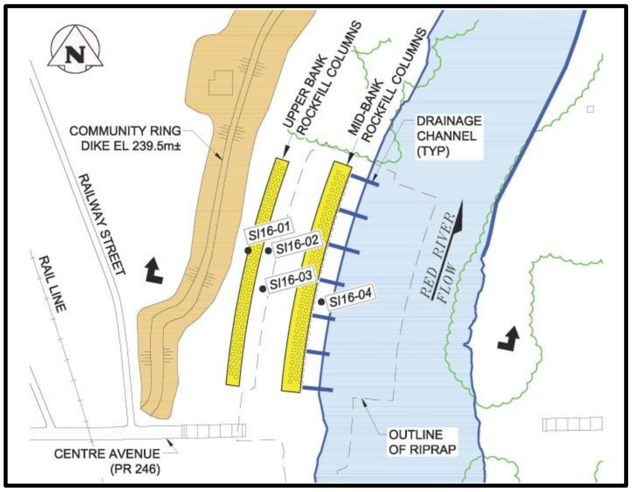

Figure 4: Plan view of Completed Stabilization Works.

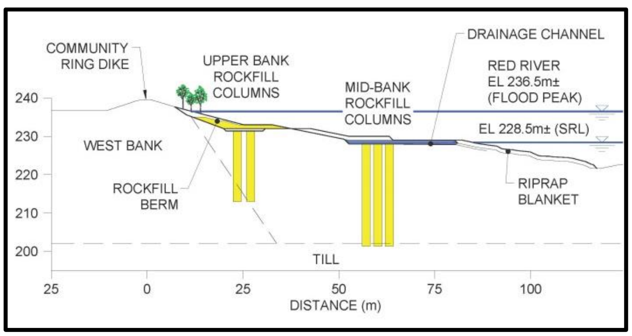

Figure 5: Section showing Completed Stabilization Works.

The 1.5 m thick riprap blanket along the toe of the bank consisted of 450 mm crushed clean limestone. The riprap was installed to prevent erosion along the toe of the bank and to resist slope movement by buttressing and increasing the effective stress on the toe of the slope.

A total of one hundred four (104) rockfill columns were installed along the mid-elevation of the bank, each 2.1 m in diameter and excavated through the lacustrine clay into the underlying till. The depths of columns varied but were generally approximately 26 m deep. These rockfill columns increased the stability of potential deep seated slide planes extending to the base of the clay and till interface.

A series of seventy (70) rockfill columns were also installed along the upper bank and downslope of the ring dike. These columns were drilled to an elevation of approximately 213.0 m, ending in the clay. These columns were designed to increase the resistance against potential shallow failures in the upper bank.

Also included in the design was a series of passive drainage channels and blankets. A blanket of crushed rockfill was installed over the rockfill columns. Channels of rockfill connect the mid bank blanket to the riprap at the toe of the bank to allow the artesian groundwater from the till to drain up through the columns and towards the river.

3.1 Limit Equilibrium Slope Stability Analyses

M.I. specified that the factor of safety (FS) of the slope be increased by a minimum of 30%. This increase was deemed acceptable because of the high level of geotechnical investigation and monitoring data that preceded final design. Several slip surfaces were analyzed including: 1) a global slip surface originating at the observed headscarp down slope of the dike and terminating at the toe of the slope, 2) an upper bank slip surface originating near the dike and terminating mid slope, 3) a potential global slip surface originating upslope of the dike and exiting at the toe of the slope and 4) a lower bank slip surface originating in the mid bank and terminating at the toe of slope. These slide planes are labelled SS1 through SS4, respectively as shown on Figure 3.

A limit equilibrium analysis was conducted to determine the optimum size and location to install the riprap blanket and rockfill columns to meet the design stabilization criteria. The method of vertical slices was used with the Morgenstern-Price method with a half sine interslice force function. The analysis was conducted using Slope/W (Geo-Slope, 2013). The effective stresses used in the stability analysis were imported from a coupled finite element model used to estimate the stress state and porewater pressures. The coupled stress-strain analysis was performed using Sigma/W (Geo-Slope, 2013) and Seep/W (Geo-Slope, 2013) and is discussed in detail in Section 3.2.

The material properties for each region are shown in Table 1. The material parameters for the lacustrine clay are based laboratory testing on Lake Agassiz clay by Freeman and Sutherland (1974) and site specific laboratory testing conducted by M.I. The clay upslope of any observed slope failures was assumed to be reasonably defined by post-peak strength. While the clay downslope of observed slope failures was assumed to be defined by residual strength. The friction angle of crushed limestone locally used for rockfill columns has been measured to be up to 64° by Abdul Razaq (2007) from large scale direct shear tests. A more conservative value of 50° was used in design. However, the effect of dilation as observed by Abdul Razaq (2007) was considered when selecting a value greater than 45°.

The estimated FS for the pre-construction conditions versus the final design are shown in Table 2. The estimated FS is increased by more than 30% for all analyzed slip surfaces and the FS is greater than 1.0 for each slip surface. These results correspond to the effective stress state and porewater pressure with the river elevation at approximate winter river level of 225.5 m, one of the lowest winter water levels on record as observed by M.I.. The artesian groundwater conditions in the till aquifer were considered by applying a total water head boundary condition of elevation 236.0 m to the clay and till interface.

Table 1: Material properties for limit equilibrium slope stability analyses.

Material	Unit Weight (kN/m^3)	c' (kPa)	ϕ'
Lacustrine Clay (Post-peak)	17.5	10	17
Lacustrine Clay (Residual)	17.5	2	9.9
Rockfill Columns	20	0	50
Riprap	18.5	0	35
Till	20	0	30

Table 2: Estimated factor of safety before and after construction of stabilization works.

Slip Surface	Factor of Safety		% Change
	Pre-Construction	Post-Construction	
SS1	<1 (0.79)	1.26	59
SS2	<1 (0.92)	1.22	33
SS3	<1 (0.80)	1.24	55
SS4	<1 (0.73)	1.09	49

3.2 Finite Element Deformation Analyses

Finite element analyses were conducted to predict the stress-strain behaviour of the slope after remediation. This analysis was carried out to estimate the deflection of rock columns as flexible members in response to changing effective stress conditions. The deformation modeling was conducted using Sigma/W (Geo-Slope, 2013). The model was coupled with a groundwater flow analysis using Seep/W (Geo-Slope, 2013) to calculate the porewater pressure at each element. The materials in the Sigma/W model were assumed to be elastic-perfectly plastic. The analysis conducted is similar to the method described by Thiessen (2010) to model the performance of a loaded riverbank with rockfill columns in Winnipeg.

The yield criterion used in the finite element model was governed by Mohr-Coulomb parameters. The shear strength parameters were consistent with the analysis conducted in the L.E. stability analysis shown in Table 1. A sensitivity analysis was conducted on the stiffness of the clay with Young's modulus of 10, 20 and 40 MPa. The rockfill columns were modelled with a hyperbolic stiffness function by Thiessen (2010) adopted from Duncan and Chang (1970). A sensitivity analysis was similarly conducted on varying stiffness functions for the columns based on different densities of rockfill.

The displacement in the columns was estimated based on the effective stress changes induced by two river drawdown scenarios: 1) from summer river level (EL. 228.5 m) down to the winter river level (EL. 225.8 m) and 2) from the high (flood) river level (EL. 236.5 m) to the regular summer river level. Both scenarios were modelled as a rapid drawdown.

A plot of the estimated displacement of a rockfill column at the mid-bank is shown in Figure 6 for drawdown Scenario 1. Figure 6 shows the deflection for varying stiffness of clay. The estimated displacement is similarly shown for drawdown Scenario 2 in Figure 7.

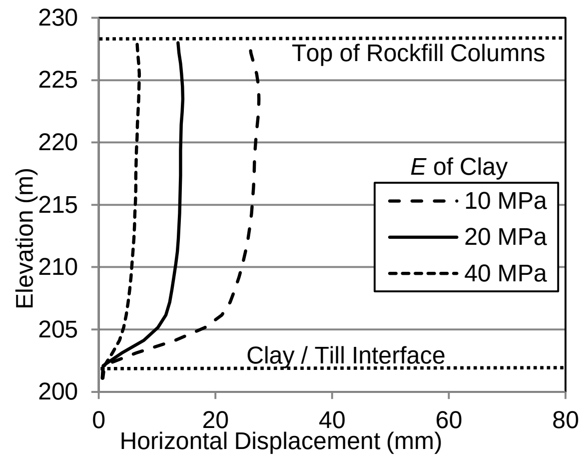

Figure 6: Estimated horizontal displacement of rockfill columns at mid-bank for river drawdown scenario 1 (from summer river level down to the winter river level) and varying Young's Modulus of clay.

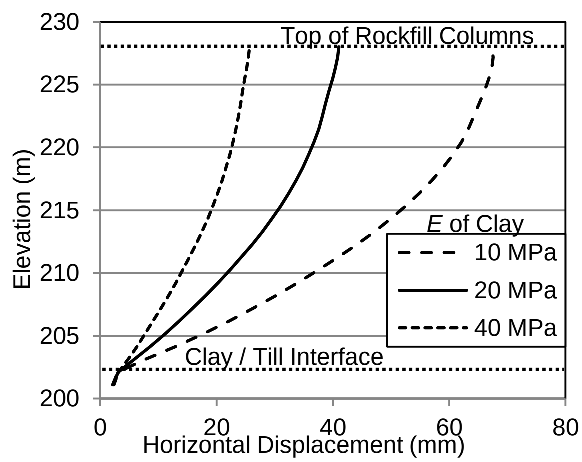

Figure 7: Estimated horizontal displacement of rockfill columns at mid-bank for river drawdown scenario 2 (from the high (flood) river level to the regular summer river level) and varying Young's Modulus of clay.

In both drawdown scenarios, the estimated horizontal displacements are significantly more sensitive to the stiffness of the clay than to the stiffness of the rockfill. Results from the sensitivity analysis on rockfill stiffness are not shown as the impact on horizontal displacements was minimal. This is logical when considering that the clay occupies the majority of space and only a small proportion is replaced with rockfill. For the summer to winter river level drawdown, the stress change occurs at the toe of the slope resulting in a predominantly horizontal stress change. Therefore, most of the shear deformation occurs at the base of the column near the till. On the other hand, the magnitude of horizontal displacement near the surface is greater for the condition of flood to summer river level drawdown. The change in stress conditions is greater in the second scenario with the larger drawdown in river elevation.

4 CONSTRUCTION

Construction of the riverbank stabilization works commenced in January, 2015 and was completed in May, 2016. The construction sequencing was carefully planned to avoid triggering excessive riverbank movement. The fluctuating river levels and its effect on bank stability were also considered in planning the construction sequence. A summary of the construction activities is provided, including a description of construction methodologies, timeline, and challenges that were overcome.

The 1.5 m thick riprap blanket was placed first in January, 2015. This riprap extended from the river's edge to the channel thalweg. This work was completed in the winter when the river levels are relatively low. Besides providing erosion protection, the riprap buttressed the toe of the slope to increase stability.

Construction of the mid-bank rockfill columns began in February, 2015. A bench was cut along the midbank and surfaced with a rockfill travelling surface. The columns were constructed by first free drilling until sloughing occurred in the upper alluvial material at which point a 3.0 m diameter casing was advanced to approximately 10 m below surface. Drilling continued to refusal and then a 2.1 m diameter inner casing was advanced (as shown in Figure 8) into the underlying till. Any excess clay remaining in the inner casing was removed prior to backfilling with 150 mm diameter clean crushed limestone. To ensure the inner casing could be removed, approximately 3 m of limestone was placed at the bottom of the hole. The inner casing was then lifted 1.5 m and the backfilling continued until it was approximately 1.5 m above the top of the inner casing. After the inner casing was retrieved, the 3.0 m shaft was backfilled prior to retrieving the larger casing. The casings were advanced and retrieved using a vibratory hammer hoisted by crane.

Rising levels of the Red River presented scheduling challenges. On March 17, 2015, construction of rockfill columns halted as the river flood level encroached onto the mid bank. Construction resumed on April 15 as the river levels receded. Work was halted once more on May 18 as the Red River elevation raised again. Due to the unusual weather with the river cresting twice, only seventy-five (75) of the one hundred four (104) columns were completed by early summer 2015.

The contractor returned to site in August, 2015 after the high river levels receded. Evidence of ongoing riverbank movements were observed following the rise and fall of the river levels. Five (5) steel casings that were left in the slope had collapsed due to continued movement of the slope after the natural drawdown of the river levels. Three (3) sleeves were damaged beyond repair. The damaged sections of two (2) sleeves were cut and the sections that remained intact were welded together to form a new sleeve. The remaining mid bank columns were completed by August 26. An aerial view of the site from August, 2015 is shown in Figure 9.

Figure 8: Advancing steel casing for rockfill columns at mid-bank.

Figure 9: Installation of rockfill columns along mid-bank in August, 2015.

Each of the rockfill columns were then further compacted using a Vibrolance®. This has been the preferred technique locally for compacting rockfill for columns and has been measured to increase rockfill dry density up to densities as high as 22 kN/m^3 (Skaftfeld, 2014). The one hundred four (104) mid bank columns were compacted from grade using a Vibrolance® from September 10 to 14, 2015. Vibrolance® compaction was in addition to using the vibratory hammer during retrieval of the steel sleeving. Both the rate of retrieval and frequency of the vibratory hammer were optimized to achieve a minimum dry density of 20 kN/m^3.

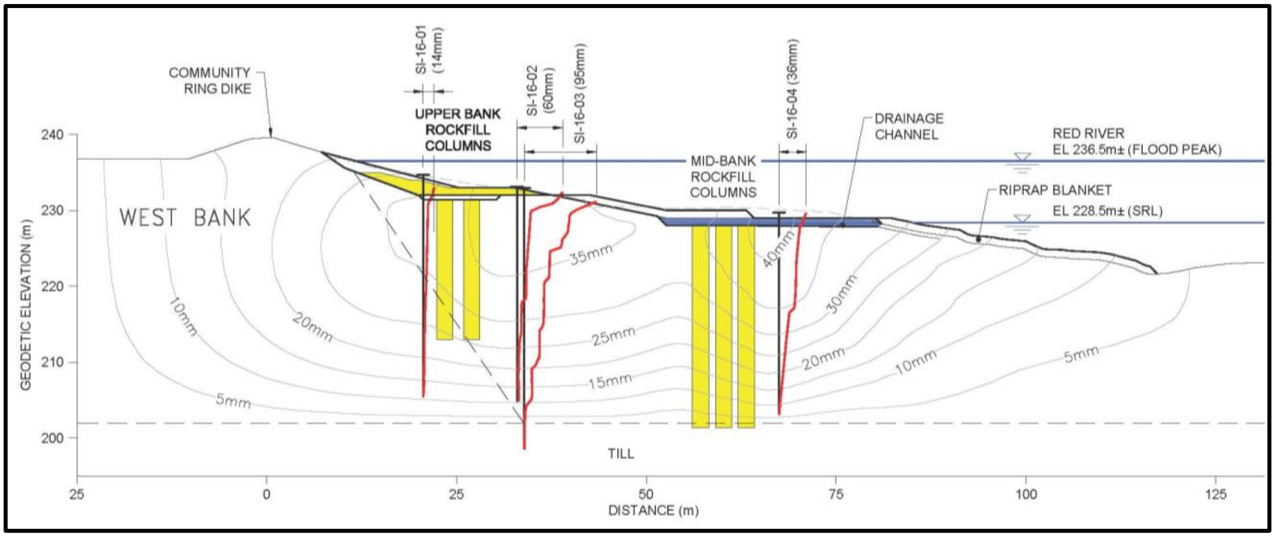

Figure 10: Slope inclinometer monitoring data and estimated horizontal displacements from finite element model.

In September 2015, drainage channel trenches were cut and backfilled with rockfill to connect a granular pad over the columns to the riprap at the shoreline. The mid bank work area and trenches were finally capped with compacted clay and graded towards the river.

Installation of the rockfill columns along the upper bank below the dike began in January, 2016. The upper bank columns were excavated to a depth of approximately 19 m below the work bench. These shorter columns were cased with a single 2.1 m diameter steel sleeve. The shallower depths allowed for much quicker installation. The seventy (70) rockfill columns were excavated, backfilled and compacted with only the Vibrolance® in approximately one month.

In May, 2016, the upper bank columns were capped with compacted clay. The slope was regraded to eliminate the over steepened slope cut for the access and work pad below the dike. The slope was backfilled with rockfill to improve drainage, lower the groundwater table, and increase stability in the upper bank in response to the observed movement from February 2016. The surface of the slope was capped with compacted clay. Finally, the slope was revegetated with natural grasses and planted trees to both naturalize the riparian reach and provide some benefit to bank stability at shallower depths because of the root networks.

5 PERFORMANCE

To date, there have been no visual signs of continued bank movement. To monitor the performance of the slope, four (4) slope inclinometers were installed by M.I. The location of the slope inclinometers are shown in plan on Figure 4 and in section on Figure 10. The base line readings were measured in February and March of 2016 after all rockfill columns had been installed. Only SI16-03 was installed into the till. Therefore, the other instruments provide less confident data.

SI 16-01 and SI 16-02 were installed in response to the upper bank movement observed in February 2016 to determine the approximate elevation of the suspected shallow shear plane. In March 2016, SI 16-03 and SI 16-04 were installed to establish a performance monitoring program for the stabilization project.

SI16-01 was installed upslope of the upper columns and was monitored from February 22, 2016 to March 11, 2016. Unfortunately the casing was damaged by construction equipment less than a month after baseline readings. The data from this time period showed no signs of distinct shear displacement, but did indicate a general straining towards the river.

SI16-02 is located downslope of the upper slope rockfill columns. A baseline reading was collected on February 22, 2016 and the most recent reading was collected on October 30, 2017. The same general displacement of the slope towards the river can be observed from the monitoring data. Distinct shear displacement can be observed at elevation 218 m which first occurred between April 28, 2016 and June 2, 2016. As of the most recent reading, the total cumulative shear displacement at this plane is approximately 5 mm.

SI16-03 is also located downslope of the upper slope rockfill columns. Here, the data indicates that the slope is generally straining progressively towards the river with time. Distinct shear zones are identified at elevations of 225.0 m, 221.0 m, 214.5 m, 209.5 m and 205.0 m. The cumulative shear displacement at each of these slide planes over 19 months of monitoring is approximately 13 mm, 5 mm, 3 mm, 5 mm and 7 mm, respectively. One possible explanation for the shear displacement along these five planes is that these could be pre-existing shear planes with clay at residual shear strength due to the historical movement at this site. The magnitudes of displacements post-construction have been relatively small and are not a concern at this point in time.

SI16-04 is located along the toe of the slope downslope of SI16-03. A baseline reading was collected on April 28, 2016 and the most recent reading was collected on October 30, 2017. The intention was to install another SI to till on the lower riverbank. However due to locking augers this was not possible. Consequently, the casing for SI 16-04 has been subject to interactions with debris and ice. To date there has been no distinct shear movement observed at this location. The general trend of progressive straining towards the river was similarly observed.

Cumulative displacements from the S.I. monitoring data are shown in cross section in Figure 10. The results overlie contours of the estimated horizontal displacement contours from the finite element analysis. The displacement contours assume river drawdown from high level to summer river level and Young's modulus of 20 MPa for the clay. The quantities are not directly comparable since the S.I. monitoring data is cumulative whereas the model provides estimates for a specific drawdown event. Figure 10 does however provide an initial comparison of the observed deflection from S.I. data versus the estimated behaviour from the modelled drawdown event.

6 CONCLUSION

Stabilization of the riverbank in St. Jean Baptiste presented several unique slope stability design challenges. The depth of the clay deposit required deeper rockfill columns than had ever been constructed in Manitoba. Also, artesian groundwater levels were present at the site. The final design included deep rockfill columns to mechanically stabilize the bank. Drainage channels were also constructed near the ground surface to control the groundwater and decrease groundwater levels. These stabilization works were designed to increase slope stability by 30%. Since the rockfill columns are of a considerable length, finite element modelling was performed to model the stress-strain behaviour of columns as flexible members. The deformations were estimated based on the stress changes induced by two river rapid drawdown scenarios: 1) from summer river level down to the winter river level and 2) from the high (flood) river level to the regular summer river level.

Based on slope inclinometer monitoring data collected at the site post-construction, the observed behaviour of the rock columns have been generally consistent with the estimated behaviour of the finite element stress-strain modelling. However, the model estimated displacements for a worst-case scenario of a rapid river drawdown which has not yet been experienced at the site. Consequently, the monitored displacement has been less than the model predicted. Additionally, the S.I. monitoring data includes several shear planes that cannot be captured in the finite element model since the clay was modelled as a continuum. The observed shear displacement is not a concern because it is required to mobilize the shear resistance of the rockfill and it is in the estimated range of the model.

Based on the experience gained from this project, there are some lessons to be learned in estimating deformations of a slope. For a more reasonable estimate of displacements, a more realistic river drawdown rate and magnitude could have been modelled. Although this drawdown rate and magnitude cannot be predicted, the rapid drawdown scenario that was modelled provides worst-case scenario displacements that are less likely to be observed in reality. By modelling the lacustrine clay as a continuum, it is not possible to capture the shear displacement that was observed in actual post-construction monitoring data. If the site had a well-defined pre-existing slide plane, it may have been reasonable to model a thin material region with different properties. At this site however, the lacustrine clay bank had a history of slope instability in which numerous shear planes would have developed in the past. This results in numerous planes, both shallow and at depth, of weaker clay at residual strength.

Overall, the site has performed as anticipated in design in regards to post-construction movement and protecting the community ring dike. The contractors were able to develop methodologies to successfully install and compact the deep rockfill columns. Accurately estimating deflections of slopes is still a challenge but this case study shows that finite element modelling can provide reasonable qualitative estimates by treating rock columns as flexible members.

7 REFERENCES

Abdul Razaq, W. 2007. *Evaluation of riverbank stabilization using rockfill and soil-cement columns* (Ph.D. thesis), University of Manitoba, Winnipeg, Manitoba.

Cruden, D.M. and Varnes, D.J. 1997. Landslide Types and Processes, in; Turner A.K. and Schuster, R.L. (editors), *Landslides: Investigation and Mitigation:* Transportation Research Board, National Research Council, Washington, DC, USA, Special Report 247, 36-75.

Duncan, J.M. and Chang, C. 1970. Nonlinear analysis of stress and strain in soils, *Journal of Soil Mechanics and Foundations Division,* 96(5): 1629-1653.

Freeman, W.S. and Sutherland, H.B. (1974). Slope Stability Analysis in Anisotropic Winnipeg Clay. *Canadian Geotechnical Journal,* 11(1): 59-71.

Geo-Slope International Ltd. 2013. GeoStudio 2012 [Computer software], v. 8.12.3.7901.

James, A. 2009. *Development of a Riverbank Asset Management System for the City of Winnipeg* (M.Sc. thesis), University of Manitoba, Winnipeg, Manitoba.

Penner, F. 2007. *St. Jean Dykes: Report on River Bank Movements* (Unpublished report), Winnipeg, Manitoba.

Skaftfeld, K. 2014. *Experience as a guide to geotechnical practice in Winnipeg* (M.Sc. thesis), University of Manitoba, Winnipeg, Manitoba.

Thiessen, K. 2010. *Stabilization of natural clay riverbanks with rockfill* (Ph.D. thesis), University of Manitoba, Winnipeg, Manitoba.

Thiessen, K.J., Alfaro, M.C. and Blatz, J.A. 2011. Measuring the Load-Deformation Response of Rockfill Columns by a Full-Scale Field Test on a Natural Riverbank. *Canadian Geotechnical Journal,* 48(7): 1032-1043.

Tutkaluk, J.M. 2000. *The Effect of Seasonal Variations in the Red River and Upper Carbonate Aquifer on Riverbank Stability in Winnipeg* (M.Sc. thesis), University of Manitoba, Winnipeg, Manitoba.