THE HARROWBY HILLS SLIDES

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Abstract

The Harrowby Hills Slides are located approximately 6 km east of the Saskatchewan-Manitoba border along an 8 km section of the Canadian Pacific Railway. The base of the slides is seated in the Cretaceous Pierre Shale Formation. Field investigations have determined that the ground movement affecting the track is relatively shallow; however, a deep-seated translational slump block has been identified below the track in the toe region of the slide area. The shallow slides are occurring in the 3 to 5 m deep weathered zone where the strength of the shale has weakened due to moisture ingress. The deep seated movement, however, is occurring along a discrete plane. Limit equilibrium analysis of both slide zones provides mobilized shear strength parameters consistent with the mode of movement.

Résumé

Les Glissements de Harrowby Hills sont situés approximativement 6 km est de la frontière entre Saskatchewan et Manitoba dans une section de la ligne ferroviaire du chemin de fer Canadien Pacifique 8 km en longueur. La base des glissements se trouve dans la Formation Crétacée de Schiste de Pierre. Les études de l'emplacement ont indiqués que le mouvement affectant la ligne ferroviaire est relativement proche de la surface. Cependant, une bloque de translation de récession profonde a été identifier dessus la ligne ferroviaire dans la région du pied de la pente. Les glissements peux profonds se produisent dans une zone superficielle par les agents, 3 à 5m en profondeur où la force du matériel a été réduit par l'entrée d'humidité. Par contre, le mouvement profond se produit sur une plaine discrète. Les analyses équilibre limites des deux zones de glissements fournissent des paramètres de force de cisaillement mobilisés conformant au mode du mouvement.

1. INTRODUCTION

In order to develop mitigative measures to control landslide movements, a thorough understanding is required of the processes that drive the movements. Slope movements in weak rocks such as clay shales are particularly problematic as the failure mechanisms are often complex. Clay shales are transitional in nature, exhibiting both soil and rock-like properties, and a tendency to exhibit strength loss with time. Their presence in North America encompass the extensive formations of shale bedrock that extend from north to south across the continent and east from the Rocky Mountains for several hundred kilometres, covering an area once occupied by the epicontinental sea of the Late Cretaceous period (Hardy, 1957 and Brooker and Peck, 1993). This sequence includes continental clastics intermixed and interspersed with beds of coal and bentonite. The water quality conditions under which they were deposited included marine, brackish, and freshwater (Morgenstern, 1977). Despite their relatively flat-lying to gentle-sloping depositional nature, their presence has become synonymous with challenging foundation conditions and natural and man-made slopes. They are well-known and well-documented problematic engineering materials (Skempton, 1964, and Bjerrum, 1967).

The Harrowby Hills Slides (Figure 1) are located approximately 6 km east of the Saskatchewan-Manitoba border near the town of Russell, Manitoba. They are within an 8 km section (between Millwood and Harrowby, Manitoba) of the Canadian Pacific Railway (CPR) built on the west valley wall of the Assiniboine River valley in the late 1800's. This section of track crosses several relict landslides. The most recent ground movement has encroached on the right-of-way at mile 86.8 (a 70 m section) and Mile 86.75 (a 90 m section) of the CPR Bredenbury Subdivision. As illustrated in Figures 1 and 2, these mileage segments are located within a broad valley of relatively gentle-sloping walls (overall slope of 13°) and are coincident with an outside bend of the meandering Assiniboine River.

Figure 1: Aerial view of the Harrowby Hills Slides. Photo courtesy of Clifton Associates Ltd. (2001).

Geotechnical investigations have concluded that these landslide activities entail a series of metastable, retrogressive blocks with failure seated in the high-plasticity clay shale bedrock, the Millwood Member of the Pierre Shale Formation. Landslides in the Cretaceous Pierre Shale are often grouped as either deep-seated sliding along a discrete plane or shallow-seated sliding within the weathered zone. Rupture surfaces in the Pierre Shale tend to be geologically controlled with common occurrences along single planes either in weak zones (bentonite seams) or along stratigraphic boundaries (Crandell, 1952 and Knight, 1963). However, shallow-seated failures in these clay shales have also been noted with movements frequently confined to comparatively shallow depths within the weathered zone and sliding coincident with the inclination of the slope (Bjerrum, 1967). In this paper we explore the mechanisms responsible for the Harrowby Hills slides.

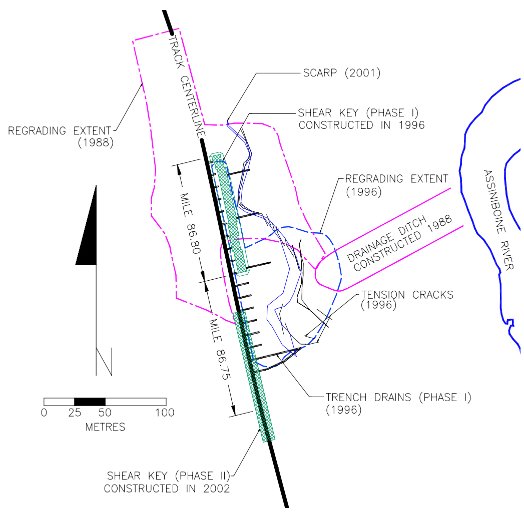


Figure 2: Summary of previous work (contours have been removed for clarity, refer to Figure 4).

2. PREVIOUS WORK

Since the early 1980’s, several site characterization programs have been carried out within this section of track. In 1986 and 1987, an extensive investigation was carried out to determine the site geology, existing slope conditions, and material strength characteristics. At that time, Miles 86.8 and 86.75 were within an area deemed the most unstable during that past year based on track maintenance records, which included replacement of culverts, track lifting and re-aligning, and a daily track patrol throughout the spring. Instability was associated with poor surface drainage and river erosion. It was concluded that these landslide activities entailed a series of metastable, retrogressive blocks with failure seated in the high-plasticity clay shale bedrock, the Millwood member of the Pierre Shale formation.

Remedial measures were undertaken in 1988 and included the placement of rip rap along the river bank at locations of active erosion and improved surface drainage both upslope and down-slope of the track in the form of swale ditches (Figure 2). In 1995, a major subgrade instability occurred following a wet spring and early summer, which resulted in substantial increases in the rate of ground movement (50 mm per week in early May to nearly 350 mm per week in late June and July). The mechanism of instability at Mile 86.8 involved movement down-slope of the track and sloughing upslope of the track.

In October 1996, a remedial program to reduce the track movements was completed; a 100-m-long granular shear key was constructed just down-slope of the track at Mile 86.8 in order to isolate the upslope portion from lower slope failure (Figure 2). Following construction, down-slope tension cracks emerged within 18 to 60 m of the track with associated vertical displacements of 600 to 900 mm. Subsequent inspection revealed that these new cracks occurred along the same paths as historic ones, and most likely resulted from the prolonged wet conditions experienced during the spring and summer of 1996. In December 1996, 50 to 80 mm of vertical and horizontal movements were noted by track personnel over a length of 45 m at Mile 86.75. In October 1999, slope inclinometers installed within the improved area showed the shear key was effective in controlling the movements and the tension cracks shifted to the east. To minimize surface infiltration during spring run-off, re-grading of the down-slope portion has been periodically undertaken in the spring and fall to close any open tension cracks.

In 2000, reconnaissance of the site revealed substantial slope movement down-slope of the track at Mile 86.75 (6 m high scarp) and to a lesser extent at 86.8 (2.5 m high scarp) with a clearly visible bulge in the bank of the Assiniboine River. In early spring of 2002, 250 m of track was temporarily relocated 5 m upslope of the area of closest cracking. In the late spring of 2002, a granular shear key was constructed directly under the track, in conjunction with surface re-grading to control surface runoff and seepage (Figure 2).

In summary, the railway traverses a clay shale slope that is metastable, where movement of the slope is often triggered by prolonged “wet” conditions.

3. GEOLOGY

The bedrock geology of the study area is a sedimentary sequence of shales. The Millwood Member of the Pierre Shale, is an olive-grey, soft, uniform, silty, clay shale with selenite crystals, numerous calcareous concretions, ironstone nodules occurring in bands, and numerous bentonite beds occurring within or close to the top contact with the overlying Odanah member. The Millwood thins towards the southeast and changes from 150 m of silty clay shale at the Saskatchewan-Manitoba border to a 15 m sequence of interbedded non-calcareous and calcareous shales and thin marls at the Canada-United

States border. (McNeil and Caldwell, 1981, and Stott and Aitken, 1993)

The Millwood Member includes fine sand dispersed throughout the member. The Millwood shale is hard, showing laminations, but not strikingly fissile and tends to break into equi-dimensional particles when dried. The shale slakes easily, turns into plastic clay in water, which often covers exposed surfaces, and becomes hard and mud-cracked as it dries. In addition, the plasticity of the material triggers slides when the slopes are steepened. (Kirk, 1929)

The Millwood Member of the Pierre Shale (also known as the Riding Mountain Formation) has been correlated with: the Lea Park, Judith River, and the Bearpaw Formations in the Canadian prairies, and the Gregory and DeGrey Members of the Pierre Shale in the Dakotas (Douglas et al., 1970, McNeil and Caldwell, 1981, and Stott and Aitken, 1993).

4. FIELD INVESTIGATION

Topographic investigations indicated that the overall nature of the valley wall was rather gently sloping and hummocky due to past landslide activity. In addition, six drainage courses and four ponded areas were identified as immediately affecting the study area. Surface water sources include not only precipitation and surface runoff, but also drainage from the layers of sand and gravel found at surface and in the blanketing till sheet, and two springs located above the track.

The subsurface has been explored by a combination of 41 boreholes and 23 test pits excavated within the slope and the upland plateau. Sampling has also been carried out via grab samples, washed cuttings, Shelby tube, and continuous coring. In addition, block sampling was undertaken in 2002 for the work in this paper. Almost all boreholes were instrumented with either piezometers or slope inclinometers. Overall, 33 piezometers and 11 slope inclinometers were installed from 1986 to 2001. However, monitoring has been sporadic and irregular.

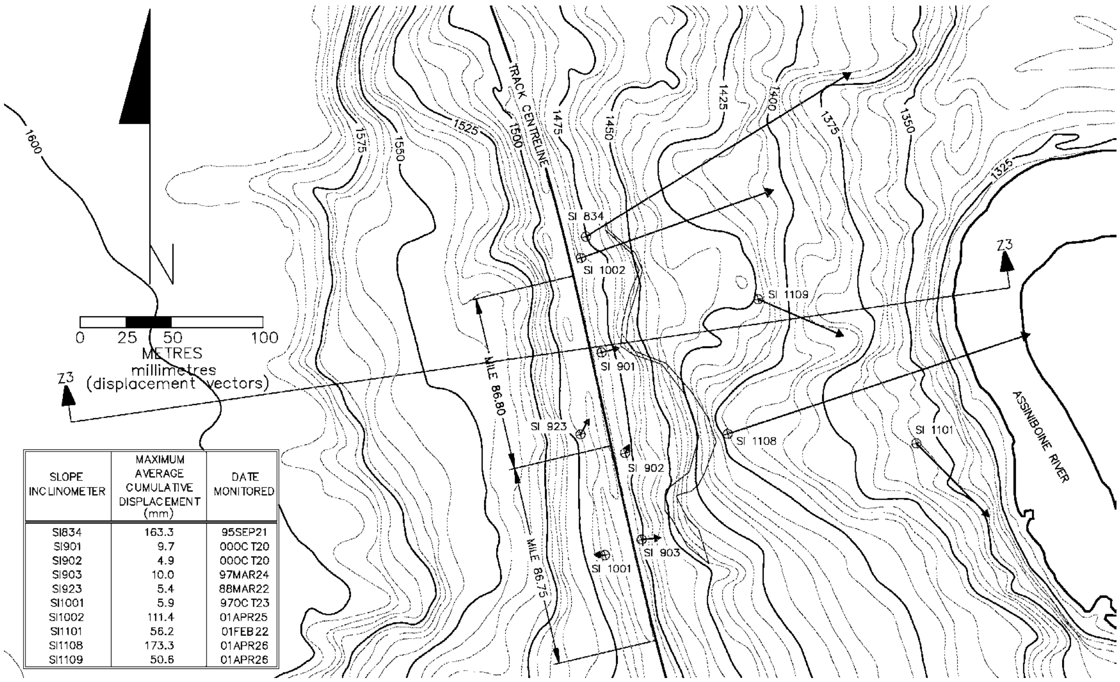
The stratigraphy was based on interpreted bedrock elevations reported in the borehole logs, and the use of water content and dry density profiles. The stratigraphy was simplified to three strata: 1) glacial till, 2) intermix of till and shale, and 3) clay shale bedrock. The water contents for the shale bedrock were approximately 20% while the dry densities were in excess of 1.7 Mg/m^3. In general, the shale bedrock appeared to follow the slope topography in a quasi stepwise fashion. Groundwater levels generally coincided with the ground surface during prolonged “wet” periods, and the pore-water pressures on the failure surfaces are not well understood due to the exclusive use of standpipe piezometers.

During the excavation of the second shear key in late May 2002, observations indicated that the weathered zone of the shale bedrock was rather blocky and jointed with evidence of orthogonal joint sets (Figure 3). The shear-key excavation did not reveal a distinct shear zone at base of the movement zone. However, a prominent horizontal iron-stained plane (dashed line in Figure 3) located within the weathered zone of the bedrock was thought to be a possible base for the movement zone. No seepage was observed from this plane, although it has undoubtedly occurred in the past. The shear-key excavation was dry and the walls were maintained at about 70 degrees without any external support. Block samples of intact clay shale were obtained from this excavation for laboratory testing purposes.

Figure 3: Degree of jointing in the weathered zone in the upslope wall of the first bench of the shear-key excavation. Note the iron-stained plane which is thought to be the base of the movement zone (shown by the dashed line).

Location of the shear plane was based on both borehole logging in the sparsely instrumented upper upslope area and slope inclinometer (SI) data of the relatively well instrumented near-track and down-slope portions. Generally, the shear plane was found to follow the surface topography for the majority of the slope with the exception being the toe area. The SI’s indicated relatively shallow movement zones within most of the slope with a somewhat deeper-seated movement zone at the bottom of the slope.

The pattern of movement from the inclinometer data is illustrated in Figure 4. The figure indicated that the maximum displacements and displacement rates occurred closest to the scarp (SI 834, SI 1002, and SI 1108 with displacements ranging from 111.4 to 173.3 mm and rates ranging from 17.8 to 645.6 mm/yr). Displacements and rates decreased (SI 1109 and 1101 with displacements ranging from 50.6 to 56.2 mm and rates ranging from 202.9 to 476.7 mm/yr) as the toe was approached. In addition, the vectors of SI 834, 1002, and 1108 pointed towards the northeast, while SI 1109 and 1101 pointed toward the southeast (Figure 4). Hence, it appeared that the lower block (defined by SI 1109 and 1101) was moving in a different manner and direction from the upslope mass.



SLOPE INCLINOMETER	MAXIMUM AVERAGE CUMULATIVE DISPLACEMENT (mm)	DATE MONITORED
SI834	163.3	95SEP21
SI901	9.7	00OCT20
SI902	4.9	00OCT20
SI903	10.0	97MAR24
SI923	5.4	88MAR22
SI1001	5.9	97OCT23
SI1002	111.4	01APR25
SI1101	56.2	01FEB22
SI1108	173.3	01APR26
SI1109	50.6	01APR26

Figure 4: Displacement vectors indicated by the slope inclinometers

5. LABORATORY TESTING

Standard laboratory tests were carried out to characterize the intact shale. In addition, observations were made on the behaviour of the material when placed in water, similar to the testing carried out by Morgenstern and Eigenbrod (1974). Cementation was also investigated and samples were examined under the scanning electron microscope (SEM)

5.1 PHYSICAL AND MECHANICAL PROPERTIES

The grain-size distribution of the material tested suggested a primary dominance in the silt-sizes with a secondary proportion of clay sizes. The material was found to be inorganic clay of high plasticity with a massive fabric displaying no fissility. SEM analysis indicated a clay matrix with an integrated coarser fraction. This is consistent with the range of hydraulic conductivities determined from consolidation tests, which ranged from 3.3×10^{-10} m/s to 6.5×10^{-12} m/s. These characteristics are consistent with a medium in which the clay fraction dominates the engineering behaviour.

The behaviour of the shale was also observed in water using in-situ samples (with respect to water content) and air-dried samples. Samples at the natural moisture content did not display any degradation when placed in water for a 1-month period. However, samples that were first air-dried, instantly disintegrated when placed in water. It was hypothesized that cementation might be responsible for controlling this behaviour. The possibility of carbonate-based cementation was discarded since there was no visible surficial reaction on exposure to a 100% solution of hydrochloric acid. However, there was strong indication of cementation under the SEM; the source of this cementation was postulated to be of organic origin, such as sugars or biological mucus (Figure 5). Regardless of the origin of the cementation, it was clear that upon drying the cementation had no effect in controlling the slaking characteristics of the shale.

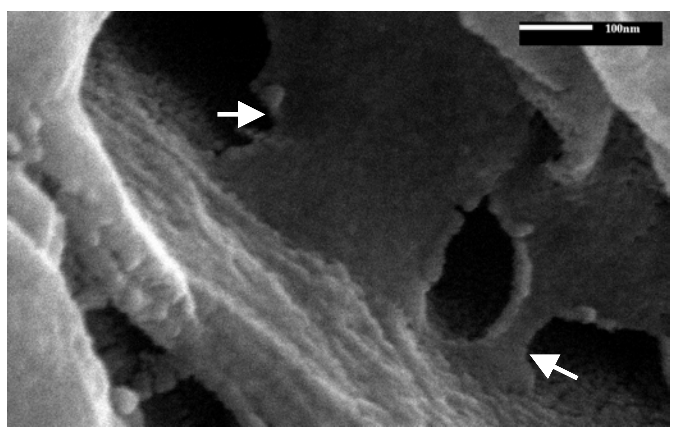


Figure 5: SEM showing the possibility of cementation (indicated by the arrows).

5.2 STRESS-STRAIN CHARACTERITICS

Samples of the Millwood member tested under direct shear and consolidated-drained and consolidated-undrained triaxial conditions indicated a brittle material with an elastic modulus ranging from 32 to 73.2 MPa. The reduction in strength from peak to residual appeared to be significant, ranging from 38% to 79%. Under the low confining pressures of both the direct and triaxial tests, the material exhibited typical overconsolidated behaviour, resulting in brittle stress-strain curves, well-defined failure planes, and a degree of dilatancy. In addition, variability was evidenced by the range of stress-strain curves that included brittle and ductile behaviour (Figure 6). This can be indicative of complexity in the soil/rock mass structure and has strong implications on the possibility of progressive failure (Chandler, 1984).

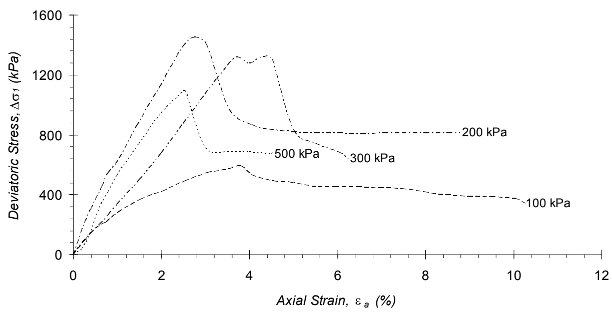


Figure 6: Variability in stress-strain relationships of lab samples

5.3 STRENGTH CHARACTERITICS

Peak strengths from the direct shear tests of intact unweathered samples resulted in a cohesion intercept of 184.7 kPa and a friction angle of 28.7°, while the triaxial tests indicated an effective cohesion of 139.3 kPa and a friction angle of 36.1°. According to the direct shear tests, the residual strength of the material had cohesion of zero and a friction angle of 16.7°.

6. LIMIT EQUILIBRIUM ANALYSIS

A limit equilibrium model was developed based on the interpretations from the field investigations. The failure mechanism (Figure 7) postulated was a shallow-seated, retrogressive failure. Slope/W was used to conduct back analysis to establish the mobilized shear strength.

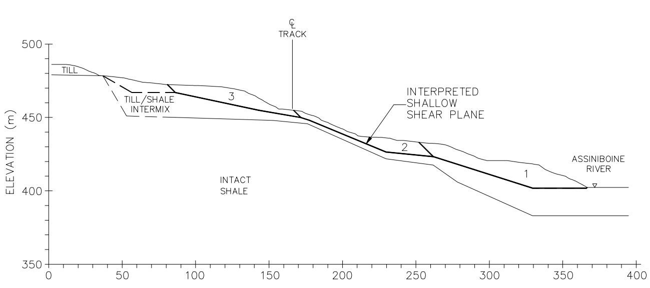


Figure 7: Cross-section of the shallow mechanism (refer to Figure 4 for location of this cross-section).

6.1 SHALLOW-SEATED MECHANISM

Initially, the shallow-seated failure was analyzed with lab-determined residual values and the pore-water conditions at ground surface, which was not unreasonable as movements in this slope has been measured since 1986 and the movement is usually associated with surface seepage and wet conditions. However, under these circumstances, the calculated factor of safety (FOS) was significantly less than unity (Table 1). The model was then analyzed with lab-determined peak values, and as expected, these FOS were significantly greater than unity (Table 1). These shear strength parameters and their corresponding factors of safety are plotted in Figure 8. From this plot, it is clear that the lab residual strength is too low, yet the lab peak strength is much too high; thus, the mobilized strength must be somewhere in between with greater proximity to the lab residual values. Interestingly, the FOS values obtained from the limit equilibrium analysis (LEA) of the residual strength values were close to each other for all the different analysis stages (maximum difference was 4 - 6%). This implied that no matter how the upper blocks were combined with the lower block, the behaviour of the lower block governed the overall behaviour of each of these combinations.

Table 1: Shallow-seated mechanism (lab residual, LR: ϕ' =16.7°, c' = 0 kPa; lab peak, LP: ϕ' =28.7°, c' = 184.7 kPa; mobilized strength, M: c' = 0 kPa).

Analysis Stage	GW Conditions	FOS_{LR}	FOS_{LP}	ϕ'_M (°)
Block 1	at surface	0.61	5.53	26.4
	at shear plane	1.27	6.76	13.3
Blocks 1-2	at surface	0.63	6.59	25.2
	at shear plane	1.33	7.85	12.8
Blocks 1-2-3	at surface	0.64	6.85	24.9
	at shear plane	1.35	8.15	12.5

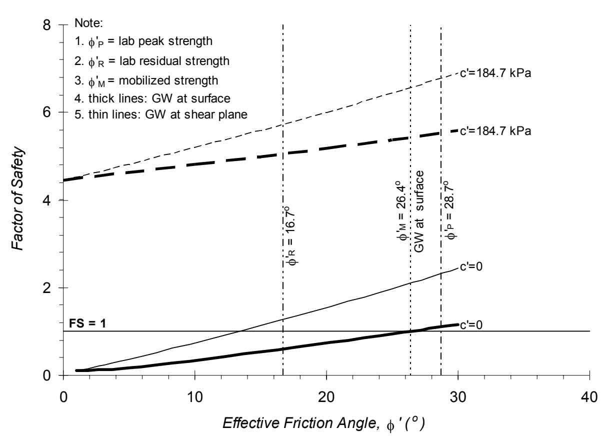


Figure 8: Shear strength parameters for the shallow-seated mechanism (lower block).

The mobilized strength of the slope was also back-calculated for a FOS of unity. For this analysis it was assumed that the cohesive strength was essentially zero due to the extensive jointing observed during shear-key construction. Mobilized friction angles between 24.9° to 26.4° provided FOS=1 (Table 1). These friction angles are almost 150% higher than the laboratory residual strength, but are only slightly less than the peak friction angle (28.7°) determined from the direct shear tests.

Chandler (1984) summarized the behaviour of landslides in over-consolidated clays and soft rocks. He noted that "softening" is one of the most active processes in shallow slides. He defined softening as a time dependent process that leads to a reduction in drained strength resulting from an increasing void ratio under constant effective stress. Chandler showed that softening leads to a loss of cohesion with little effect on the frictional strength. Hence, it would appear from the back analysis of the mobilized strength that softening is a likely mechanism controlling the stability of this slide.

An alternative explanation to softening is that the slide is controlled by retrogressive block movement. Sauer (1983) showed that the mobilized strength back-calculated from such slides is a function of the number of blocks included in the analysis (Figure 9). For example, back-analysis of the toe block results in lower mobilized strength parameters than when all the blocks are included. The results from the retrogressive analysis are shown in Figure 10. These results clearly conflict with the results suggested by Sauer (1983) in Figure 9. In fact, the relative relationships of the stages were in reverse order. The last two stages (blocks 1-2 and blocks 1-2-3) were very close to each other with a difference of 1.2% in friction angle and 3.5% in cohesion. The analysis also implied that the behaviour of the lower block may deviate from its combination with the other blocks owing to its relatively large distance from the subsequent stages. However, it also implied that once the lower block was combined with its adjacent upslope block, its overall behaviour would be substantially altered.

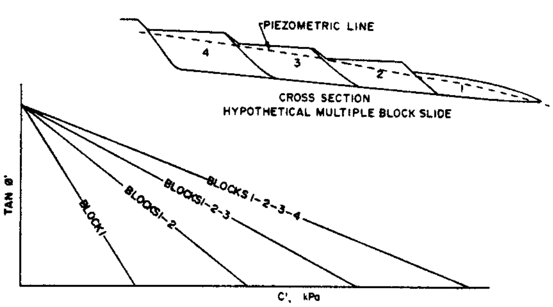


Figure 9: Idealized retrogressive failure (after Sauer, 1983).

Based on the deformational pattern of the displacement vectors in Figure 4, it is unlikely that a retrogressive mechanism is present in these movements. Thomson and Hayley (1975) at the Little Smoky site indicated that the deformational pattern distinctive of a retrogressive failure mechanism was one where the shortest displacement vectors were found closest to the scarp while the maximum vectors were found at the toe.

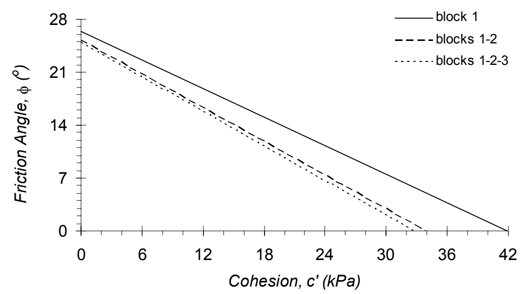


Figure 10: Analysis for a retrogressive mechanism in the shallow-seated failure.

6.2 DEEP-SEATED MECHANISM

The evidence for deep seated movement was limited to one inclinometer (SI 1101) with the back-scarp defined by SI 1109. Movement of approximately 50 mm occurred near the toe of the slope very close to river water level; movements in SI 1101 were recorded at Elev. 401.8 and river level was at Elev. 402.3. In this region, the outside of the river bend, river erosion was most active. The slope inclinometers indicated movement occurred along a discrete plane <1 m thick. Hence, only the lower block (block 1 in Figure 11) was analyzed with material properties based on the lab residual shear strength parameters. The resulting FOS for pore-water conditions at ground surface was 0.8 (Figure 12). Reducing the pore-water pressure to 4 m below the surface gave a mobilized friction angle of 17° for a FOS=1.

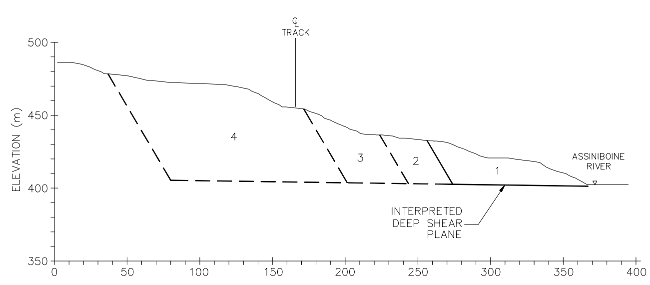


Figure 11: Cross-section of the deep mechanism (refer to Figure 4 for location of this cross-section).

The mobilized strength of the lower block was also back-calculated for a FOS of unity (Figure 12). Again, for this analysis it was assumed that the cohesive strength was essentially zero. Like the shallow-seated failure, the resulting mobilized strength was lower (26.8%) than the peak strength determined from the direct shear tests, but higher than the residual lab values (Figure 12). Unlike the shallow-seated failure, it was closer to the residual lab values (20.5% variability compared to a difference of 32.9 - 36.7% in the shallow-seated scenario).

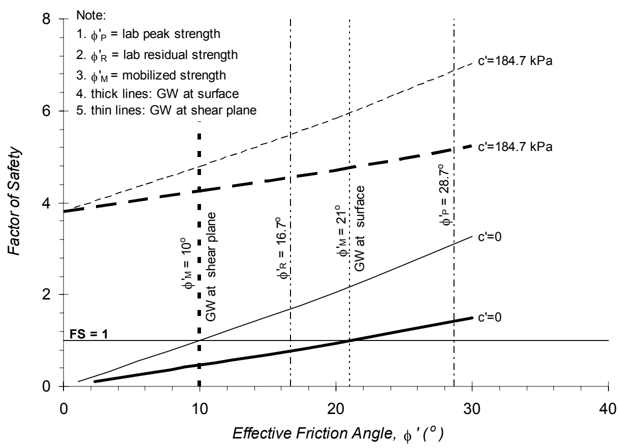


Figure 12: Shear strength parameters for the deep-seated mechanism (lower block).

Results from the retrogressive analysis are illustrated in Figure 13. Unlike the shallow-seated failure, the feasibility of a retrogressive mechanism was more apparent and realistic in the deep-seated mechanism; the plotted results of the back-analysis revealed a relationship not far from the expected ideal retrogressive mechanism of Figure 9. An exception to this was the behaviour of the lower block as a single entity as it had a much larger friction angle, but not a markedly smaller cohesion than the subsequent stage. Mobilization of the combined blocks resulted in a strength near the lab-determined residual (5.1 to 13.5%), while that of the lower block lay farthest away from the residual (20.5%). Again, the anomalous behaviour of the lower block as a single entity was evident in the deep-seated failure. Similar to the case of the shallow-seated failure, once the lower block was combined with the adjacent upslope block, this unexplained behaviour disappeared.

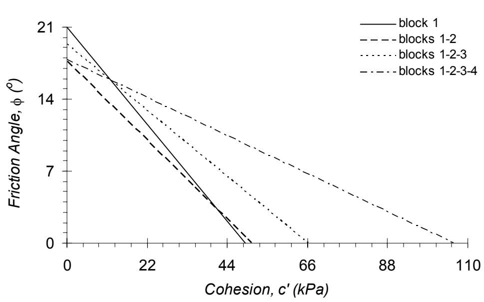


Figure 13: Analysis for a retrogressive mechanism in the deep-seated mechanism.

The back-calculated friction angle for the lower block implied a mobilized strength between the peak and residual values. This was unexpected for movement along a pre-sheared surface or a soil whose strength had somehow been sufficiently degraded to a residual value. Chandler (1984) suggested three basic mechanisms by which the discrepancy between the laboratory and field strengths can be explained: softening, time-dependent limit states, and progressive failure. Progressive failure may occur with respect to the bulk strength or due to softening and/or time-dependent limit states (i.e. rheological loss of strength resulting in critical state), with the mobilized strength lying between peak and residual (Chandler, 1984).

The principal difficulty with the concept of progressive failure is the number of different mobilized strengths which can exist along the slip surface at any one time owing to the fact that at no two points along the slip surface will the movements be equal. Skempton (1964) has claimed that first-time slides in clay shales that occur at strengths significantly greater than residual, but mobilize a softened strength, are not uncommon. Hence, it is quite possible that movement of the lower block represents a first-time slide where either the back-analyzed strength represented an average of the bulk strength along the slip surface or a softened strength significantly greater than the measured residual.

7. SUMMARY AND CONCLUSIONS

The original hypothesis for the failure mechanism occurring at Miles 86.8 and 86.75 was a shallow-seated retrogressive mechanism. This was postulated based on the field investigations and the inclinometer data. However, the LEA has proven otherwise:

1. LEA of the shallow-seated mechanism with lab residual values resulted in FS << 1, but resulted in FS>>1 with lab peak values
2. back-analysis of both the shallow- and deep-seated mechanisms using c' = 0 resulted in friction angles very near the peak lab value in the shallow-seated mechanism, but required a value nearly the average of the lab peak and residual in the deep-seated
3. a retrogressive mechanism in the shallow-seated failure was found to be unlikely; conversely, the opposite was true for the deep-seated failure
4. in both LEA of the shallow- and deep-seated mechanisms, the lower block as a single entity appeared to behave in a manner different from its combination with adjacent upslope blocks
5. movements recorded in the inclinometers indicated that the lower block was moving in a direction and at a rate different from the upper mass

Accordingly, a dual failure mechanism has been proposed to explain the movements and LEA at this site (Figure 14). The toe, defined by the lower block, appears to be moving under a deep-seated mechanism along a discrete plane independent of the upper portion of the slope which consists of shallow-seated mass wastage moving in response to removal of its toe support when the lower block moves; the two failures occur along different planes of weakness. Loss of cohesion, as indicated by higher water contents most likely due to weathering, has been proposed as the cause of failure in the shallow-seated mechanism, whereas progressive failure appears to be the most likely cause of the deep-seated mechanism.

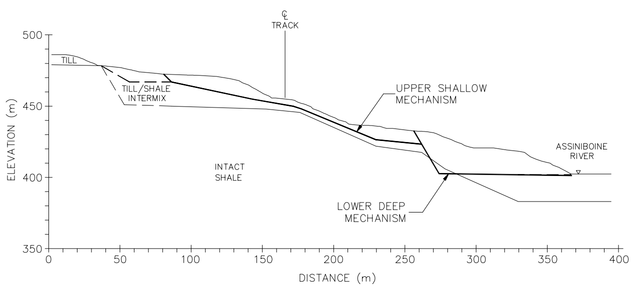


Figure 14: Proposed failure mechanism.

The effects of retrogressive failure seem to elude the shallow-seated failure, but impart a strong presence in the deep-seated failure. This implies that if the lower block moved to an extent, which resulted in the propagation of its plane of weakness into the adjacent mass, retrogressive failure is a very likely occurrence. As the LEA illustrated, once this transpires, the independent behaviour of the lower block disappears and becomes amalgamated with any combination of the upslope blocks. In this case, the operational strength required for mobilization of the blocks is considerably lower than the peak strength and closely concurs with the lab residual values. Hence, it appears that stability of the lower block would have a significant influence on the overall performance of the slope.

ACKNOWLEDGEMENTS

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