Near Collapse of the St. Adolphe Bridge - An Exercise in Emergency Geotechnical Engineering

R.M. Kenyon, Ph.D., P.Eng, FEIC, B. P. Arpin, P.Eng.
KGS Group, Winnipeg, Manitoba, Canada
R.J. Eden, M.Sc., P.Eng.
Manitoba Infrastructure and Transportation, Winnipeg, Manitoba, Canada

ABSTRACT
In late August of 2009, routine bridge inspection by Manitoba Infrastructure (MI) of the St. Adolphe Bridge crossing of the Red River revealed that Pier SU 3 on the west bank of the river had started to move dramatically. Movement had been metres in magnitude. The emergency objective response was to stabilize the moving/failing riverbank sufficiently to allow for safe demolition of the damaged bridge sections and to avoid any collateral damage to the remainder of the structure. Temporary stabilization included offloading the top of bank plus a large toe berm in the river. Permanent stabilization included a shear key of rockfill columns. This paper highlights all aspects of the emergency engineering response, including the design of first stabilization works, safe demolition of the damaged portion of the bridge structure, followed by permanent bank stabilization for bridge reconstruction.

RESUME
Une inspection du pont de Saint-Adolphe traversant la rivière Rouge effectuée par Infrastructure Manitoba à la fin du mois d'août en 2009 a révélé que le pilier SU 3 de la rive ouest de la rivière s'était déplacé considérablement. L'ampleur du mouvement s'étendait sur des mètres de longueur. L'objectif d'une intervention d'urgence consistait à stabiliser suffisamment la berge mouvante / défaillante afin d'assurer la démolition sécuritaire des parties endommagées du pont tout en protégeant le restant de la structure. La stabilisation temporaire de la berge comprenait le déchargement du sommet de la berge et l'ajout d'une risberme dans la rivière. La stabilisation permanente comprenait une clé de cisaillement de colonnes d'enrochement. Cet article met en évidence tous les aspects de l'intervention d'urgence d'ingénierie, y compris la conception des premiers travaux de stabilisation, la démolition sécurisée de la partie endommagée du pont ainsi que la stabilisation permanente des berges pour permettre la reconstruction du pont.

1 INTRODUCTION

Riverbank failures along the Red River are typically found on the outside bends where the bank is subject to erosion and fluctuating river levels (James, 2009, Tutkaluk et al., 1998). At other times failure may be due to high artesian conditions in the bedrock aquifer (Friesen et al., 2012, Arpin et al., 2016).

The St. Adolphe Bridge (also known as the Pierre Delorme Bridge) is a seven span bridge crossing the Red River and connecting the community with PTH 75, the main north-south connection between Manitoba and the United States. The town is located in the middle of the flooded plain or "Red Sea" which forms during major Red River flood events. The bridge is also the last emergency access route during extreme flooding.

The 2009 spring flood on the Red River was second only to the 1997 "Flood of the Century" in magnitude and in impact on bridges and dikes and other infrastructure along the river. Spring flooding was followed by sustained summer flooding resulting in saturation of riverbanks and failures following summer flood recession. The Pierre Delorme bridge failure occurred at this time. Other failures included the St. Jean Baptiste Dike (Bartz et al., 2016).

This paper describes the failure itself, the emergency geotechnical response and emergency design that stabilized the bank sufficiently to allow safe demolition of the impacted elements of the bridge and which prevented further loss of the structure. The paper concludes with design details of permanent stabilization of the bank to allow bridge reconstruction.

2 BACKGROUND

St. Adolphe is located approximately 15 km south of Winnipeg on the east side of the Red River, located as shown on Figure 1. Failure was on the west or outside bend of the river opposite the town. Figure 2 presents a satellite image of the bridge crossing prior to failure

The bridge, constructed in 1974, consists of seven post-tensioned pre-cast I-shaped girder sections supported by six concrete piers and two abutments.

Figure 3 presents a 1974 stratigraphic section of the bridge. Overburden stratigraphy consists of some 15 to 17 m of soft and compressible high plasticity clay on the west or outside bend with the same depth of alluvial flood plain deposits of intermediate plasticity on the east or inside bend of the river. The clay is underlain typically by a thin soft layer of ablation till followed by competent dense basal tills and limestone bedrock.

As shown on Figure 3 the bridge abutments and piers are supported on a variety of foundation types including driven steel H piles, driven timber piles, driven precast pre-stressed hexagonal concrete piles, and shallow footings.

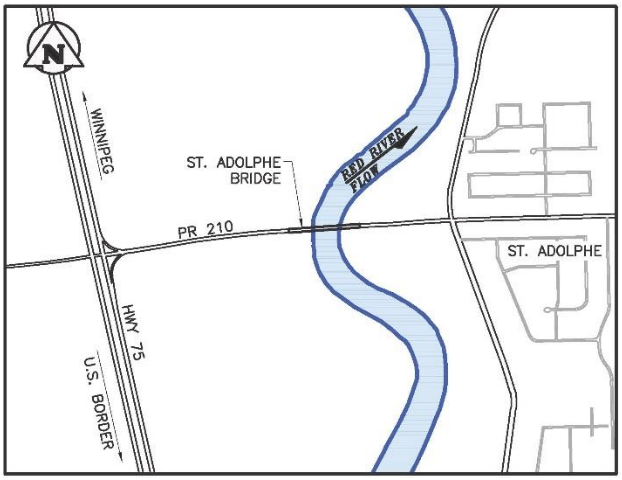

Figure 1. Location of St. Adolphe and Bridge.

Figure 2. Bridge crossing before failure.

The material properties of the Lake Agassiz clays have been studied extensively with much of the research based upon the Red River Floodway investigations (Freeman and Sutherland, 1974, Baracos et al., 1980, Skaftfeld et al., 2009). Typically the clay possesses a plasticity index, PI, in the order of 60, a normally consolidated or fully softened large strain shear strength of 14° to 17°, with a residual strength ranging from 8° to 12°. The clay tends to be weathered, and over-consolidated for the upper five to six metres, becoming much softer with depth due to the artesian conditions in the basal bedrock aquifer. An excellent overview of geological and hydrogeological conditions is found in Kjartanson (1983).

3 EMERGENCY RESPONSE

On August 20, 2009, following drawdown of the summer flood event, Manitoba Infrastructure (MI) noted that severe displacements and rotation of Pier SU 3 had begun. Pier SU 3 is located on the lower west bank of the river. Figure 4 is a profile view of the bridge deck upon first site inspection by the authors. Figure 5 shows the rotation and settlement of the affected pier SU 3. Figure 6 presents the surveyed movement of that pier during the early stages of the emergency.

MI immediately initiated a survey monitoring program of all piers and concluded that Pier SU 3 was settling and translating at a rate of 0.8 to 1.0 m per day.

Figure 4. Bridge deck profile upon first inspection.

Figure 5. Pier SU 3 upon first inspection.

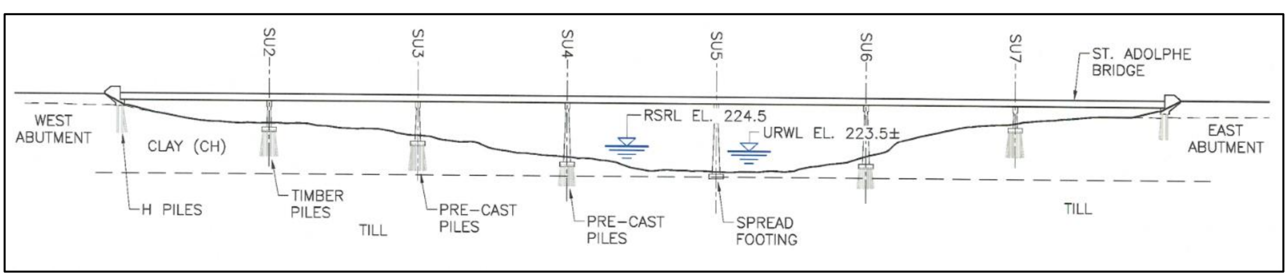

Figure 3. Stratigraphic Section with Pier Foundations.

At the same time, MI engaged a General contractor (PCL), an Earthworks contractor (HMC), a Demolition contractor (Rakowski), the original bridge designer (Dillon), and KGS as geotechnical engineers. KGS retained Blatz Engineering for modeling assistance.

The first emergency geotechnical inspection and meeting between bridge engineers, geotechnical engineers, and MI occurred late on August 21, 2009. During that meeting, the bridge and bank stability conditions were as shown on Figures 4, 5, and 7. The only geotechnical information available at the inspection was the stratigraphic profile of the bridge shown in Figure 3.

Several conclusions and concerns were identified by the structural, geotechnical, and owner's engineers during this first inspection.

First of all, Pier SU 3 was failing by settling and rotating actively and visibly at the time. Movements were ongoing and active, and you could hear the bridge structure responding audibly. A well-defined and localized head-scarp, approximately 1 m high, was noted immediately upstream of Pier SU 3. This accounted for the pier failure. Figure 8 illustrates the lower bank failure as it was assumed to be that first night. It was concluded immediately that the precast pre-stressed concrete piles had been sheared off by the movements and that the pier was probably resting and settling on its pile cap.

Secondly, the entire riverbank, from river's edge to top of bank presented multiple headscarps and a hummocky appearance. It was concluded that any attempt to access the bridge from the riverbank on either side would very likely initiate further bank movements and this might initiate catastrophic loss of more elements of the bridge structure. Therefore the geotechnical objective was to develop an emergency stabilization plan that would improve stability to the point where equipment could safely access the bridge structure to complete demolition.

The bridge girders were precast and post-tensioned and the structural concern was that the girders might actually explode if they were to fall. And, the girders were no longer on the bearing pads at Pier SU 3. MI decided, from a safety perspective, that no person or equipment would be allowed within 15 m of the bridge until the bank had been sufficiently stabilized.

That first evening a full time survey and video monitoring program of all bridge piers was implemented on a continuous basis. The purpose was first to provide any early warning of any accelerated movement, and secondly to provide a video record of all piers should Pier SU 3 collapse catastrophically. Here the concern was that any collapse mechanism and its resultant dynamic loading on the riverbank would result in further bank movements and possibly movement of other piers and spans adjacent to the failure itself. The emergency team disbursed late that first evening, agreeing to meet again the next morning, once the geotechnical team had developed an emergency stabilization response plan.

3.1 Geotechnical Emergency Response Plan

The objectives of the emergency response plan were first to improve overall or global stability by approximately 20% so that equipment and materials could start to access that upper riverbank area safely. The second objective was to improve lower toe stability by +30% for the conditions of demolition crane and truck loading on the bank, plus dynamic loading of the bridge girders as they fell and struck the bank, assuming a pore pressure response to loading, B-Bar = 1.

The elements of emergency stabilization design began with offloading of the crest of the bank to improve global stability, followed by construction of a large rockfill toe berm in the river to provide the necessary lower toe and midbank

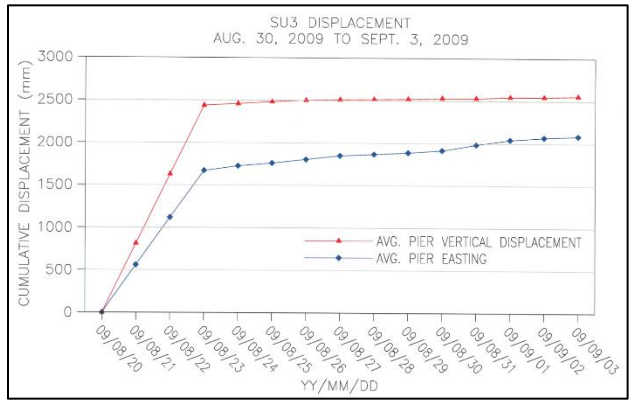

Figure 6. Survey movements of Pier SU 3.

Figure 7. Typical pervasive and retrogressive bank failure conditions.

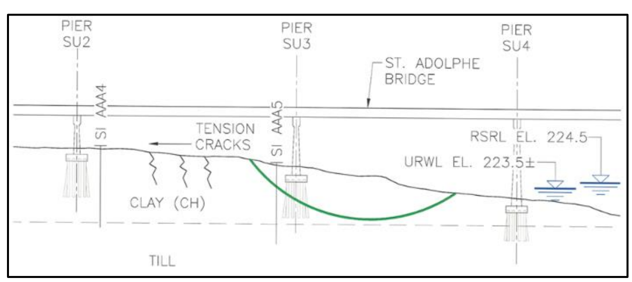

Figure 8. Failure conditions as assumed upon first review.

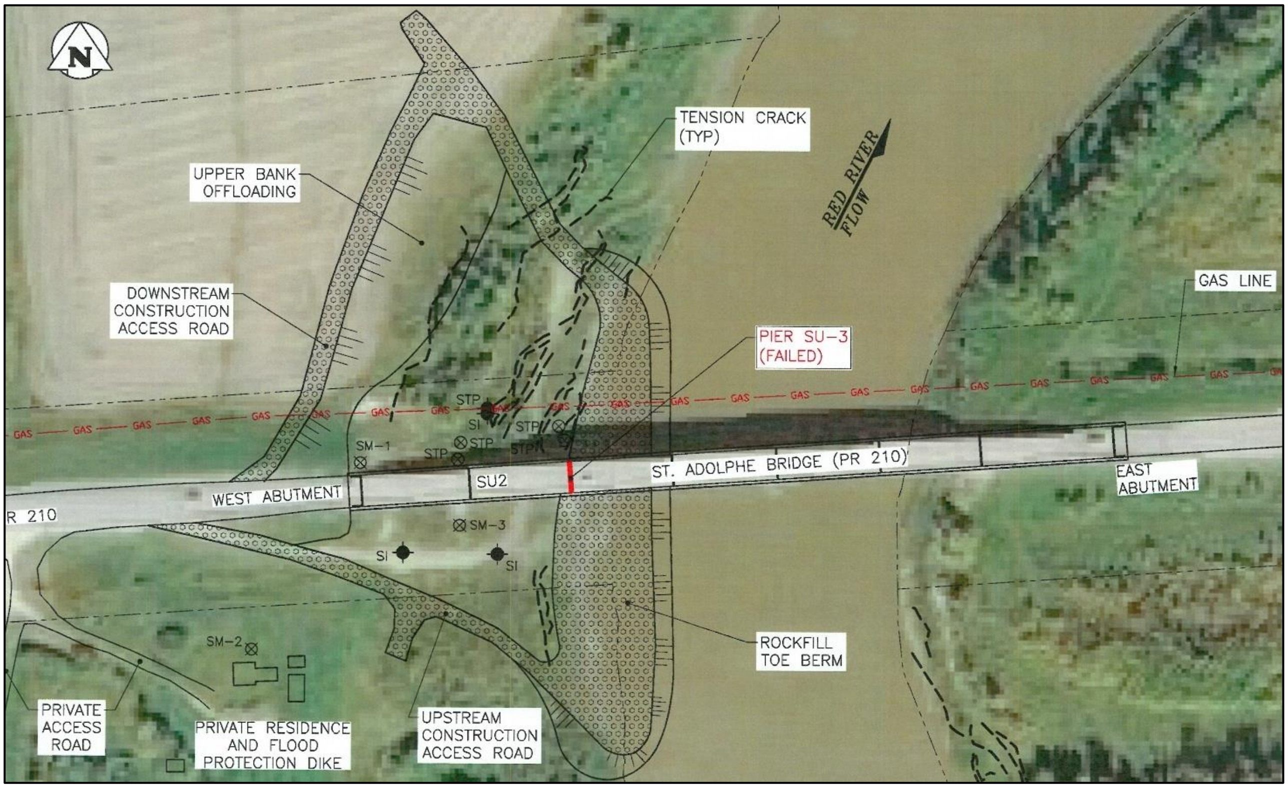

Figure 9. Plan of emergency stabilization works.

stability which would allow for demolition of Spans 02 and 03 as well as Pier SU 3. The plan as it developed that overnight and as it was later drafted upon paper is shown on Figure 9.

3.2 Emergency Response Stability Modeling

Based upon the experience of modeling hundreds of similar riverbanks, it was concluded that each metre of vertical offloading of the top of bank would result in an approximately 10% improvement to global stability. The first decision was that the top of bank would be offloaded by two metres vertically, with the bench width extending back a minimum 20 m from the top of bank and 70 m downstream of the bridge. The upstream riverbank had been offloaded previously to provide borrow for ring dike construction such that it did not require the same level of offloading.

Based upon detailed stability modeling for the Red River Floodway Channel (Skaftfeld et al., 2009) it was concluded that a 20 m wide bench would allow for equipment access to run along the top of bank and not induce any additional stresses upon the bank provided the equipment remained a minimum 10 to 12 m back from the crest. The length of 70 m downstream of the bridge was an arbitrary decision which represented approximately twice the slope length of the riverbank itself. These elements and decisions were never analyzed or modelled further because the decisions as to how to effectively stabilize the lower toe of the riverbank were more complex and would require more time.

Typically the concern with toe berm or riprap blanket construction on high plasticity clays is that initial loading will induce a B-Bar response of one which in itself might cause further movements. This is the scenario whenever the river bottom stratigraphy is dominated by clays as is the case further south in the valley.

However, it was noted that first overnight that the centre pier was founded on a shallow spread footing which strongly suggested that the river bottom was controlled by dense basal tills or perhaps limestone bedrock. The conclusion was therefore that a large rockfill toe berm could be constructed in the river channel without undue concern regarding the pore pressure response to loading. What remained therefore was the detailed designed of the geometry of the berm.

Emergency stability modeling followed the Limit Equilibrium method using the Morgenstern Price method for calculating interslice forces. Pore pressure conditions or pore pressure responses to loading were specified as an equivalent phreatic surface to the applicable soil layer. It was assumed that the bank was saturated at the time of failure and that any change in loading would result in a B-Bar response of 1.0 in the clay.

Back analysis of the failure was then completed to determine approximately what residual shear strengths should be assigned the clay. Table 1 below lists those material properties.

Intact, post-peak shear strengths were assumed for all clays behind the crest of the riverbank. Clays on the bank were assigned residual strengths. The toe berm was to be constructed of crushed rockfill which would then be reused and recycled for permanent shear key stabilization. Its shear strengths were taken from large scale direct shear testing by the University of Manitoba (Razaq, 2007) with 38° being the measured critical state shear strength.

Table 1: Material properties for limit equilibrium slope stability analyses as estimated from back analysis of failure.

Material	Unit Weight (kN/m^3)	C' (kPa)	Φ'
Lacustrine Clay (Post-peak)	17	5	14
Lacustrine Clay (Residual)	17	2	11
Rockfill Berm	15	0	38

Once the stability model was appropriate calibrated, design loadings included the surcharge loading of the demolished spans and pier, assuming a dynamic loading when structures first struck the bank. Dynamic or impact loading was modeled by arbitrarily assigning a dynamic load factor of 1.5 to the equivalent static load. The resulting berm geometry as shown on Figure 11 achieved an overall improvement to lower toe stability of +30% for those demolition conditions. The combination of the 2 m of vertical offloading of the top of bank plus the toe berm provided an equivalent +30% improvement to overall global stability of the bank. This was important because the intent was that the abutment would be saved and reused in the reconstructed bridge structure. It also was critical to maintaining the fibre optic line in service to the community.

3.3 Construction of Emergency Response Measures

All elements of the emergency response plan discussed above were outlined to the emergency response team the following morning. What remained to be confirmed was the exact geometry of the lower toe berm but otherwise the general contractor was able to begin mobilization and construction of stabilization measures. Figure 9 represented that plan.

Sequencing of emergency stabilization works is quite often the most critical element of such a project. The work began with upper bank offloading, starting at the approach embankments and working progressively away from the bridge, upstream and downstream simultaneously. In that way, critical global stability of the bridge structure began to improve with the removal of the first load of embankment fill.

Once offloading was completed, a haul road was completed following from the highway embankment along the back edge of the offloading and then down the riverbank to the start of the lower toe berm construction. Figure 10 shows the construction access and haul road being constructed, set back approximately 20 metres from the top of bank. Note that the two meters of offloading had already been completed between the rockfill access road and the top of bank. The access road on the bank was sub-cut continuously so that loaded trucks would present no net loading to the existing bank.

The toe berm construction was initiated away from the bridge beginning at the upstream and downstream ends, meeting under the bridge. This approach resulted in the berm progressively "pinching off" the lower bank and minimized the risk of movement during berm construction. Figure 11 shows the lower toe berm as it was nearing completion.

With stabilization complete, plans were immediately executed to demolish the two spans and supporting pier.

Demolition began shortly after 6:00 P.M., continued through the night (Figure 12), and was completed early in the next morning as shown on Figure 13.

Figure 10. Construction access haul road, set back 20 m from top of bank and extending 70 m downstream from bridge.

Figure 11. Construction of lower toe berm nearing completion under the bridge.

4 DESIGN OF PERMANENT STABILIZATION MEASURES

Once emergency stabilization and demolition was completed, MI selected a design factor of safety of 1.5 to be achieved for global and any local slip surfaces that could potentially impact any elements of the reconstructed bridge. The method of stabilization selected consisted of a shear key constructed as closely spaced arrays of large diameter rockfill caissons. The caissons are constructed one at a time to minimize the potential for bank movement during installation.

Individual columns were to be backfilled with crushed clean rockfill, as recovered from the emergency toe berm. The rockfill consisted of clean crushed lime which was hard

Figure 12. Demolition during nighttime conditions.

Figure 13. Completion of demolition the following morning.

and durable and whose properties were similar to those used by Razaq (2007), or Thiessen et al. (2011).

Once the rockfill columns were filled and completed, the rockfill was densified using a vibrating lance which has been reported to achieve dry densities as high as 22 kN/m^3 (Skaftfeld, 2014). Here stability modeling assumed that the rockfill would be compacted to a dense configuration such that effective friction angles in the rockfill greater than critical state could be assumed. Table 2 shows the material properties assigned the permanent analysis.

The resulting permanent stabilization measures are shown in section on Figure 14 and in plan on Figure 15.

Permanent stabilization works were completed in the winter of 2010 with the bridge restored and re-opened to public traffic in that fall.

Table 2: Material properties for limit equilibrium slope stability analyses of permanent works.

Material	Unit Weight (kN/m^3)	C' (kPa)	Φ'
Lacustrine Clay (Post-peak)	17	5	14
Lacustrine Clay (Residual)	17	2	11
Rockfill Columns	20	0	45-55
Riprap	18	0	38
Till	20	0	30

5. CONCLUSIONS

Although geotechnical engineering is generally acknowledged to be a risky sub-discipline of engineering, few geotechnical engineers experience the situation where there is an opportunity to save a structure by developing an immediate stabilization plan that can be safely implemented. Sound engineering judgement and experience are a critical component of any such project but emergency decisions must be followed by sound and careful engineering analysis and design. Usually the analysis and detailed design is being performed to provide just in time delivery. In this case, that timing required the collaboration and support of a separate firm to complete the necessary stability modeling in a timely fashion.

All of the emergency stabilization works and all of the demolished bridge materials were recovered and re-used. The toe berm rockfill became the backfill material for the rockfill shear key. The rockfill caisson method of permanent

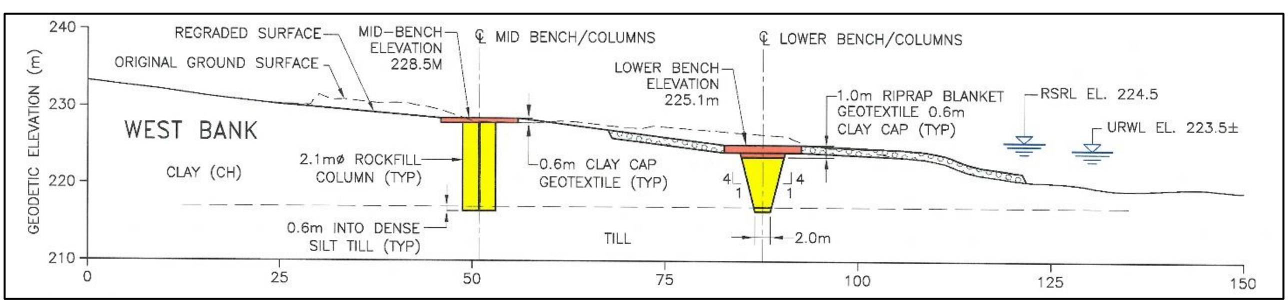

Figure 14. Section of permanent stabilization works.

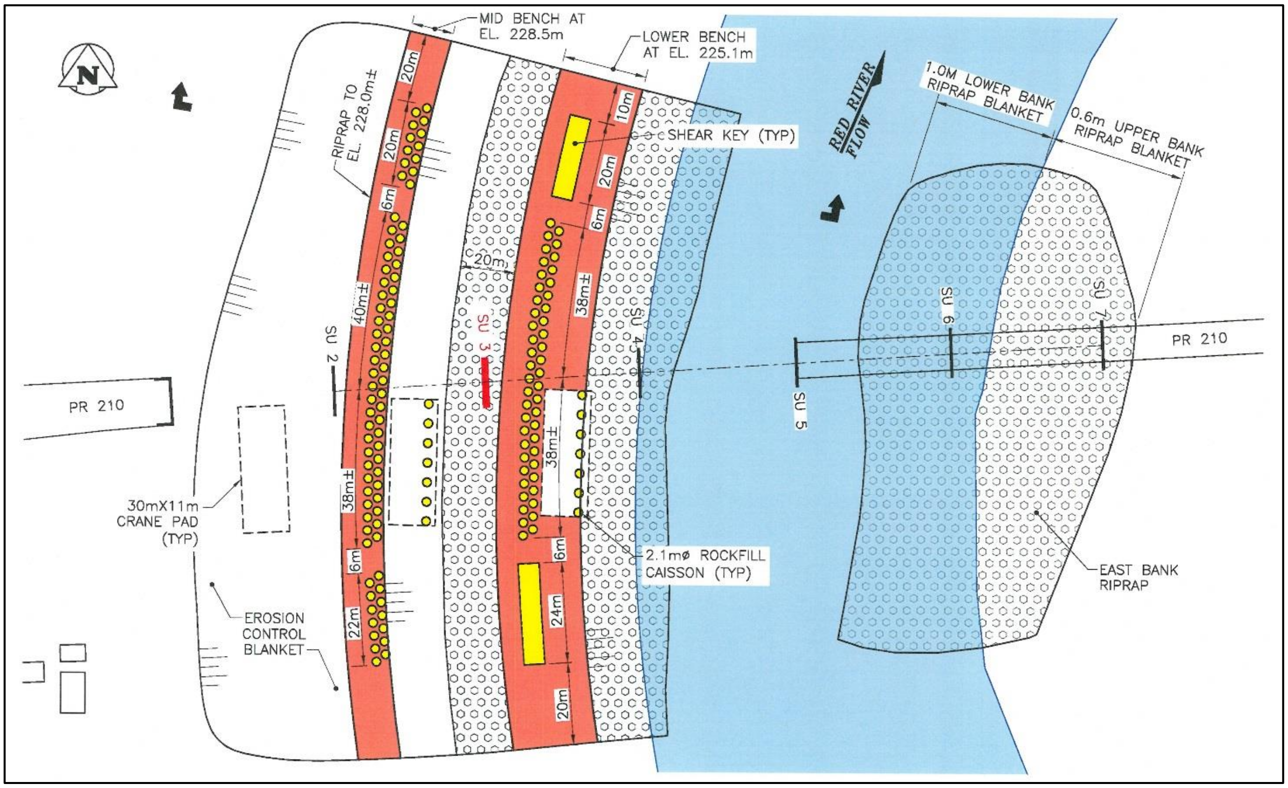

Figure 15. Plan of permanent stabilization works.

bank stabilization resulted in a stabilized riverbank upon which conventional reconstruction of the bridge could proceed. The old axiom that excellent engineering requires excellent construction was proven true over and over on this project. Here the Owner, the Contractors, and the Engineers worked around the clock in a very collaborative and problem solving mode to achieve successful stabilization and demolition.

6. REFERENCES

Arpin, B.P., Kenyon, R.M., Mann, J., Rahman, K., Eden, R. and Umadat, J., 2016. *Passive Groundwater Depressurization System for Improving Road Embankment Stability*, Proceedings of 69th Canadian Geotechnical Society Annual Conference, Vancouver, B.C.

Baracos, A., Graham, J. and Domaschuk, L., 1980. Yielding and Rupture in a Lacustrine Clay, Canadian Geotechnical Journal, 17, 559-573.

Bartz, J.R., Kenyon, R., Arpin, B., Flynn, D. and James, A., *Design and Construction of Rockfill Columns in Deep Clay Deposits with Artesian Groundwater Conditions.,* Proceedings of 71st Canadian Geotechnical Society Annual Conference, Edmonton, AB.

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

Friesen, J., Adedapo, D., Kenyon, R. and Eden., R., 2012. *Bridge Embankment Stabilization in Deep Soft Lacustrine Clays Under High Artesian Pressures*, Proceedings of 65th Canadian Geotechnical Conference, Winnipeg, MB.

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

Kjartanson, B., 1983. *Geological Engineering Report for Urban Development in Winnipeg*, Department of Geological Engineering, University of Manitoba, Winnipeg, Manitoba.

Razaq, A., 2007. *Evaluation of Riverbank Stabilization Using Rockfill and Soil-Cement Columns, (Ph.D., thesis*), University of Manitoba, Winnipeg, Manitoba.

Skaftfeld, K.M., Kenyon, R.M. and Van Helden, M.J., 2009. *The Design Construction and Expansion of the Red River Floodway*, In proceedings of University of Minnesota 57th Annual Geotechnical Engineering Conference, Minneapolis, MN.

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.s

Tutkaluk, J., Kenyon, R., and Graham J., 1998. *Effects of Riverbank Hydrogeology on Riverbank Stability*, In proceedings of 51st Canadian Geotechnical Society Conference, Edmonton, AB.