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The Problem of Estimating the Shear Strength of Unstable Rock Slopes

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INTRODUCTION

The problem of estimating the shear strength of rock masses has been dealt with at great length in the literature. Much of this work has concentrated on the strength of discontinuities since failure through rocks with a high intact rock strength normally occurs along discontinuities. Extensive laboratory tests have been conducted to determine the strength of these discontinuities and a relationship which fits this experimental data well is that proposed by Barton and Choubey (1977).

$$\tau = \sigma'_n \, \mathrm{Tan}(\mathrm{JRC} \log_{10}(\mathrm{JCS}/\sigma'_n) + \phi_b) \qquad (1)$$

where
τ = peak shear strength
σ'_n = effective normal stress
JRC = joint roughness coefficient
JCS = joint wall compressive strength
ϕ_b = basic friction angle of smooth planar discontinuities

However Hoek (1983) has also pointed out that there are other curve fitting equations for the shear strength data of discontinuities, not the least of which is the Hoek and Brown failure criterion (Hoek and Brown (1980)). This failure criterion is as follows:

$$\sigma'_1 = \sigma'_3 + (m\sigma_c\sigma'_3 + s\sigma_c^2)^{1/2} \qquad (2)$$

where
σ'_1 = is the major principal stress at failure
σ'_3 = is the minor principal stress
σ_c = is the uniaxial compressive strength of the intact rock
m s = are empirical constants.

Hoek (1983) has fitted both equations (1) and (2) to direct shear tests results on greywacke conducted by Martin and Miller (1974) and found that equation (2) fits the experimental data equally well as equation (1). Hoek (1983) has also related equation (2) to instantaneous c and Ø values which are normally quoted widely in the literature. Therefore for purposes of this discussion, the Hoek and Brown failure criterion is used as well as the linear c and Ø criterion, rather than other empirical curve fitting methods.

One of the major advantages of equation (2) is that it enables a curved failure envelope to be used which is generally more representative of actual laboratory test results. It can also be used for both intact samples as well as for discontinuities and does not over-estimate the strength of discontinuities at low stress levels. However this paper demonstrates in some instances, a linear c Ø fit is equally valid, particularly for the relatively low and narrow normal stress ranges that are often encountered in rock slope failures.

2 ROCK SLOPES

Before considering the problem of shear strength in detail, it is useful to examine how rock slopes actually fail, because this should then guide future laboratory test programmes and stability analyses.

Table I lists a range of typical rock slopes heights in Australia with the approximate maximum normal stress range. It can be seen that the maximum normal stress is only 9 MPa for deep open pit slopes, whereas for open strip coal mines the maximum normal stress is typically only 2 MPa. However, if the typical size of failure in such mines is considered rather than the overall slope height, the maximum normal stress is much less, normally less than 1 MPa, as shown in Table I. This only assumes simple overburden stresses and high horizontal stresses may cause higher figures than those shown in Table I. In addition, using Sarma's method of stability analysis (Sarma, 1979) it can be shown that the actual normal stresses on the failure plane can be different to those generated from simple overburden considerations alone. However, the basic premise remains that normal stresses within rock slopes are generally low.

TABLE I
MAXIMUM NORMAL STRESSES FOR TYPICAL
OPEN PITS IN AUSTRALIA

Rock Slope Type	Examples		Max Depth*	Approx Max Normal Stress
Deep Open Pit	Mt Newman	Overall slope	350m	9 MPa
		Double bench	30m	0.75 MPa
Open Pit **	Saxonvale, Koolan Island, Coronation Hill, Iron Monarch etc	Overall slope	100 - 200m	2.5 - 5 MPa
		Double bench	20 - 24m	0.5 - 0.6 MPa
Open Strip Coal	Most Bowen Basin Mines	Overall slope	60 - 100m	1.2 - 2 MPa
		Typical failure height	30m	0.6 MPa

* Planned or actual depth. There are obviously exceptions to this and much deeper pits exist (eg. Bougarville or Bingham Canyon). However even these only have maximum normal stress in the order of 25 MPa. Moreover, overall slope failures are much less frequent than bench scale failures. In this case a double bench failure has been used as a typical example.

** A considerable number of shallow open pit mines (less than 100m) also exists particularly for gold operations. In this case, the normal stress ranges are consequently lower.

Secondly, it is the experience of the writer that rock slope failures in Australia almost always occur where structure or discontinuities are 'daylighted' at or near the toe of the failure.

Examples of this range from simple undercut bedding on a batter face, to exposing predominant joint sets, to faults or shear zones occurring near the toe, or having an in-dipping stratigraphic contact exposed in a pit face. Some examples of this are listed in Table II and they range from failures in very strong quartzites at Koolan Island to highly weathered overburden materials at Riverside and Ora Banda.

TABLE II

EXAMPLES OF FAILURES CAUSED BY 'DAYLIGHTED' DISCONTINUITIES AT THE TOE

Mine	Location	Failure Type	Predominant Discontinuity Daylighting Near Toe
Koolan Island WA Iron Ore	Hanging Wall	Wedge	Two joint sets
Newman WA Iron Ore	South Wall	Complex bedding slides	Bedding
Saxonvale NSW Coal	East Wall	Large wedge	Bedding
Riverside Qld Coal	Highwall	Slide on in-dipping contact	Base of Paleochannel/ Shears
Ora Banda WA Gold	South Wall	Irregular wedge	Steep, joint set/ shear zone sub-parallel to face

It is interesting to note that even in very weak overburden materials at Riverside and Ora Banda (ie. UCS less than 5 MPa), failure is still controlled by weak discontinuities daylighting at or near the toe. All of these examples listed in Table II involve the exposure of some predominant weakness surface at or near the toe of the failure and therefore discontinuity controlled failures are not solely restricted to strong rocks.

Thirdly, stress analysis studies of open pit slopes (eg Stacey 1970) also clearly show that shear stresses are generated at the toe of a slope whereas tensile stresses are generated near the crest. Moreover failure surfaces near the crest of a slope are normally sub-vertical, normal stresses on them are low, movement is therefore tensile rather than in shear, and hence the roughness of the failure surface at the crest of the slope is of little significance. For this reason it is relatively simple for a failure surface to propagate near the crest of slope since only the tensile strength of the weakest link need be exceeded. A predominant discontinuity at the crest of a slope is therefore not normally required for failure to occur. Numerous examples of this are available at mines around Australia. For example, at Mt Newman, WA, many failures have occurred where tensile failure has developed in the upper part of the failure surface along a series of complex joint sets forming an extremely 'rough' failure surface near the crest of the slope.

However the opposite applies at the toe of a slope. Here normal stresses are relatively high, shear stresses are at a maximum and it is the writers experience that predominant, relatively smooth discontinuities are normally required for failure to occur. Failure through intact rock bridges is possible but unlikely given the relatively high intact strength of the rock compared to the relatively low shear stresses imposed on them.

These points are discussed further in relation to the Newman example which is a hard rock mine in complex geology, as well as in relation to the Saxonvale example which is a sedimentary coal mine in simple geology.

The Newman example is interesting because detailed information on previous failures is available and this can be compared directly to laboratory data.

Example 1: Mt Whaleback Iron Ore Mine, Newman, WA

Mt Whaleback is the largest single pit iron ore mine in the world and currently have plans to develop slopes up to 350m high. Slope failures have mainly occurred on the South Wall where the major pit slope development has taken place to date. Several of these failures have been back analysed and these are presented in Table III. It can be seen that all the slope failures listed except failure No 7, involve sliding along a predominant discontinuity at the toe, being either a bedding surface or a fault.

TABLE III

FAILURES ON MT WHALEBACK

	Horizon	Failure Mechanism	Dip Angle	Approx Date of Failure	c	ø
1	DG/MCR*	Bedding slide on synclinal keel and joints across bedding	?	Aug 77	35 - 50	18 - 21.5
2	MCR/BB*	Bedding slide on anticlinal nose with tension crack	35°	June 78	5 - 15	24 - 26.5
3	DG/MCR*	Flat bedding slide	34°	May 80	35 - 50	18 - 21.5
4	MCR/MCR*	Curved bedding slide	30 to 70°	Aug 81	5 - 15	24 - 26.5
5 6	BB/SYL*	Bedding slide on synclinal keel and joints across bedding	Variable	Oct 82	30 - 45	25.5 - 29.5
7	NOD	Toppling		Aug 84	-	-
8	MCR/MCR*	Flat bedding slide	39°	Sept 84	5 - 15	24 - 26.5
9	EFFZ*	Slide on in-dipping fault zone	Variable	March 87	5 - 10	25 - 30

* Note:

The 'Horizon' indicates the horizon in which the main sliding occurred. Where failure occurred between two stratigraphic horizons, then the upper horizon is listed first and the lower horizon is listed last. The abbreviations represent the following stratigraphic formations:

DG/MCR - Dales Gorge / Mt McRae Shale
MCR/BB - Mt McRae Shale / Bruno's Band
MCR/MCR - Mt McRae Shale / Mt McRae Shale
BB/SYL - Bruno's Band / Mt Sylvia Formation
NOD - Nodule zone in Mt McRae Shale
EFFZ - East Footwall Fault Zone

Back Analysis

The back analysis results listed in Table III are also presented in Fig 1 and the method of back analysis used is as follows.

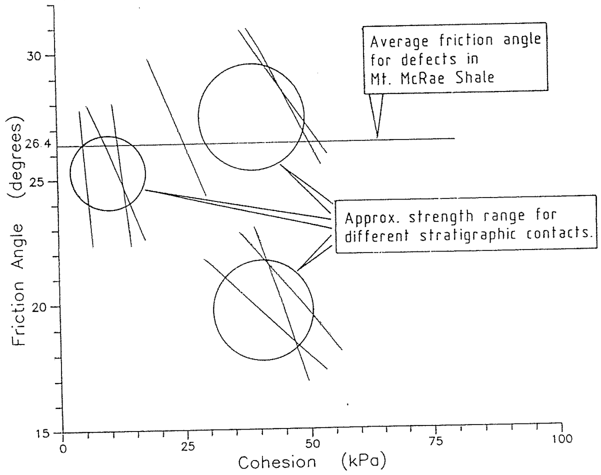

Figure 1 Back Analysis Results for Newman Slope Failures

The information required from a back analysis is to determine the shear strength along the failure surface at the moment of failure. This shear strength is most commonly defined in terms of c and Ø, and therefore there are two variables that can define the shear strength at any given normal stress. If we take the normal stress acting along an entire failure surface, it varies from zero at the two ends to some peak normal stress value approximately in the lower third of the failure surface. However, if c and Ø are varied in order to determine a factor of safety of 1.0, it is apparent that there are many combinations of c and Ø that will produce a factor of safety of 1.0 for the same failure surface. In this case the back analysis does not present a unique solution to the problem.

However if more than one failure has occurred on the same or a similar stratigraphic horizon, ie. the failure surface material properties are similar, then a back analysis can be undertaken for each failure and the results plotted together as shown in Fig 1. This figure shows a plot of c against Ø for nine different back analysis results from Newman. Where the curves in Fig 1 intersect, a unique combination of c and Ø values are given for that particular failure surface material type.

Figure 1 shows three main material types and the circles represent an approximate range in material strengths. It should be noted that the c and Ø curves have been truncated for clarity. Given the large number of variables between different failure surfaces, the results are in good agreement both with each other and with laboratory test results as shown.

One of the major rock types involved in these failures at Newman is the Mt McRae Shale and as a further check on laboratory test results for the Mt McRae Shale at Newman, the back analysis results have also been compared to them. Lilly (1982) presented the results of shear tests on Mt McRae Shale at Mt Tom Price and Paraburdoo in WA and stated that 'pit slopes in Mt McRae Shale were almost entirely controlled by the orientation and shear strengths of the bedding planes'. Lilly's shear test results for smooth bedding surfaces are presented in Fig 2 as c = 110 kPa, Ø = 24° (data points have been omitted for clarity).

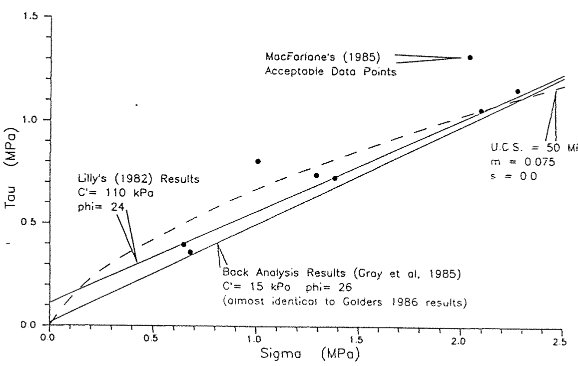

Figure 2 Shear Strength of Mt McRae Shale Bedding Surfaces

MacFarlane (1985) has also critically reviewed the large amount of shear test data available at Mt Newman and concluded that much of it is not relevant due to excessively high normal stresses during the shear test or other poor test procedures. However he demonstrated that where laboratory tests are undertaken at normal stresses similar to those existing in the field and that erroneous test results are not included, the laboratory results match the back analysed results very well. MacFarlane's acceptable data points are shown in Fig 2. It can be seen that Lilly's curve is a reasonable lower bound fit to MacFarlane's data between the normal stress range 0.5 - 2.5 MPa, but possibly overemphasises it in the normal stress range 0 - 0.5 MPa. It should be pointed out that Lilly (1982) recognised this and drew a curve down to c = 0 in this low stress range.

However if we compare the back analysed shear strength for failures along bedding surfaces in the Mt McRae Shale we get c = 15 kPa, Ø = 26° and this is an excellent lower bound fit to MacFarlane's data. A detailed investigation has also been undertaken by Golder Associates (Swindells et al 1986) to determine the shear strength of Mt McRae Shale bedding surfaces. After evaluating detailed surface roughness data they concluded that a curve fit almost identical to the back analysis results shown in Fig 2 was appropriate.

Finally, we can also fit a Hoek-Brown curve to this data as also shown in Fig 2. This curve is defined by parameters UCS = 50 MPa, m = 0.075, s = 0) and although it is a good average fit to the data, it may overestimate the strength of the actual failure surface, since failure in this instance appears to be governed by the lower bound laboratory strength rather than average strength. Therefore it is quite clear that the linear c Ø back analysed shear strength is applicable both to the in-situ strength and to the lower bound strength of the laboratory data.

Variability in Rock Strength

It is also enlightening to consider the natural variability in rock strengths which occur in a slope and as an example of this, the shear strength of rocks on the North Wall of Mt Whaleback are shown in Fig 3. These rocks basically comprise dolerites, shales and fault shale and fault gouge material and the laboratory results are the result of a very extensive test programme to determine the shear strength of rocks on the North Wall of Mt Whaleback (Gray and Preston (1987)).

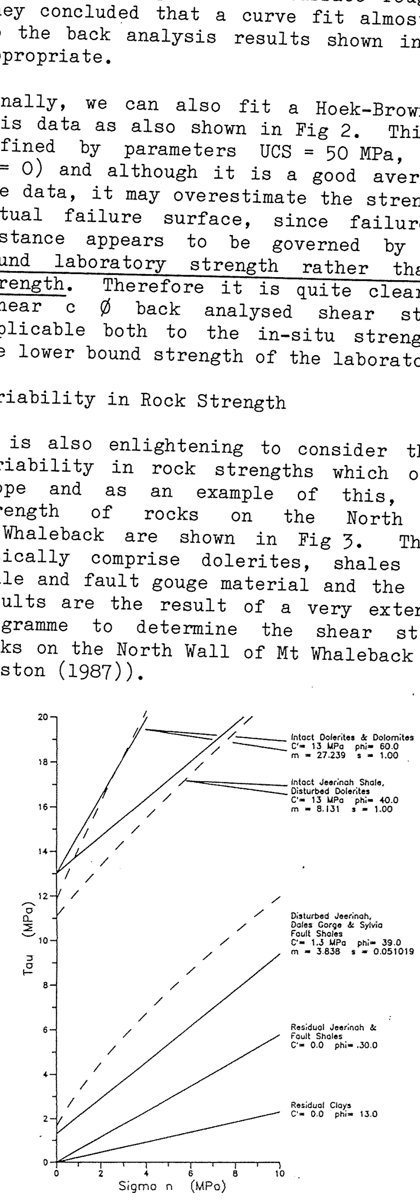

Figure 3 Variation in Material Strengths for Newman North Wall

This test programme included:

(i) conventional uniaxial and triaxial tests on intact rock core,

(ii) conventional uniaxial and triaxial tests on highly disturbed rock core which required very careful preparation and testing techniques,

(iii) 100mm x 100mm shear box tests on reconstituted fault shale material,

(iv) 300mm x 300mm shear box tests on discontinuities in rock blocks cast in plaster,

as well as a re-evaluation of previous Hoek shear box tests on discontinuities in rock core.

Materials tested included intact dolerites and shales, disturbed dolerites and shales, fault shale and clay. A failure has occurred within fault shale on the North Wall and his has been back analysed and shows almost identical strengths to those obtained from laboratory tests. This is shown as failure No.9 in Table III which gives back analysed results of c = 5-10 kPa, ϕ = 25-30°, whereas laboratory results are c = 0, ϕ = 30°. It should be noted that these laboratory tests on fault shale were shear box tests tested to <u>residual</u> values which can be expected to accurately represent fault shale which has undergone extensive shearing.

However the shear strengths of both <u>intact</u> and <u>disturbed</u> samples were determined by extensive <u>triaxial</u> testing and the results are shown in Fig 3. In this case the disturbed results refer to samples which were disturbed in the core either through extensive shearing and or intense folding. However they do not have a preferred weakness direction on a large scale, but are considered to be uniformly weak. Even these disturbed shale values have shear strength values of c = 1.3 MPa, ϕ = 39° in terms of a linear fit or UCS = 50 MPa, m = 3.838, s = 0.05 in terms of Hoek and Brown parameters. It can be seen that these strength values are considerably higher than those for fault shale (as shown in Fig 3) or for any of the back analysis results shown in Table III.

Moreover if the normal stress range that is applicable to actual slopes at Newman is applied to these disturbed shale results (ie. 0 - 9 MPa), it can be seen that a linear c ϕ fit produces a lower estimate of strength than a Hoek-Brown fit (Hoek, 1987). It also simplifies the strength values to be input in stability calculations.

Therefore it is logical that failure will preferentially occur along these pre-defined weakness surfaces (ie fault shale and clay) rather than through this disturbed shale material. In addition the shear strength of both the intact dolerite and shale is very high in comparison to the shear strengths of the disturbed shale, the fault shale and the back analysed results. Therefore the choice of either a linear or a curved fit to this data makes little difference to stability calculations. In practice failure through the intact rock substance will not normally occur except in the case of very small rock bridges.

Stability analyses of the Newman North Wall confirm that failure will preferentially occur through these weakest materials even though the overall dip of stronger materials is unfavourable for stability.

Example 2 : Saxonvale Open Pit Coal Mine
Hunter Valley, NSW

Introduction

Saxonvale is a one million tonne per annum, multi-seam open pit coal mine in the Hunter Valley of New South Wales and the interburden consists of sandstone, siltstone, conglomerate, mudstone, shale and tuff.

A large wedge failure has been developing on the East Wall at Saxonvale which is potentially up to 500,000 m^3 in size. It is interesting because this is one of the few failures which was both predicted and fully monitored prior to failure occurring. In addition, the dip as well as the location of the failure planes were known with a high degree of confidence.

East Wall Failure

The geotechnical investigations indicated that there were very weak clay and tuff layers present in the East Wall which were in-dipping into the pit and hence this wall was only marginally stable even in dry conditions. As a result of these investigations, part of the East Wall was re-orientated in order to minimise the size of the potential failure. In addition, a series of water monitoring bores and survey stations were installed along the crest of the East Wall in order to check water pressures and possible movements. (Gray et al (1983), Preston (1983), Preston and Wise (1984))

Figure 4 shows a general plan of the East Wall at Saxonvale with the water bores and survey stations marked. It can also be seen that the main waste haul road is located on the crest of the East Wall and hence there was serious concern about possible failure. The water monitoring bores and survey stations were installed in May 1984. In late July 1984 approximately 50mm of intense rain occurred and this initiated failure on the East Wall. Although such rainfall events are common, it is now known that a large area of the haul road itself was channelling water to the crest of the haul road.

However the initial failure was not dramatic with cracks developing at the crest of the East Wall. Since July 1984 a maximum of 400mm of horizontal movement has occurred. This movement has been progressive with periods of acceleration corresponding to extraction at the toe or periods of heavy rainfall (see Fig 5).

The geology comprises an in-dipping sedimentary sequence dipping at 12° with several weak clay layers occurring within it. Two of the most prominent weak layers are a clayey mudstone within the Vaux seam and a clay tuff in the Piercefield C seam. Failure of a small wedge initially occurred on the Piercefield C seam tuff. However the size of the failure has grown considerably since July 1984 and has a potential maximum size back to the sub-crop line of the Vaux seam (see Fig 4).

It is also interesting to note that a normal cross-section through the East Wall shows an apparent dip on the failure plane of only 7°, whereas the true dip angle is approximately 12°. Further it should also be noted that the initial movement of the East Wall was <u>normal</u> to the wall, ie. sliding at 7°, whereas <u>subsequent</u> movement was in the maximum dip direction. This is shown

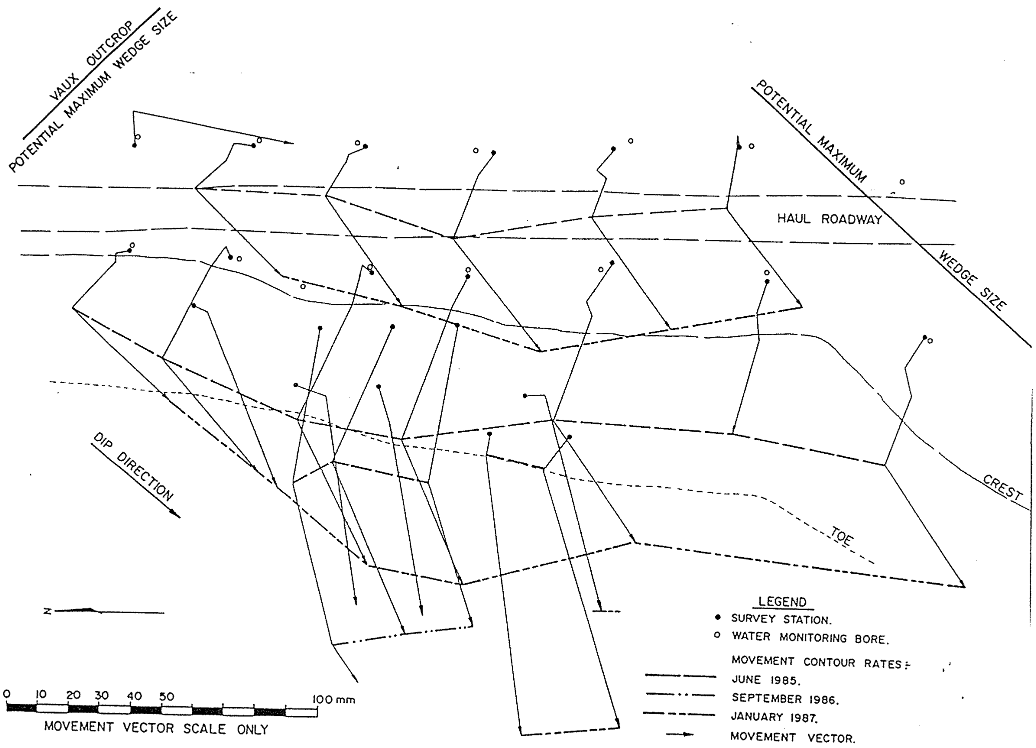

Figure 4 Plan of Saxonvale East Wall Failure

by the movement vectors plotted on Fig 4.

Laboratory Data

Extensive laboratory testing has been undertaken at Saxonvale and several weak layers have been identified in the East Wall. As mentioned previously, the Vaux and the Piercefield C seams contain the weakest materials and these were carefully sampled and tested in 100mm x 100mm shear boxes to determine both peak and residual shear strength parameters. The average laboratory results are as follows:

	Peak		Residual	
	c(kPa)	Ø	c(kPa)	0
Piercefield C tuff	16.2	17.2°	4.3	11.5°
Vaux Mudstone	15	18.6°	12.0	10.9°

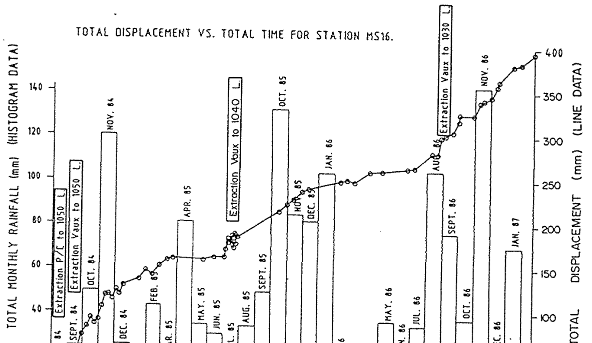

Figure 5 Movement Data vs Rainfall and Major Mining Events for Saxonvale East Wall

although some isolated samples of puggy clay had $\emptyset'_r$ values of only 8°. These results were initially used for design purposes but they also compare favourably with the back analysis results.

Back Analysis

The initial movement direction towards the north-west also corresponds to the maximum horizontal stress direction as determined by Enever and Wooltorton (1984). It is therefore postulated that, although it was kinematically possible for the East Wall failure blocks to initially move in the maximum dip direction, the failure blocks moved in a north-westerly direction partly in response to stress relief and partly in response to water pressures acting normal to the main joint direction, ie. acting in a northerly direction. In addition, movement in a north-westerly direction relieves any end-restraint and shear resistance on the down-plunge face of the moving wedge.

Because of the simple geometry of the East Wall, the only problem to solve in the back analysis is the magnitude of water pressures at the time of failure. Since the initial failure occurred after heavy rain and open tension cracks were visible at the crest after the failure it is reasonable to assume that water pressures were acting both along the tension crack and along the failure surface.

The back analysis results for Saxonvale are shown in Fig 6. If a section is taken in a north-westerly direction, ie. the initial direction of movement, and water pressure in a tension crack as well as uplift pressures are present, then the back analysis results almost exactly match the peak laboratory results (initial failure on Fig 6). Similarly, if a section is taken in the maximum dip direction and assuming no water pressures (present failure on

Fig 6), the back analysis results also are very close to the residual strengths obtained from the laboratory tests.

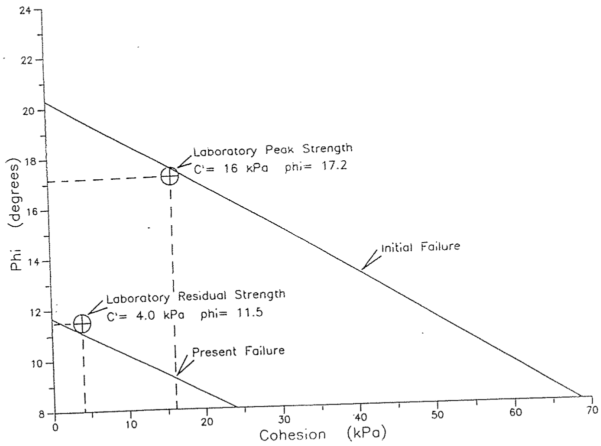

Figure 6 Back Analysis Results for Saxonvale East Wall

CONCLUSIONS

The shear strength of a range of rock slopes in Australia has been discussed. In particular it has been shown that the normal stress range existing within rock slopes is generally fairly low and that failure often occurs along some pre-existing weakness surface at or near the toe of the failure.

Despite the large variation in strength of rock slopes, a linear c Ø strength envelope appears to fit both laboratory and back analysis results for these weakness surfaces fairly well for the examples considered. This may be partly explained by the low normal stress range existing within these slopes. Nevertheless it may not always be necessary to determine the overall rock mass shear strength in order to accurately determine rock slope stability.

Finally, a simple back analysis procedure is presented which enables a unique definition of c and Ø parameters. Moreover since the frictional properties of a material are normally known with a higher degree of confidence than cohesion values, the technique enables the practising engineer to rapidly determine realistic in-situ cohesion values for rock slope failures. In this way the prediction of slope failures can more accurately match actual slope performance.

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