LANDSLIDE STABILIZATION MEASURES FOR A BRIDGE CROSSING

Bradley S. Prince, M.Eng., P.Eng., AMEC Earth & Environmental, Edmonton, Canada
David J. Walter, Ph.D., P.Eng., AMEC Earth & Environmental, Edmonton, Canada
E. C. (Ed) McRoberts, Ph.D., P.Eng., AMEC Earth & Environmental, Edmonton, Canada

ABSTRACT
The CNRL Horizon project required construction of a primary access road to a major oil sands mine in northern Alberta. This included a bridge over the MacKay River adjacent to valley slopes that had previously undergone extensive slope failures. The roadway design accounted for the possibility that future slope failures could occur at the bridge site. Due to uncertainties in determining the actual safety factor of the valley slopes, the design assumed that valley slopes were originally in a near-failure state and remedial measures were taken to improve stability. Based on strength parameters determined from back-analysis of the south valley slope and a comparison of weak clay materials within the north and south slopes, it was determined that slope stabilization measures would not be required on the northern slope. A shear key and MSE wall were required to improve stability of the south valley slope.

RÉSUMÉ
Une route et un pont ont été construit le long des berges abruptes de la rivière MacKay, pour accéder la mine de sable bitumineux à ciel ouvert ‘Horizon’ de la compagnie CNRL au Nord de l'Alberta. Auparavant, les berges abruptes sur le parcours de cette route d'accès, qui traverse la rivière, ont subit quelques glissements. Étant donné la difficulté d'établir, avec une certaine exactitude, un facteur de sécurité pour la stabilité des berges le long de la rivière, la conception du pont a pris en considération que les berges sont dans un état de quasi-mouvement et que des mesures mitigatives seront prises afin d'améliorer leur stabilité. Une analyse en retour paramétrique pour le versant sud et une comparaison entre les argiles sensibles des versants nord et sud de la dite vallée ont indiqué que des mesures mitigatives de soutènement ne seraient pas nécessaires sur le versant nord. Cependant, un mur de butée et une clef de cisaillement ont été requis pour améliorer la stabilité des berges le long du versant sud de la vallée.

1. INTRODUCTION

The primary access road to the CNRL Horizon oil sands mine in northern Alberta was constructed as a design-build project, financed by CNRL. As part of the roadway project, a two lane, 50 m high, 262 m long bridge was constructed over the MacKay River, approximately 4 km northwest of the town of Fort MacKay. Field investigations and detailed design were initiated in February 2003 and the bridge was open to traffic ten months later in November 2003.

The selection of the bridge crossing site was constrained by a requirement that the roadway bypass land owned by a local First Nations band, and thus, the only available crossing site was located near river valley slopes that had previously undergone deep-seated slope failures. The final roadway alignment was located west of a bowl-shaped landslide feature on the northeast side of the valley and west of a deep gully on the southeast side of the valley (see Figure 1). Dewer (1996), which presents a detailed review of the landslides along the lower 75 km of the MacKay River valley, hypothesized that the slide along the north valley slope originated in basal clays at the base of the McMurray Formation. Other shallower “skin slides” on steep oil sands slopes were observed on both sides of the river valley both upstream and downstream of the bridge crossing.

The design and construction of the site access road and MacKay River Bridge was put out to competitive tender in September 2002 and was awarded in January 2003. The design/build project team included Kiewit Management Company, Associated Engineering Limited, AMEC Infrastructure, and AMEC Earth and Environmental. The specific geotechnical issues concerning the bridge abutments were identified in the bid, as well as the proposed methodology for dealing with them in a timely manner that would not effect the overall project schedule for a multi-billion dollar project. Risks were identified and part of the design included requirements for long term monitoring.

2. VALLEY TOPOGRAPHY

The topography of the project site in relation to the bridge is shown in Figure 2. The crossing is at a skew of about 26 degrees to the river and the river valley is about 305 m wide from crest to crest at the crossing location. The valley becomes wider to the west of the bridge crossing, which results in steeper valley slopes along the east side of the right-of-way than on the west side of the right-of-way. The uplands on the north and south sides of the bridge crossing are at about El. 295 m. The river valley is about 60 m deep, with the river level at about El. 236 m.

The overall slope of the north and south valley walls are approximately 19 and 34 degrees, respectively, with a skew as shown on Figure 2.

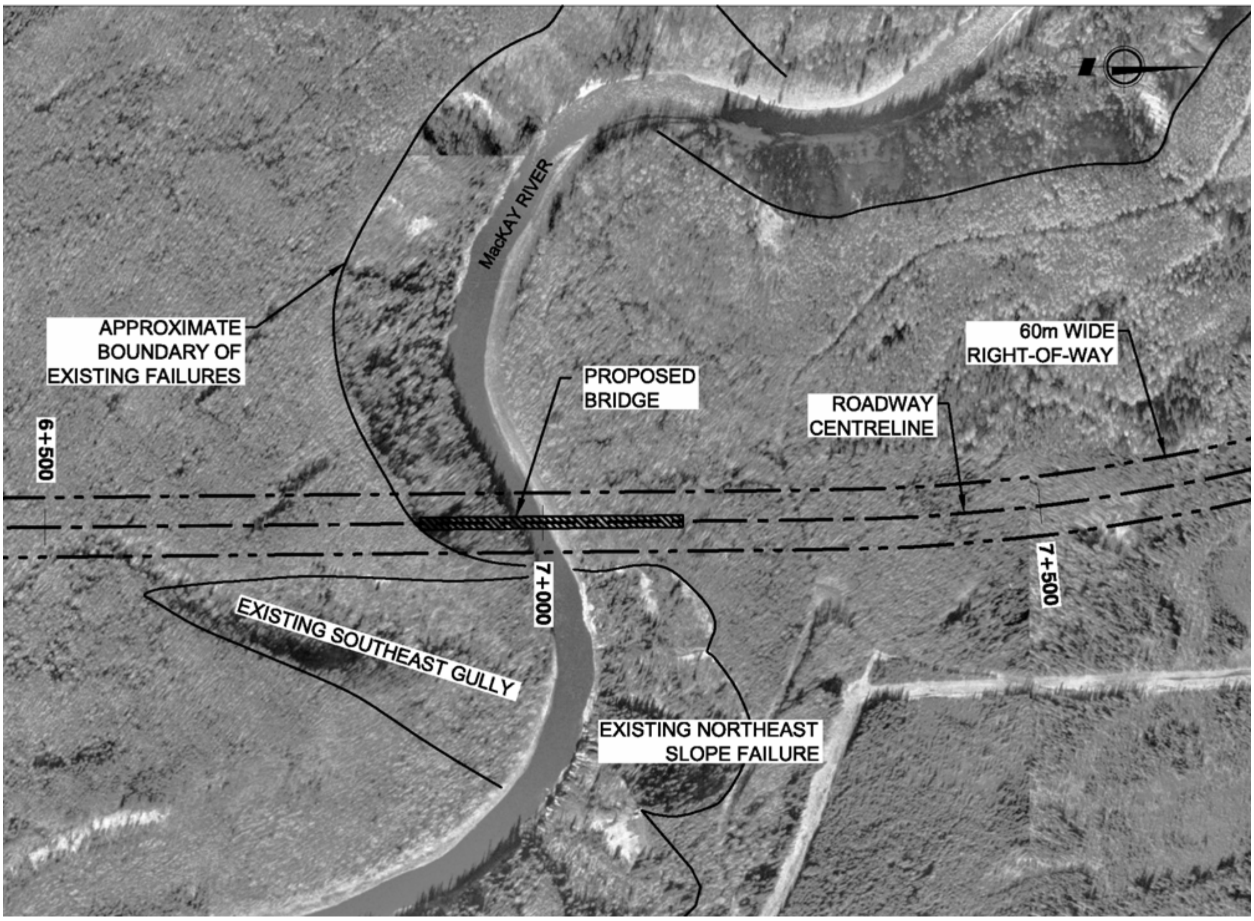

Figure 1. Aerial View of MacKay River Crossing Showing Nearby Slope Failures

3. BRIDGE CONFIGURATION

The MacKay River bridge structure includes three long spans and a shorter jump span, with an overall bridge length of 262 m between abutments (see Figures 2 and 3). The two centre piers are located on either side of the MacKay River above the 100-year flood level and the third pier is located approximately mid-slope on the north side of the valley.

The finished road surface follows a one percent gradient from El. 286.7 m at the north abutment to El. 284.0 m at the south abutment. The approach cut depths are about 8 m and 11 m near the north and south abutments, respectively. The roadway rises at a three percent gradient away from the bridge on both the north and south sides of the river valley.

4. SUBSURFACE CONDITIONS

4.1 Generalized Soil Stratigraphy

A geotechnical investigation was carried out to evaluate the soil and groundwater conditions at the project site. A simplified representation of the sub-surface conditions encountered across the MacKay River valley is provided in Figure 3. The subsurface conditions at the site generally consisted of McMurray Formation overlying Devonian Waterways Formation. At some locations, colluvium and/or aeolian sand overburden was encountered at the ground surface to a maximum thickness of 6 m.

The existing deep-seated valley slope failures observed in the project area likely occurred along weak basal clays located within the Lower McMurray Formation and the Paleosol clay located at the top of the Devonian Waterways Formation. Upon shearing, both of these materials exhibit relatively low residual strengths that are significantly less than their peak strengths.

In addition to the brief descriptions below, the reader is referred to Dusseault (1977) and Dewer (1996) for additional information on the geotechnical characteristics of the McMurray and Devonian Waterways Formations, including basal clays and Paleosol.

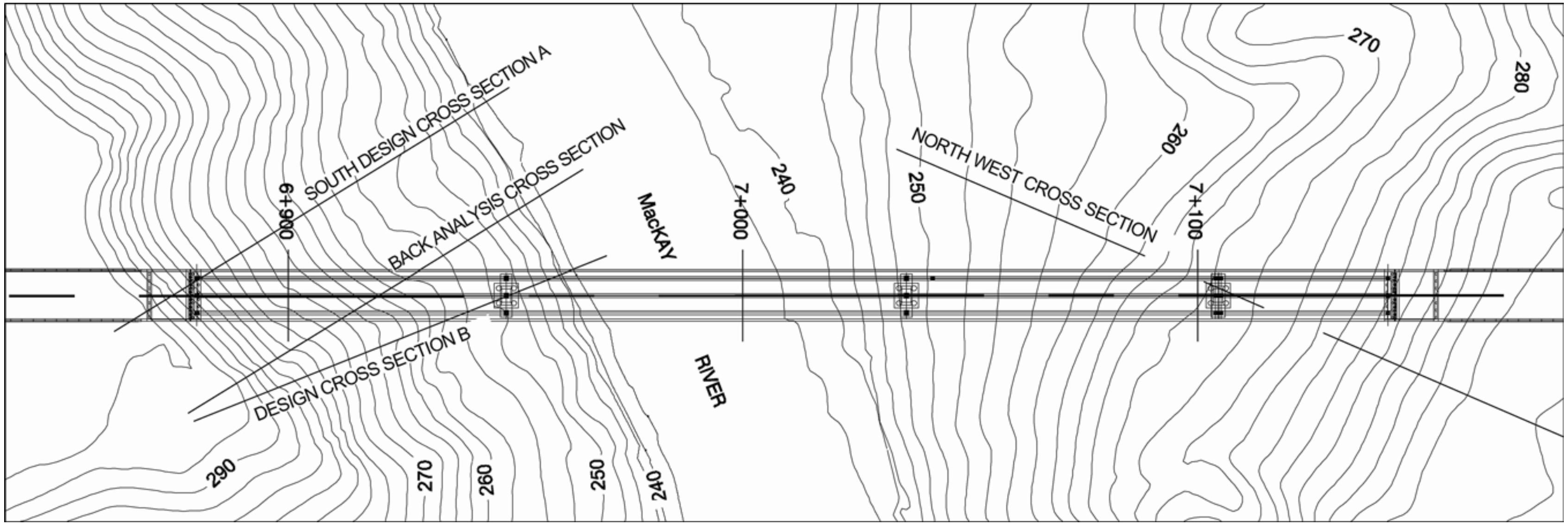

Figure 2. Site Plan and Topography

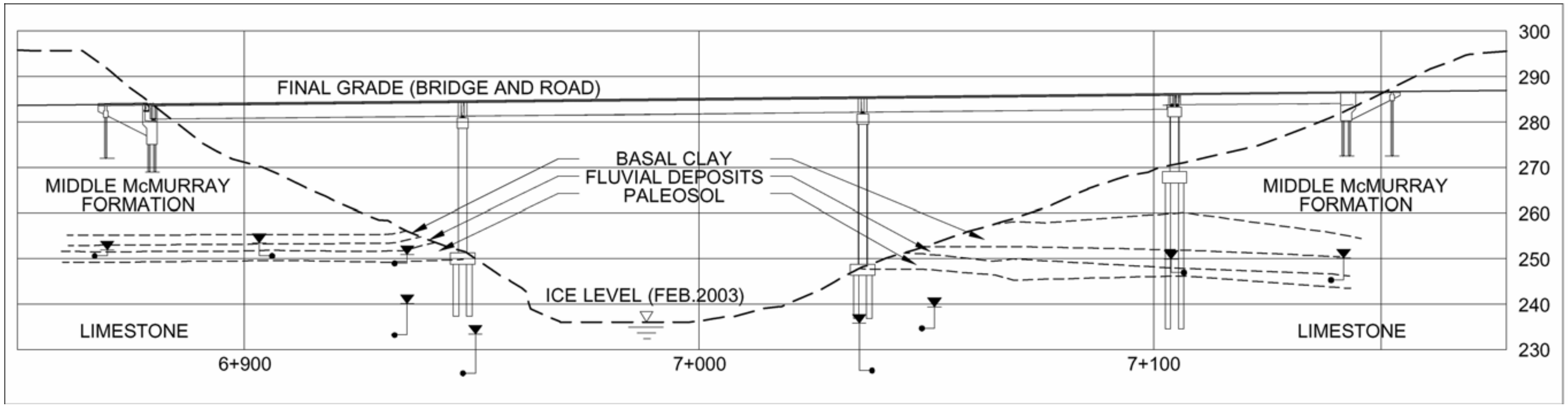

Figure 3. Bridge Profile and Generalized Soil Stratigraphy

4.2 McMurray Formation

The McMurray Formation is informally divided into three members: lower, middle and upper. The upper member was not present in the core obtained from the site.

The middle member of the McMurray Formation is a sequence of estuarine channel sand deposits overlain by shale and interbedded sand. The dominant Middle McMurray facies observed in the core included Tidal Flat Mud, Tidal Channel Sand, and Tidal Flat Sand.

The Lower McMurray is a sequence of coarse-grained sands exhibiting cross-stratification overlain by silt and shale deposited on floodplains. Overbank and pond muds, referred to as basal clays, were encountered in the upper portion of the Lower McMurray. The basal clays were predominantly fine-grained materials containing numerous slickensides, particularly within the Pond Mud facies. Underlying the basal clays were fluvial deposits that only developed locally in deeper depressions on the surface of the underlying Devonian Waterways Formation. The dominant fluvial deposits observed in the core included Coal, coarse Fluvial Channel deposits, Fluvial Channel Watersands, and Fluvial Channel Muds. The fluvial deposits were generally coarser grained deposits containing some clay layers, with a few isolated slickensides. The Lower McMurray was observed between the base of the Middle McMurray, which varied from about El. 255 to 260 m, and the Devonian Waterways Formation contact, which generally varied from about El. 244 to 250 m.

4.3 Waterways Formation

The Devonian Waterways Formation, underlying the Lower McMurray, generally consisted of gently-folded, alternating successions of rubbly, thinly-interbedded, argillaceous limestones and shales interbedded with massive jointed, limestones. An apparently continuous, thin Paleosol clay unit was observed along the upper surface of the limestone bedrock at the top of the Waterways Formation. The Paleosol encountered was generally a non-calcareous, very stiff, low to high plastic, waxy, massive, blue to greenish grey, silty clay that was highly slickensided in places.

4.4 Groundwater Conditions

Pneumatic and standpipe piezometer tip locations and the corresponding water levels measured in April 2003 are shown in Figure 3.

5. STABILITY OF VALLEY SLOPES

5.1 History of Slope Movements in Basal Clays

Basal clays are of geotechnical importance in the development of oil sands mines in the region. They have contributed to failures of pit slopes and have occasionally contributed to the overall movement or failures of existing valley walls or of tailings or waste storage structures built on these clays. Treen et al (2002) reports on a case record of deep-seated landslides in the Wood Creek valley underlying a major tailings dam being constructed at this location. In this case record, back-calculated strengths of 14 to 15 degrees were obtained for clay with liquid limits up to 100 percent. These strengths were considered representative of a fully softened peak friction angle. Testing of these clays typically indicated full residual angles in the order of 9 degrees, depending on plasticity.

At the project site, the instabilities occurring to the east of the alignment appear to be directly associated with the occurrence of a low in the Devonian bedrock and the presence of weak basal clays and Paleosol. These slopes exhibit signs of ongoing and continuing movement. West of this instability and at the crossing location, the valley slopes appear to be more stable, but do exhibit signs of shallow seated instabilities. The following is a specific discussion of the valley slopes at the proposed crossing.

5.2 Observed Stability of North Valley Slope

In the area of the bridge crossing, the north slope of the river valley exhibits two distinct slope types. The bridge alignment is located near the boundary between these different slope zones. To the east of the alignment (see Figure 1), the north valley slope contains a large bowl shaped slide feature centred on a low point in the Devonian bedrock. Within the bowl shaped feature, there were signs of seepage and active movement of colluvium towards the river. On the western flank of this bowl feature, there were signs of active cracking and failing of material into the bowl. The bridge centreline is located about 35 m west of the western flank of this feature.

Along the bridge alignment and to the west, the slope appears to be relatively uniform and well drained, and does not exhibit any visual signs of ongoing deep-seated instability. The overall valley slope becomes flatter from east to west across the right-of-way. This stretch of the valley wall is located on an inside bend of the MacKay River and is not subject to active toe erosion, therefore, it is considered that the current slope configuration has likely reached a longer term stable slope angle. It is possible that this slope was cut to a flatter angle during downcutting of the valley wall.

5.3 Observed Stability of South Valley Slope

To the southeast of the river crossing is a north-northeast trending gully that exhibits signs of both shallow and deep-seated instability. At the mouth of the gully near the edge of the river (east of the alignment), the overall slopes are undulating with angles ranging between 20 and 25 degrees, and appear to have been formed by deep-seated landsliding towards an observed low in the Devonian limestone. Further up the gully, to the south, material that has slid from the upper east and west slopes of the gully has been deposited at the base of the valley and is subject to active and ongoing movement towards the small stream at the base of the gully.

Along the bridge alignment and to the west, the original south valley slope consisted of a steep upper exposed McMurray Formation face up to 15 m high, with a variable apron of colluvium underlying the steep face. The lower portion of the slope consisted of a shallowly sloping apron overlying a steep cliff of Devonian limestone that extended up to about 14 m above the MacKay River. The primary visible signs of instability in this area were shallow slips / spalling of material from the steeper sloped McMurray Formation units. These shallow slides are apparent in the aerial photo mosaic shown in Figure 1. There is no visual evidence of deep-seated instability.

6. DESIGN METHODOLOGY

It is not unusual for a river valley slope at a bridge location to experience instabilities that require the implementation of remedial measures. What is atypical about the MacKay River crossing is that extensive remedial measures were implemented prior to the slope at the crossing location experiencing any signs of deep-seated movement. As the roadway formed the primary access to a major new oil sands mine, closure of the roadway due to a slope failure at the bridge crossing could potentially have severe implications relative to mine operations. Due to uncertainties in determining the actual factor of safety of the original valley slopes, a design basis was adopted whereby it was assumed that the valley slopes at the crossing were originally in a near-failure state and remedial measures would be implemented to improve the valley wall stability.

The assessment of valley wall stability and remedial design was based on the assumption that a minimum factor of safety of F=1.4 should be achieved for likely failure modes of the north and south slopes. One of the challenges with this project, however, was that the absolute factor of safety of the existing valley slopes was not known. Back analysis is commonly used to determine the mobilized strength of a soil unit within a slope at the time of failure. For slopes that have not yet reached a condition of failure, which is believed to be the case for both the north and south valley slopes at the crossing location, determination of the mobilized strength of the soil units making up the slope is a much more difficult task. It would be possible to estimate the strength of the in situ materials based on the results of an extensive suite of laboratory tests and a knowledge of the stress history of the soil within the slope; however, this would not have been compatible with the required construction schedule.

Because the bridge alignment is located adjacent to active deep-seated slope failures on both sides of the river, it was considered possible that the factor of safety of the existing valley slopes could be marginally above unity for a deep-seated potential movement, say F=1.05. If the strength and behaviour of the Paleosol and basal clay units were different at the north and south valley slopes, then the existing factor of safety could potentially be near unity on both the north and the south slopes. If the strength and behaviour of the Paleosol and basal clay units were similar at the north and south valley slopes, then the existing factor of safety of the flatter north slope would be greater than the factor of safety of the steeper south slope, which was assumed to be about F=1.05.

Back analysis (assuming F=1.05 for all likely failure mechanisms) was used to determine the likely lower bound mobilized strength of the Paleosol and basal clay units. The strength parameters determined from back analysis were then used to determine the extent of stability improvement measures that would be required to achieve the design factor of safety of F=1.4 (i.e. a 35 percent improvement in slope stability).

7. COMPARISON OF NORTH AND SOUTH SLOPES

A key aspect of the design was to assess whether the engineering properties of the Paleosol and basal clay units were likely to be similar between the north and south valley slopes. The implication if the Paleosol or basal clay units on the north and south slopes behaved differently was that each slope would be back-analysed to a factor of safety of near unity, and remedial measures designed such that both slopes would have a 35 percent improvement in stability. Thus, extensive stabilization measures could potentially be required on both the north and south valley slopes. If the Paleosol and basal clay units were deemed to be comparable on the north and south valley slopes, the design of remedial measures for both slopes could be based on a back-analysis of the steeper south slope, thereby reducing the remedial measures on the north slope. It was therefore considered critical to adopt a rational means of comparing the Paleosol and basal clays below the north and south valley slopes.

For comparing drill core from either side of the river, a system for characterizing defects (i.e. slickensides) in the Paleosol and basal clay was adopted. With this system slickensides were characterized by various criteria including spacing, continuity, geometry, orientation, force required to break core, roughness, and sheen. Slickensides were given a numerical ranking under each of the above criteria that provided a basis for determining the “quality” of the slickensides. Based on the defect assessment, it was concluded that the differences between the slickenside characteristics on the north and south sides of the valley were small for both the basal clays and Paleosol.

A detailed visual examination of the core by experienced senior geologists and geotechnical engineers indicated that the plasticity of the slickensided zones in the basal clay and Paleosol were similar on the north and south valley slopes. A suite of Atterberg limits tests were carried out to allow a more objective comparison of the plasticity of basal clays and the Paleosol units between the north and south slopes. The results of the Atterberg limits tests are summarised in Table 1.

Table 1. Atterberg Limits Test Results

Soil Unit	Parameter	North Slope			South Slope		
		W_L	W_P	I_P	W_L	W_P	I_P
Basal Clay	minimum	26.8	26.8	26.8	49.7	19.6	29.8
	mean	52.8	19.8	33.0	52.2	20.6	31.7
	maximum	84.8	27.2	57.6	54.4	21.4	33.7
	Std. Dev	15.9	3.3	13.3	2.1	0.9	1.9
	No. Tests	8	8	8	4	4	4
Paleosol	Minimum	32.1	16.2	12.8	36.6	16.2	20.2
	Mean	43.6	18.0	25.7	44.3	18.7	25.6
	Maximum	53.2	21.1	32.4	58.4	21.4	38.6
	Std. Dev	6.5	1.5	6.3	7.6	1.8	6.5
	No. Tests	8	8	8	7	7	7

These data indicate that the basal clays on the north side of the valley have very similar plasticity characteristics to the basal clays on the south side of the valley. The same can be said for the Paloesol deposits on either side of the valley. T-tests indicated 79 and 99 percent probabilities that the sets of basal clay and Paleosol samples analysed came from the same two populations with the same mean.

Based on the above evaluations, it was rationalized that basal clay and Paleosol were sufficiently similar between the north and south valley slopes such that the river valley slope stability assessment could be carried out on the basis of back-analysing the south valley slope, and applying the parameters determined from that analysis to the assessment and design of slope improvement measures for both the north and south slopes.

8. STABILITY IMPROVEMENT MEASURES

8.1 Back-Analysis of South Valley Slope

A back-analysis of the steepest section at the south valley slope (see Figure 2) suggested a mobilized friction angle of 21.8 degrees in the Paleosol and basal clay layers for shear parallel to the bedding. Noting that shear strength is based on the tangent of the friction angle, this is approximately 75 percent of the average peak strength (28.6 degrees) and 2.2 times the average residual strength (10.5 degrees) measured in three direct shear tests on Paleosol. Similarly, this is significantly higher than the residual and mobilized strengths reported by Treen et al (2002) for higher plastic basal clays at Wood Creek dam, and the strength used in major tailings dam foundations in more plastic basal clays.

8.2 North Valley Slope

Following excavation of the roadway approach cut and construction of the pile supported bridge abutments, the factor of safety for deep-seated failures that could impact the bridge was determined to be greater than F=1.4 for all likely potential failure modes. There were some shallow slip surfaces generated with factors of safety less than F=1.4, however, it was considered unlikely that those potential near surface failures would impact the stability of the bridge. Therefore, specialized slope improvement measures were not required for the north valley slope.

8.3 South Valley Slope

The excavation required to achieve the design roadway grade at the south abutment helps to unload the crest of the slope, and results in a small improvement in stability for deep failure surfaces. However, this unloading was not considered sufficient to achieve the design factor of safety of F=1.4. Based on analyses of post-construction conditions on the south valley slopes, it was determined that stability improvement measures were required to protect both the south centre pier and the south abutment against potential deep-seated failures. Various options were considered for improving valley wall stability, including slope flattening, increasing resistance of bridge piers to withstand impacts due to slope failures, installation of a tangent pile wall, and construction of a shear key. Of these measures, a shear key was determined to be the most economical option.

The shear key approach required excavation to remove the existing Paleosol and basal clay units from above the limestone surface over a specified width near the toe of the slope, and replacement of the excavated soil with engineered fill. To achieve maximum benefit from the shear strength along the base of the shear key, an MSE wall was utilized at the front face of the shear key (see Figures 4 and 5). This type of slope stabilization method has been used successfully on many other slope stabilization projects, however, there was a risk associated with this option, as the potential for triggering a slope failure during construction increased as the size of the excavation was increased. A construction method was adopted whereby the excavation was carried out and the shear key/MSE wall was constructed in panels to reduce the risk to the bridge due to potential slope failures that could be triggered during construction.

Construction of the shear key/MSE wall was carried out primarily during the summer of 2003. Full-time geotechnical monitoring was carried out during construction, which included frequent reading of inclinometers installed upslope of the shear key. Although the panel size adopted by the design/builder was somewhat larger than visualized during design, the slope performed well with no significant instabilities triggered during construction. The bridge was opened to traffic in November 2003 and the shear key/MSE wall has performed well since that time. Photos of the completed bridge and shear key / MSE wall are shown in Figures 6 and 7.

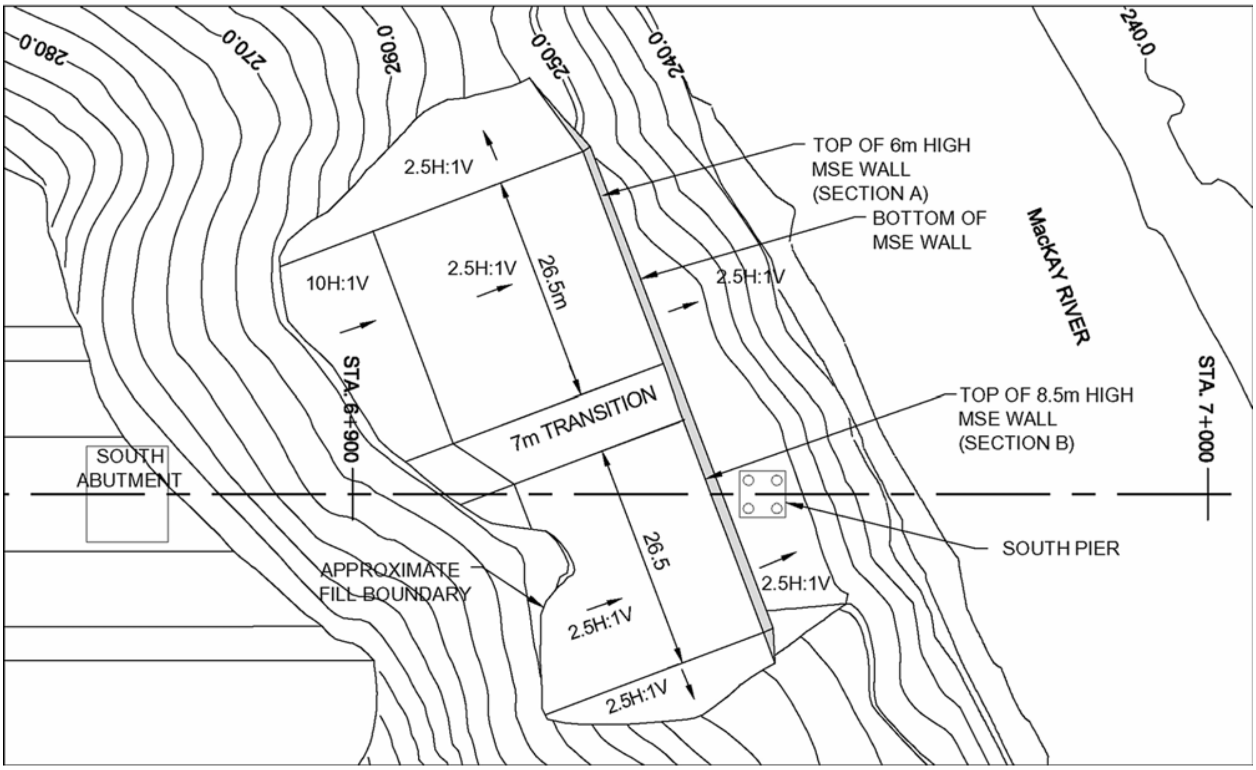

Figure 4. Design Plan – South Slope MSE Wall/Shear Key

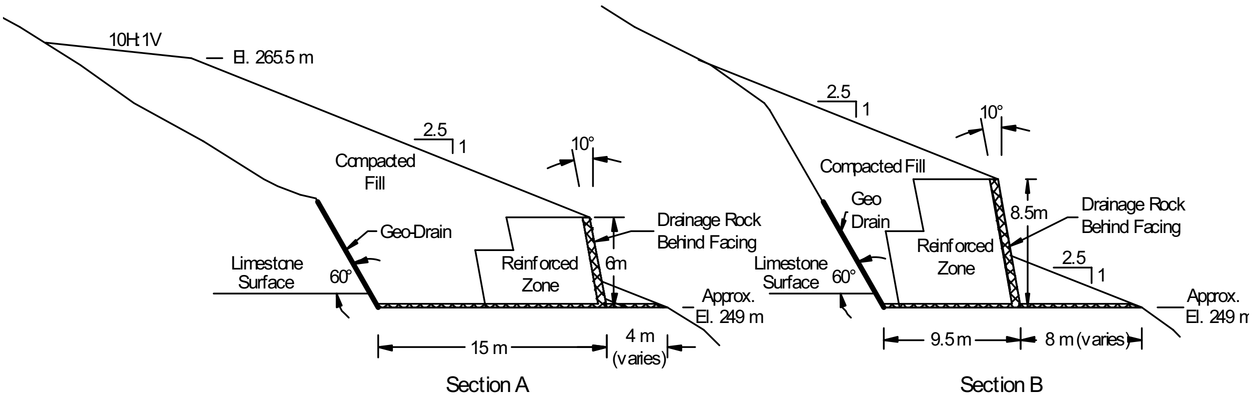

Figure 5. Design Cross Sections – South Slope MSE Wall/Shear Key

9. CONCLUSION

Designing the slope stabilization measures to conventional factors of safety on the basis of lower-bound residual strength parameters would have been prohibitively expensive to this project and the schedule did not permit extensive laboratory testing. A back-analysis of the steepest slope at the bridge location, assumed to be in a state of near failure, was carried out to estimate mobilized soil strengths. This, in combination with statistical analyses of noted drill core deficiencies and laboratory data to demonstrate that the soil properties were similar across the site, allowed for more economical designs without compromising safety. With this approach there is a residual risk that soil conditions were worse than assumed. The owner is managing this risk through on-going slope monitoring.

10. ACKNOWLEDGEMENTS

The authors would like to acknowledge CNRL for granting permission to publish this paper.

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Figure 6. Photo of the Completed Bridge and Shear Key / MSE Wall Looking Southwest

Figure 7. Photo of the Completed Bridge Looking Northwest (note adjacent features)