Application of Back Analysis to Hong Kong Landslides

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SYNOPSIS

Heavy rainfall in Hong Kong frequently causes landslides in the deeply weathered granite and volcanic rocks. For slope design, the aspects most poorly identified are representative shear strengths for the heterogeneous materials in the weathered profiles, and critical pore water pressures. Practical improvements in our knowledge of these can best be made from careful field studies of slope failures.

Three case studies of landslides are presented to illustrate the values of field studies in providing information on the mechanisms which control slope failure in Hong Kong. These demonstrate, however, that available data for complex, water-induced failures of the type that commonly occur will rarely be adequate to allow precise quantification of the various factors by back analysis.

INTRODUCTION

The topography, geology and climate of Hong Kong render it susceptible to landslides, and typically shallow landslides occur every year during seasonal tropical rainstorms. On steep natural slopes, such minor failures in the weak, weathered rock mantle during intense rainfall are considered part of the natural ageing of the youthful topography (Figure 1). Moreover, concentrated urban development has resulted in the formation of many large steep cuttings and fill slopes, and these are also prone to failure during intense rainfall. Such failures in the past have resulted in high loss of life and property damage (Lumb, 1975; Brand, 1982, 1984), and it is therefore of vital importance for designers to be able to avail themselves of accurate predictive methods. One way of improving prediction techniques is by critical examination of well-documented failures through the application of back analysis.

In 1982, Hong Kong experienced 3 248 mm of rain, the highest since records began in 1884. Two severe storms occurred in May and August which resulted in more than 1 500 landslides (Brand et al, 1984). On 29th May, 394 mm was recorded in 24 hours at the Royal Observatory, and 111 mm was measured in one hour at another location. On 16th August, the comparative figures were 362 mm and 95 mm. Eleven of the failures were selected for detailed study, partly because of their size and apparent significance, but also because good background data were thought to be available. Six of the failures actually had piezometers within or close to the failure scarps. The conclusions reached from these investigations are summarised in this Paper, and three of the case histories are discussed in order to illustrate the usefulness and difficulties of back analysis. Two other case studies have previously been described in detail (Hencher & Martin, 1984; Hudson & Hencher, 1984).

SLOPE DESIGN IN HONG KONG

Up until the early 1970s, slope design in Hong Kong was based on empirical rules which were progressively modified in the light of experience (Brand & Hudson, 1982). However, as a result of disastrous landslides in 1972 and 1976, and with the establishment of the Geotechnical Control Office in 1977, limiting equilibrium methods of slope analysis have become almost universally used for the design of new cuttings and fill slopes. Stability analyses are normally based on the results of intensive site investigation and laboratory testing programmes to provide input data on site ground conditions, groundwater response to rainfall, and material shear strengths (Geotechnical Control Office, 1984).

Despite the use of sophisticated analytical techniques and continuing efforts to improve the

Fig. 1 Clustered Landslides on Natural Hillsides

quality of site investigation and testing data, the available predictive methods have been demonstrated not to be sufficiently reliable in some circumstances. Steep slopes which have been predicted to be unstable have remained standing during some of the most intense rainstorms in recent history without signs of distress. Other slopes which had been designed on apparently sound assumptions have collapsed or undergone large and progressive displacements. Such occurrences throw doubts on our ability to identify materials adequately, to establish representative geological models, to quantify shear strengths, and to measure and predict critical pore water pressures for safe and economical slope design.

Table I summarises our state of knowledge in respect of slope stability for weathered profiles of the kind that exist in Hong Kong and identifies areas where major improvements are required.

ROLE OF LANDSLIDE STUDIES FOR ADVANCING KNOWLEDGE

There are only a few ways in which our knowledge of the key factors that control slope stability can be tested and advanced. Table II attempts to summarise these possible approaches, with particular reference to those areas most poorly understood at present.

The problem of predicting representative mass strengths to incorporate the effects of jointing and other heterogeneities is of particular note. For weathered profiles, laboratory measurements alone are unlikely to advance our knowledge significantly. Careful field observation combined with insitu tests is likely to be more fruitful, but even this approach is severely limited by practical considerations. The greatest potential for advancement is by the study of slope failures and by the application of back analysis in those cases where the contributory factors are sufficiently understood. This view is in sharp contrast to that of Sweeney & Robertson (1979), who emphasized the importance of laboratory testing rather than field studies as follows :

" *Research into the stability of residual soil slopes in Hong Kong is at a very early stage. Some recommended lines of further research are as follows :*

1 - *Effective stress testing at low stress levels to determine basic soil strength - some data has been presented in this paper but far more testing is required in order to develop a full understanding of the factors influencing effective stress shear strength parameters such as grading, density, fabric, structure, mineralogy, etc.*

2 - *A study of soil strength under plane strain conditions as triaxial testing may lead to significant underestimates of soil strength.*

3 - *A study of the effects of sample disturbance on soil strength.*

4 - *Suction measurements and the influence of suction on soil strength.*

5 - *Monitoring of groundwater levels in order to develop a better understanding of groundwater behaviour during and after heavy rainfall in a wide variety of situations.* "

Although the Authors are convinced that the post-failure investigation of landslides, which includes back analysis, is potentially the most fruitful means of advancing our knowledge in this area, they are mindful of the many pitfalls of the back analysis approach, as expounded by

TABLE I State of Knowledge of Aspects of Slope Stability for Hong Kong Conditions

Aspect	Current State of Knowledge for Hong Kong Conditions	Overall Rating of Knowledge
Methods of Stability Analysis	Janbu (1973) method of analysis for non-linear surfaces thought satisfactory. Recommended factors of safety of 1.2 to 1.4 are satisfactory (GCO, 1984). Experience has shown that data is often poorly handled (Lumsdaine & Tang, 1982).	very good
Geometry of Failure	Pre-failure geometry easily defined. Sometimes difficult to decide critical potential failure surface for design.	good to very good
Geology	Site investigation procedures adequate, but description often poor. Complex weathering profiles difficult to describe (Hencher & Martin, 1982). Poor understanding of influence of geological details on hydrogeology	fair
Shear Strength	Mass strength as distinct from SAMPLE strength poorly understood. Laboratory tests commonly used to determine saturated strengths of SAMPLES, in terms of effective stress, but doubt exists about applicability of test results (Brand, 1982). Limited amount of insitu strength testing carried out (Brand et al, 1983). Weakening effect of relict joints recognised (Koo, 1982(a) & (b), Harris, 1984). Effects of boulder and corestone content unknown (Hencher & Martin, 1982).	fair to poor
Groundwater	Useful correlations between landslides and rainfall (Lumb, 1975; Brand et al, 1984). Rapid changes in pore pressure with rainfall very difficult to predict for design (Anderson et al, 1983). Only limited attempts to model groundwater (Leach & Herbert, 1982). Extrapolation of insitu measurements seems best design approach (Endicott, 1982). Some progress with field instrumentation (Pope et al, 1982; Brand et al, 1983b). Importance of erosion pipes in transmitting water (Nash & Dale, 1983).	poor

TABLE II Scope for Advancing State of Knowledge within Poorly Understood Areas

Key Areas \ Methods	Field Observation	Testing: Field	Testing: Laboratory	Theoretical Studies	Back Analysis
Shear Strength	Advances can be made by observation of factors which influence the mass strengths of materials	Limited advances can be made in relating sample strengths to mass strengths through field testing	Advances can be made but only for small samples	Advances can be made in establishing models for the mass behaviour of materials	Only means of checking validity of relationships between sample and mass properties
Groundwater	Useful for checking theories and methods of prediction	Case studies relating subsurface profiles to hydrogeology and of infiltration characteristics, can lead to improved prediction	Nil	Advances are possible e.g. development of more sophisticated infiltration and hydrogeological models	Only means for checking that methods of incorporating groundwater into design are satisfactory

Leroueil & Tavernas (1981). The technique must be applied with care and the results interpreted with caution. The fundamental problem involved is always one of data quality. The sensitivity of a back analysis as a tool for testing a particular aspect of stability depends on the degree of confidence with which values may be assigned to the other parameters involved. In many cases in Hong Kong, where the groundwater pressures which trigger a landslide may be extremely transient, the degree of uncertainty that exists over this can make back analysis very unreliable as a test of real values of shear strength at failure. In these circumstances, 'worst case' conditions must be assumed to enable lower bound strengths to be determined, and only broad conclusions can be drawn from the analyses. These points will be emphasized by three case studies taken from the detailed landslide investigations of 1982.

CHAI WAN LANDSLIDE

Description

The landslide shown in Figures 2 & 3 occurred during intense rainfall in the early morning of 29th May 1982. A 15 m high portion of the 50° slope collapsed suddenly, resulting in approximately 1 000 m³ of debris. The case was selected for study largely because the unfailed slope had been previously investigated and

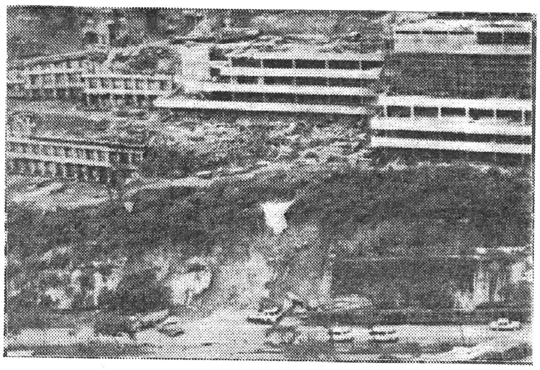

Fig. 2 General View of Chai Wan Landslide

several piezometers had been installed at that time near the rear scarp of the failure that subsequently occurred. A schematic section of the slope is given in Figure 4.

Quality of Data

The geometry of the failure scarp was established by a detailed 1:100 survey, and the original

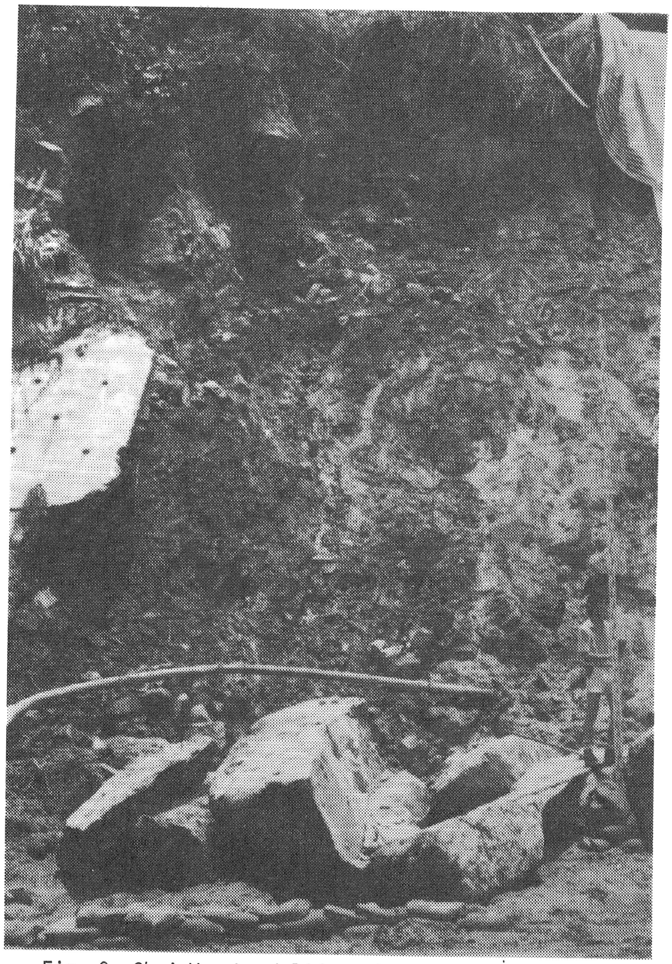

Fig. 3 Chai Wan Landslide under Investigation

Factors of Safety

Trial	colluvium c'	colluvium ø'	volcanics c'	volcanics ø'	piezometric surfaces 1	2	3	4	5	6
A	12	35	12	36	0·65	0·72	0·79	0·81	0·92	1·00
B	50	58	11	32·5	0·82	0·92	1·01	1·03	1·14	1·23
C	0	80	11	32·5	0·59	0·70	0·79	0·80	0·93	1·02

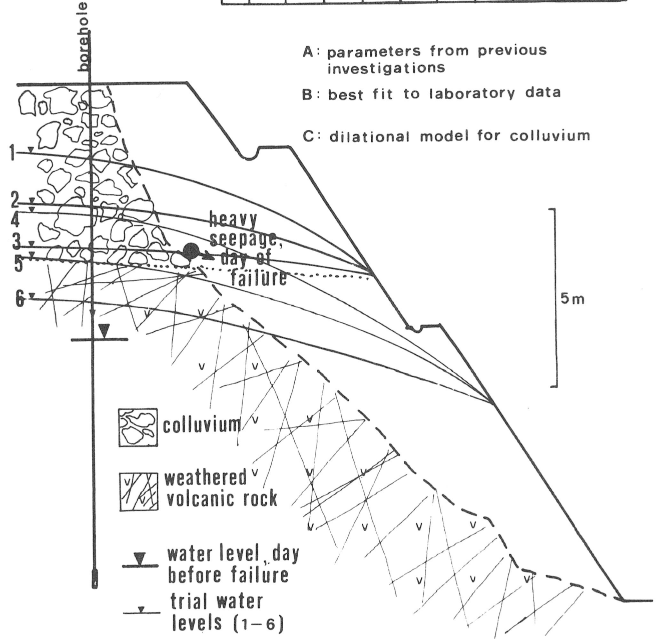

Fig. 4 Schematic Cross-section of Chai Wan Landslide

slope geometry was determined by reference to previous drawings and photographs. Examination of the failure scarp confirmed prefailure interpretations of borehole data, in that the geology of the site was reasonably simple, with bouldery colluvium overlying closely-jointed (25 mm in places) highly decomposed volcanic rock.

Block samples of the volcanic material and the colluvial matrix were taken to the laboratory, and their strengths were determined by direct shear and triaxial tests. The tests gave values reasonably close to prefailure determinations. It was considered that the laboratory-derived strength of the highly decomposed volcanic rock might reasonably be taken as representative of the jointed volcanic mass. It was recognised, however, that because the colluvium contained about 50% angular boulders and cobbles, its mass strength was likely to be very different from the matrix strength measured in the laboratory. A theoretical approach was therefore used to estimate the mass strength of the colluvium, by supplementing the matrix strength by a minimum dilation angle calculated for boulders to override one another. Dilation angles were determined from measurements along trial failure surfaces drawn through idealised cross-sections which contained the correct percentage, size and angularity of boulders and cobbles.

The other unknown in the landslide analysis was the groundwater pressures at failure, despite the presence of one of the piezometers very close to the crest of the slope. This was due to two main factors :

(i) Observed major seepage points soon after the failure demonstrated that groundwater flow through pipes in colluvium was important to the hydrogeology of the area. The piezometers, however, were embedded deeply within the volcanic material and were therefore unable to measure the level of groundwater perched within the colluvium.

(ii) The piezometers were only read manually on a daily basis, and the maximum transient values, that might have been recorded by 'buckets' or some other automatic measuring device (Brand et al, 1983), were not observed because failure occurred during intense rain at 3 a.m.

Back Analysis

Back analyses were carried out using Janbu's simplified and rigorous methods (Janbu, 1973) for six different piezometric surfaces and combinations of likely effective strength parameters for the decomposed volcanic rock and colluvium. The various assumptions made, together with selected results, are illustrated in figure 4.

The results show that, for each assumed piezometric surface, a factor of safety of 1.0 could be obtained by using one or other of the 'acceptable' strength combinations for the colluvium and decomposed volcanics. It was therefore impossible to evaluate the strength of either material or to ascertain the probable groundwater levels at failure.

Conclusions

This case study illustrates the difficulties inherent in back analysis despite apparently good data. The study itself was useful in indicating and emphasizing a number of research areas of importance to investigation and design, namely: (a) the contribution of boulders to shear strength, (b) the role of natural pipes in carrying groundwater, and (c) the importance of matching field instrumentation to the geological structure to ensure that meaningful data are obtained. The quality of the available data, however, would not permit adequate evaluation of either material strengths or peak water levels.

Fig. 5 Junk Bay Landslide

Factors of Safety

Trial	kaolin		Piezometric surface			
	c'	φ'	1	2	3	4
A	7	27.5	0.49	0.67	0.92	1.05
B	31	27.5	1.05	1.23	1.45	1.52
C	243	24.5	4.84	5.02	5.20	5.28

A : pre-existing kaolin joint (data from other study)

B : pre-existing kaolin joint (this study)

C : failure through intact kaolin (this study)
IV / V granite c' 10 kPa , φ' 39.5°
VI granite c' = 5 kPa , φ' = 32°

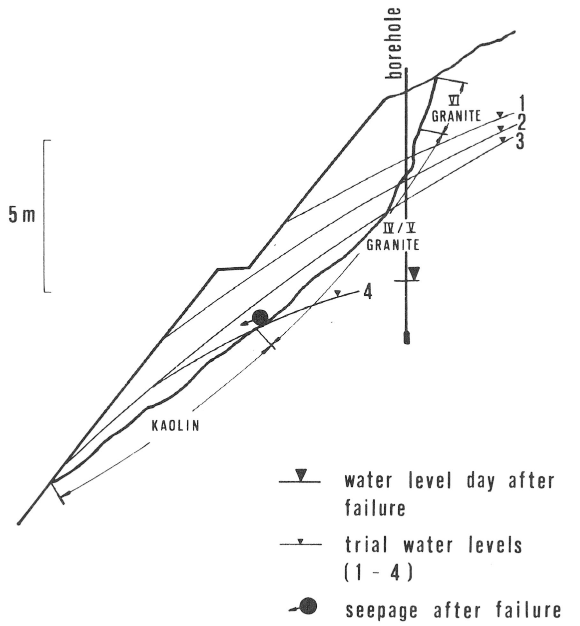

Fig. 6 Schematic Cross-section of Junk Bay Landslide

JUNK BAY LANDSLIDE

Description

The major failure at Junk Bay occurred in a 20 m high portion of a slope cut at approximately 50° and resulted in about 1 300 m^3 of debris. As in the Chai Wan case, the slope had previously been investigated and instrumented, which made it a potentially rewarding case for study. A fire had occurred at the crest in November 1981, leaving ideal conditions for rapid infiltration and ponding of water. A view of the failure is shown in Figure 5, and a cross-section is given in Figure 6 which shows the major features.

Quality of Data

The geometry was easily defined in the same way as for the Chai Wan study, and close inspection of the failure surface quickly revealed a likely mechanism of failure. Joints infilled with thick kaolin daylighted about half-way down the face and covered much of the lower failure surface. Furthermore, distinct seepage flows were observed from above the kaolin. The upper part of the failure passed through decomposed granite of grade VI (residual soil) to grade IV (highly decomposed rock).

Block samples of granite were taken for laboratory shear strength testing. Kaolin samples were also taken, and these were tested intact and along joints within the kaolin. The results of the tests appeared reasonable, and they confirmed the values obtained on samples from the site tested during pre-failure investigations and for similar materials elsewhere.

Although there were two piezometers close to the failure crest, these gave distinctly different readings. A standpipe very close to the crest had recorded a water level well below the post-failure observed seepage from the face, even though it was equipped with 'buckets' to record the peak water level. The levels from the more distant piezometer, although higher, were still not as high as the observed groundwater in the face of the failure scarp. It was therefore concluded that, despite the good instrumentation, the deep locations of the piezometers resulted in their not being able to measure the perched groundwater above the thick kaolin joints.

Back Analysis

For this study, the unknowns were not quite as extreme as for the Chai Wan case. Strength parameters from laboratory testing were considered applicable to the failure surface. Groundwater was still an unknown factor, but sensitivity analysis could be carried out within a narrow range to examine its influence on stability.

Analyses were carried out using Janbu's rigorous and simplified methods. Selected results are presented in Figure 6, from which it can be concluded that failure through intact kaolin can be dismissed. The alternative postulation, that failure occurred by sliding along a pre-existing joint, is more acceptable, although the results are sensitive to the value of cohesion assumed.

The back analyses showed that the factor of safety was only slightly affected by changes in the strength parameters of the granite within the range of likely values. It was found, however, that the calculated factor of safety was highly sensitive to the water level and to the cohesion on the kaolin joint. Unique solutions could not be derived.

Conclusions

As for the Chai Wan study, it was concluded that the quality of data was not high enough to allow back analysis to be used as a sensitive test, despite the fact that piezometers had been installed prior to failure. However, the benefits of landslide studies are again demonstrated, in that the importance of geological complexity in controlling mass strength and hydrogeology was clearly established. In addition, this case highlighted once more the inherent difficulties in correctly locating instruments for recording groundwater pressures.

CHUNG HOM KOK LANDSLIDE

Description

The minor landslide shown in Figure 7 was chosen for the study on account of its apparent simplicity. The geometry was uncomplicated, and the

Fig. 7 Aerial View of Chung Hom Kok Landslide

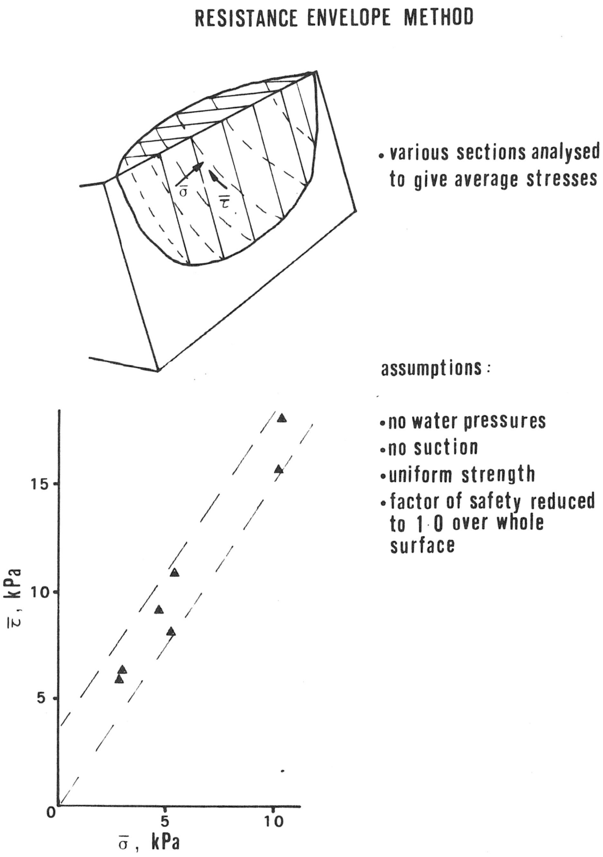

Fig. 8 Analysis by Resistance Envelope Method

failure plane passed through seemingly homogeneous decomposed granite. Field observations and the geomorphology of the slope indicated that it might be reasonable to disregard groundwater pressure, and the failure therefore seemed eminently suitable for back analysis to provide shear strength parameters for the decomposed granite of the region.

Resistance Envelope Method

A modified version of the resistance envelope technique of Janbu (1954) was first used for the analysis. Average stresses were calculated for several cross-sections, as shown in Figure 8. On the assumption that the factor of safety was 1.0 on each section, a plot of average shear stress against average normal stress would then represent the failure envelope for the material. On the basis of seven sections, however, this gave a friction angle of almost 60 degrees and a cohesion intercept of only 0 to 5 kPa (Figure 8). A number of major assumptions which, if wrong, might invalidate this result are listed in Figure 8. The field examination showed that the assumption of uniform strength, in particular, could not be justified, and an alternative method of analysis was therefore necessary.

Sliding Block Method

Field descriptions based on index tests showed that the materials within the failure scarp could be split into two main zones. The upper layer, which extended from the top of the slope to a change in gradient on the failure plane, gave no rebound to a Schmidt hammer and was otherwise considerably weaker than the underlying, less decomposed granite which gave average rebound values of 12. Microfracturing was intense throughout the granite, making it difficult to obtain undisturbed samples and therefore impossible to measure strengths in the laboratory. Vertical joints, which were particularly evident in the lower part of the scarp, may have played a part in the failure.

To take account of this heterogeneity, the central section of the failure was analysed first as a single block and later as two interactive

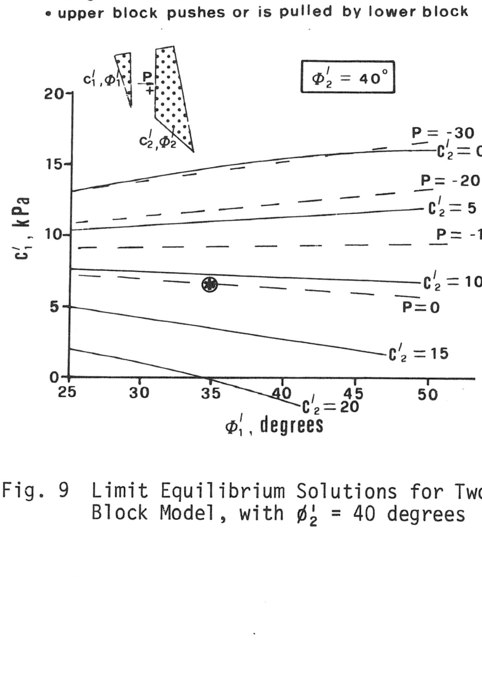

Fig. 9 Limit Equilibrium Solutions for Two-Block Model, with $\phi_2' = 40$ degrees

blocks. The latter model was the most flexible, allowing interslice forces and different strength parameters to be taken account of for the two layers.

The results obtained for an assumed friction angle (ϕ_2') of 40 degrees for the lower layer are given in Figure 9. It can be seen that there are many possible sets of strength parameters for the decomposed granite. For example, for a zero interslice force (P = 0), the cohesion of the lower layer would be about 11 kPa, and possible parameters for the upper layer might be $c_1' = 7$ kPa and $\phi_1' = 35$ degrees.

Conclusions

This case illustrates that, even for a simple landslide, the number of solutions by back analysis is many. This is particularly striking in this case, since the results presented here were only for the specific case of no groundwater; different assumptions would have altered the results considerably.

CONCLUSIONS

The case studies presented here demonstrate that, despite the unrivalled potential of back analysis for testing mechanisms of failure and for deriving average shear strengths at failure, seldom does the available data allow precise conclusions to be drawn. This is even true of apparently well investigated and instrumented slopes.

Despite these reservations about back analysis, landslide studies, particularly if carried out immediately after the failures have occurred, can provide many insights into slope stability in a qualitative sense. In all eleven major landslide studies carried out in Hong Kong in 1982, post-failure examinations provided satisfactory explanations for their occurrence, even though quite complicated models were required in some cases to explain all the observed features (Hudson & Hencher, 1984). It might be emphasized that, for weathered profiles of the kind encountered in Hong Kong, many failures can be attributed to complex hydrogeological conditions which cannot generally be accounted for in stability analyses.

ACKNOWLEDGEMENTS

This paper is published with the permission of the Director of Engineering Development, Hong Kong Government.

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