A slope stability problem in Hong Kong

Introduction

In the early 1970s a series of landslides occurred in Hong Kong as a result of exceptionally heavy rains. These slides caused some loss of life and a significant amount of property damage. Consequently, an extensive review was carried out on the stability of soil and rock slopes in the Territory.

During this review, a rock slope on Sau Mau Ping Road in Kowloon was identified as being potentially unstable. The stability of this particular slope was critical because it was located immediately across the road from two blocks of apartments, each housing approximately 5,000 people.

Figure 1 gives a general view down Sau Mau Ping Road, showing the steep rock slopes on the left and the apartment blocks on the right.

The concern was that a major rock slide could cross the road and damage the apartment blocks. In order to decide upon whether or not the residents of the two apartment blocks should be evacuated, the two questions which required an immediate response were:

What was the factor of safety of the slope under normal conditions and under conditions which could occur during an earthquake or during exceptionally heavy rains associated with a typhoon?

What factor of safety could be considered acceptable for long term conditions and what steps would be required in order to achieve this factor of safety?

Description of problem

The rock mass in which the slope adjacent to the Sau Mau Ping Road was cut is unweathered granite with exfoliation or sheet joints similar to those illustrated in Figure 2. These joints are parallel to the surface of the granite and the spacing between successive joints increases with increasing distance into the rock mass. Undercutting of these sheet joints can cause a rock slide such as that illustrated in Figure 3.

During excavation of the original slopes for the Sau Mau Ping Road, a small rock slide was induced by blasting. The surface on which this failure occurred is illustrated in Figure 4. Blasting, such as that used in civil construction in an urban environment, does not impose very large loads on rock slopes and it can be assumed that the factor of safety of the slope was close to unity.

Figure 1: A view down Sau Mau Ping Road in Kowloon showing apartment blocks across the road from the steep rock slopes.

Figure 2: Sheet jointing in granite. These features, sometimes referred to as ‘onion skin’ joints, are the result of exfoliation processes during cooling of the granite.

Figure 3: A rock slide on a road caused by the undercutting of sheet joints in a granite slope. In hard rocks such as granite, failure can occur very suddenly if the factor of safety of the slope is close to 1. A rise in groundwater levels during a heavy storm or ice jacking in winter may be sufficient to induce failure.

Figure 4: The failure surface defined by a sheet joint surface on which a small slide occurred during blasting of the original cut slope for the Sau Mau Ping Road. The potentially unstable slope under consideration is visible in the back-ground.

The potentially unstable slope under consideration is visible in the background of this photograph. It is obvious from this photograph that the sheet joint surface continues under the potentially unstable slope. Hence, from the evidence of the small scale failure, it can be deduced that the factor of safety of the slope in question is not very high.

The geometry of the slope is illustrated in Figure 5 which shows a 60 m high slope with three 20 m high benches. The overall slope angle is 50° and the individual bench faces are inclined at 70° to the horizontal. An exfoliation joint surface dips at 35° and undercuts the slope as shown in the figure. The slope face strikes parallel to the underlying exfoliation surface and hence the slope can be analysed by means of a two-dimensional model.

Tension cracks are frequently observed behind the crest of slopes which have a factor of safety of less than about 1.2. These cracks are dangerous in that they allow water to enter the slope at a particularly critical location. Unfortunately, in the case of the Sau Mau Ping slope, recently cultivated market gardens located on the top of the slope made it impossible to determine whether or not such tension cracks were present and hence it was decided to carry out two sets of analyses - one with and one without tension cracks. These analyses were carried out for both the overall slope and for individual benches.

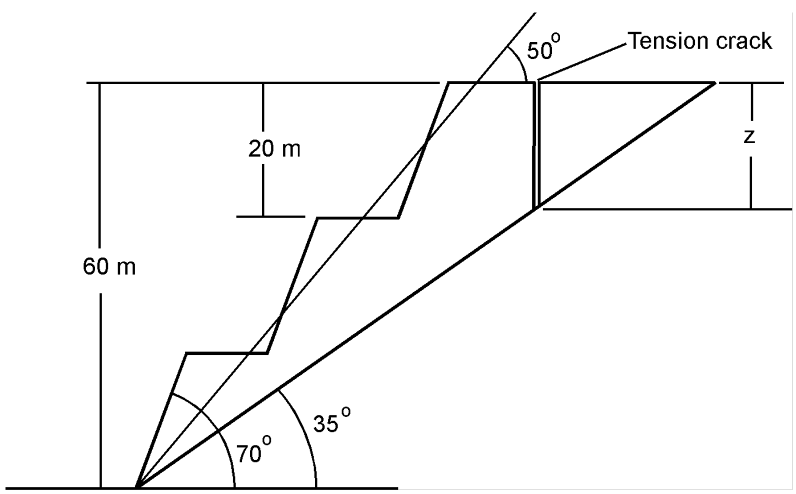
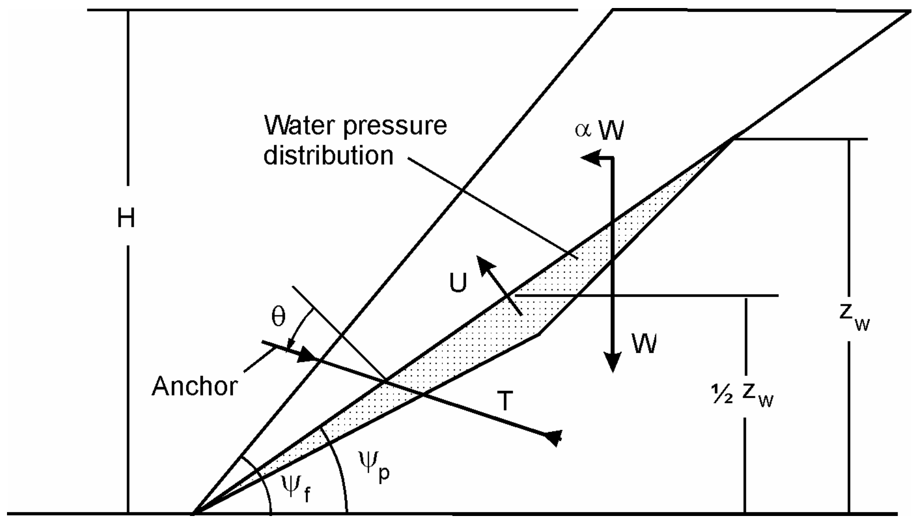


Figure 5: Geometry assumed for the two-dimensional analysis of the Sau Mau Ping Road slope.

Limit equilibrium models

At the time of this investigation, no rock mechanics facilities existed in Hong Kong and no diamond drilling or laboratory testing had ever been carried out on the granitic rocks in which this slope had been excavated. Consequently, the problem was tackled on the basis of a crude form of risk analysis, using simple analytical models to predict the response of the slope to a range of possible conditions. The two models are defined in Figure 6 and Figure 7.



$$F = \frac{cA + \left(W(\cos\psi_p - \alpha\sin\psi_p) - U + T\cos\theta\right)\tan\phi}{W(\sin\psi_p + \alpha\cos\psi_p) - T\sin\theta} \tag{1}$$

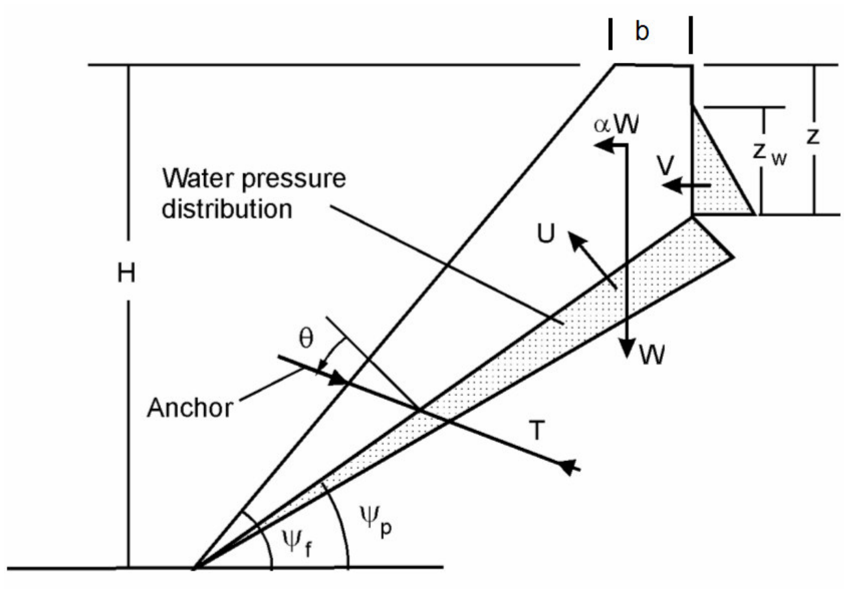
where

$$A = \frac{H}{\sin\psi_p} \tag{2}$$

$$W = \frac{\gamma_r H^2}{2}(\cot\psi_p - \cot\psi_f) \tag{3}$$

$$U = \frac{\gamma_w H_w^2}{4\sin\psi_p} \tag{4}$$

Figure 6: Factor of Safety calculation for a slope with no tension crack.



$$F = \frac{cA + \left(W(\cos\psi_p - \alpha\sin\psi_p) - U - V\sin\psi_p + T\cos\theta\right)\tan\phi}{W(\sin\psi_p + \alpha\, con\,\psi_p) + V\cos\psi_p - T\sin\theta} \tag{5}$$

where

$$z = H\left(1 - \sqrt{\cot\psi_f \tan\psi_p}\right) \tag{6}$$

$$b = H\left(\sqrt{\cot\psi_f \cdot \cot\psi_p} - \cot\psi_f\right) \tag{7}$$

$$A = \frac{H - z}{\sin\psi_p} \tag{8}$$

$$W = \frac{\gamma_r H^2}{2}\left(\left(1 - \left(\frac{z}{H}\right)^2\right)\cot\psi_p - \cot\psi_f\right) \tag{9}$$

$$U = \frac{\gamma_w z_w A}{2} \tag{10}$$

$$V = \frac{\gamma_w z_w^2}{2} \tag{11}$$

Figure 7: Factor of Safety calculation for a slope with a water-filled tension crack.

The Symbols and dimensions used in these models are as follows:

Symbol	Parameter	Dimensions
F	Factor of safety against sliding along sheet joint	Calculated
H	Height of the overall slope or of each bench	60 m or 20 m respectively
ψf	Angle of slope face, measured from horizontal	50°
ψp	Angle of failure surface, measured from horizontal	35°
b	Distance of tension crack behind crest	Calculated (m)
z	Depth of tension crack	Calculated (m)
zw	Depth of water in tension crack or on failure surface	Variable (m)
α	Horizontal earthquake acceleration	0.08 g (proportion of g)
γr	Unit weight of rock	0.027 MN/m3
γw	Unit weight of water	0.01 MN/m3
W	Weight of rock wedge resting on failure surface	Calculated (MN)
A	Base area of wedge	Calculated (m2)
U	Uplift force due to water pressure on failure surface	Calculated (MN)
V	Horizontal force due to water in tension crack	Calculated (MN)
c	Cohesive strength along sliding surface	Variable (MN/m2)
ϕ	Friction angle of sliding surface	Variable (degrees)
T	Force applied by anchor system (if present)	Specified (MN)
θ	Inclination of anchor, anti-clockwise from normal	Specified (degrees)

Note that this is a two-dimensional analysis and these dimensions refer to a 1 metre thick slice through the slope. It is also important to recognise that this analysis considers only force equilibrium and assumes that all forces pass through the centroid of the wedge. In other words, moment equilibrium is not considered in this analysis. While this is a simplification of the actual situation depicted in Figure 6 and Figure 7, the errors introduced are not considered to be significant, given the uncertainty of the other input data used in these analyses.

In Figure 7 the depth z of the tension crack is calculated by equation 6. This equation is obtained by minimising equation 5 with respect to the tension crack depth z (Hoek and Bray, 1974). This minimisation is carried out for a dry slope and the accuracy of equation 6 decreases as the water depth in the tension crack increases. However, for the purposes of this analysis, the estimate given by equation 6 is considered acceptable.

Estimates of shear strength

One of the most critical steps in any limit equilibrium analysis is the determination or the estimation of the shear strength parameters (c and ϕ) for the surface along which it is anticipated that sliding will take place. In the case of this slope on Sau Mau Ping Road, no information on shear strength was available at the time of the initial studies and so estimates had to be made on the basis of published information for similar rocks.

Hoek and Bray (1974) published a plot, reproduced in Figure 8, of cohesive strengths and friction angles for rocks and soils, based upon the results of published back analysis of slope failures. Superimposed on this plot is an elliptical zone which encompasses the estimated range of shear strength for sheet joints in unweathered granite. In choosing this range it was considered that the friction angle ϕ probably ranges from 30° for very smooth planar surfaces to 45° for rough or partly cemented surfaces. The cohesive strength c is more difficult to estimate and the range of 0.05 to 0.2 MPa was chosen on the basis of the results of back-analyses of slope failures, plotted in Figure 8.

Some readers may be surprised that a cohesive strength has been assumed for joint surfaces which obviously have no tensile strength or 'stickiness' as would be found in a clayey soil. In fact, this assumed cohesive strength is defined by the intercept, on the shear strength axis, of a tangent to a curvilinear Mohr envelope. This curvature is the result of the interlocking of asperities on the matching surfaces of the joints and the increase in shear strength given by this interlocking plays a crucial role in the stability of slopes such as that under consideration in this chapter.

Estimate of earthquake acceleration

Hong Kong is not considered a highly seismic region but relatively minor earthquakes are not unknown in the region. Consequently, it was felt that some allowance should be made for the possible influence of earthquake loading on the stability of the Sau Mau Ping slope.

The traditional method of incorporating the acceleration induced by earthquakes or large blasts in slope stability analyses is to add an outward force αW to the forces acting on the slope (see Figure 6 and Figure 7), where α is the acceleration as a proportion of g, the acceleration due to gravity. This 'pseudo-static' form of analysis is known to be very conservative but, in the case of the Sau Mau Ping slope, this conservatism was not considered to be out of place.

In discussion with local engineers and geologists, the consensus opinion was that the horizontal acceleration which could be induced by a 10 year return period earthquake in the region would be approximately 0.08 g. This value was used in all of the sensitivity analyses discussed in the following sections.

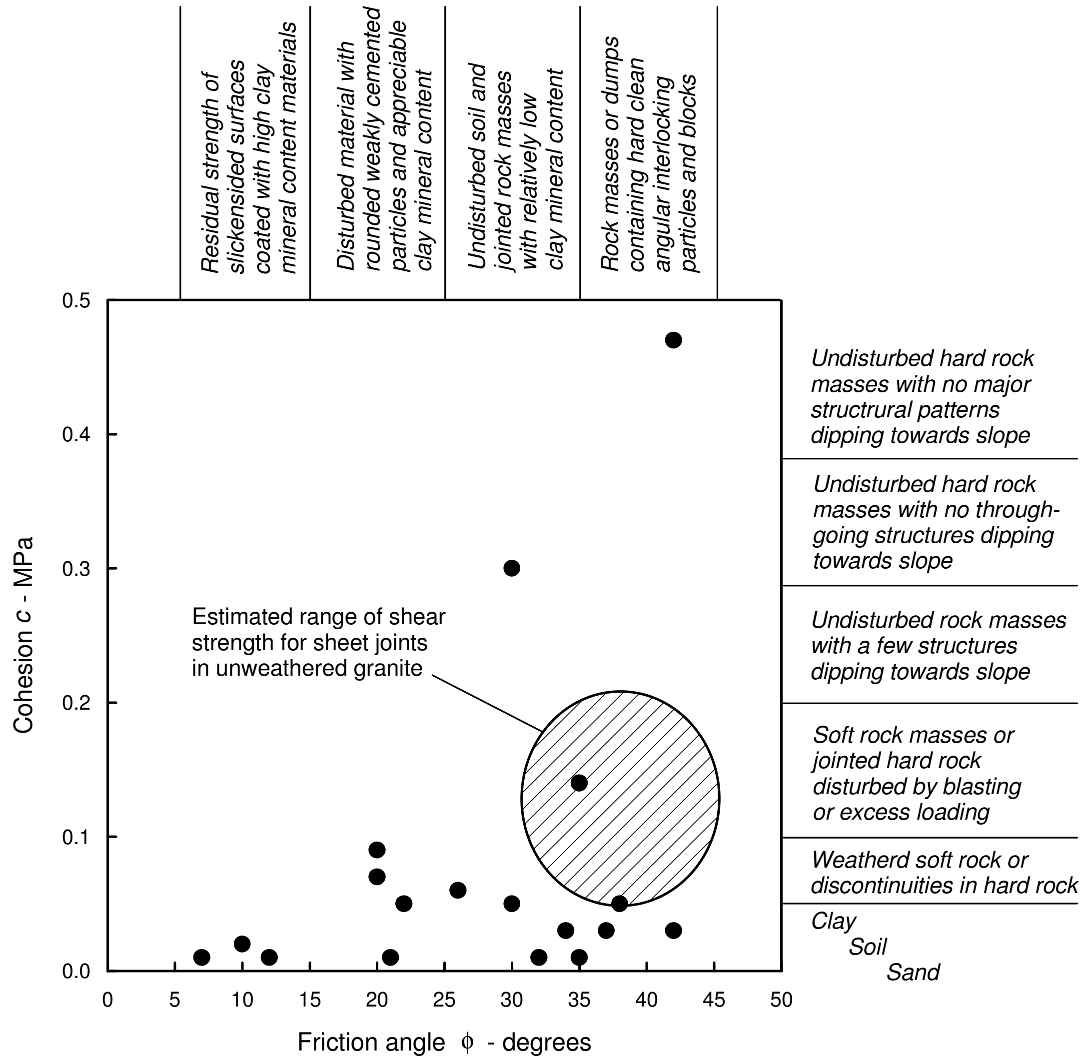


Figure 8: Relationship between friction angles and cohesive strengths mobilised at failure of slopes in various materials. The plotted points were obtained from published information from the back analysis of slope failures. (After Hoek and Bray 1974).

Analysis of mobilised shear strength

One method for assessing the stability of slopes is to calculate the shear strength that would be mobilised at failure and to compare this strength with the shear strength which is available along the failure surface. In the case of the Sau Mau Ping slope, this was done by substituting F = 1 in equations 1 and 5 and solving for the cohesive strength c and the friction angle ϕ. The results of this analysis are plotted in Figure 9. The estimated range of available shear strength (from Figure 8) is also shown on this plot.

Figure 9 shows that only two of the cases analysed result in conditions where the shear strength mobilised at failure falls within the estimated range of available shear strength. These two cases are designated 2 and 4 and they are for fully saturated slopes, with and without tension cracks.

Decision on short-term stability of the Sau Mau Ping slope

From the results of the sensitivity study described above it was concluded that instability of this particular slope could occur if the slope was fully saturated and subjected to earthquake loading. Typhoons occur several times every year in Hong Kong and the intensity of precipitation during these events is certainly sufficient to saturate the slopes. As discussed earlier, minor earthquakes do occur in the region but they are not very frequent. Consequently, the chance of simultaneous saturation and earthquake loading was considered to be small and in was concluded that there was no serious short-term threat of instability of the Sau Mau Ping slope.

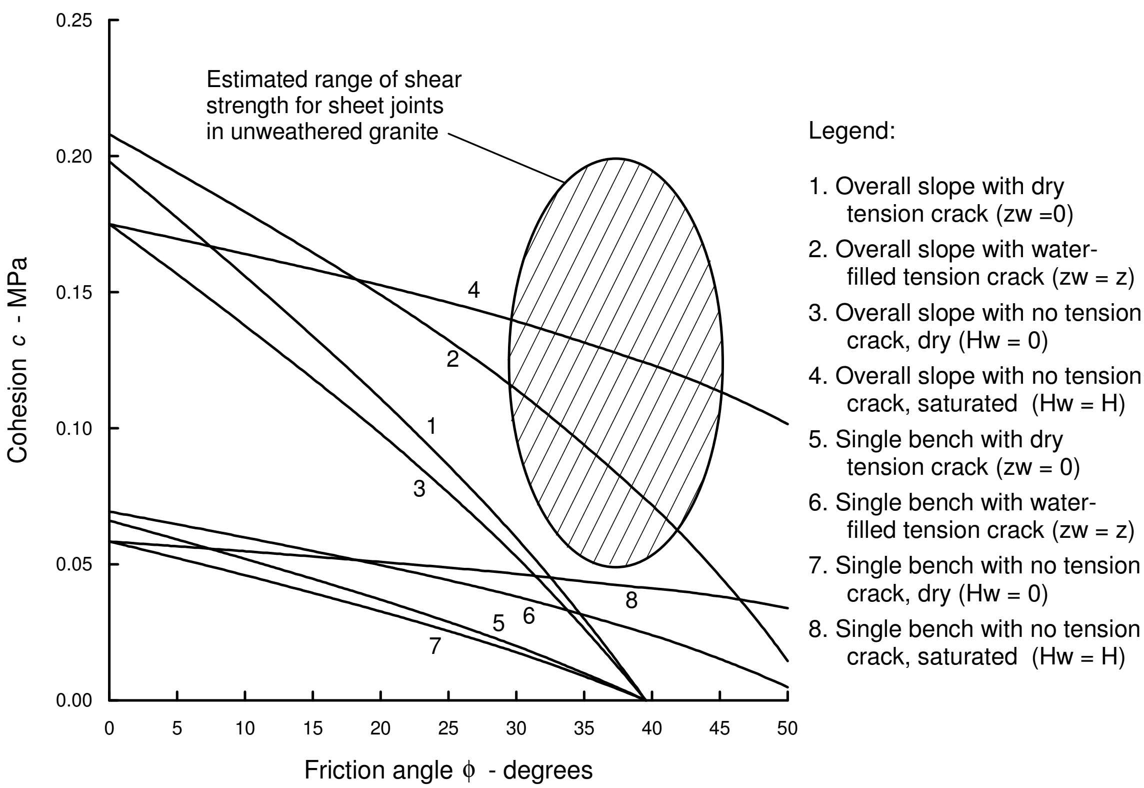


Figure 9: Comparison of the shear strength mobilised by failure under various conditions with the estimated shear strength available on sheet joints in unweathered granite.

In discussion with the highway authorities in Hong Kong, the following decisions were made:

No evacuation of the residents of the two apartment blocks, located across the street from the slope in question, would be carried out.
Horizontal drainage holes would be drilled into the slope face to penetrate the potential failure surface in an attempt to reduce uplift pressures in the slope.
Piezometers would be installed in holes drilled from the top of the slope. These piezometers would be measured regularly during periods of significant rainfall and the road would be closed to traffic if water levels rose to levels decided by the engineers responsible for the project.

An investigation would be carried out into the most effective remedial measures to stabilise the slope for the long-term.

Figure 10 shows the drilling of the horizontal drain holes into the slope face and Figure 11 shows the drilling of the vertical holes into which the piezometers were installed. These piezometers were monitored for the next few years, while preparations for the final stabilisation of the slope were made, and the road was closed to traffic on two occasions when water levels were considered to be dangerously high.

Figure 10: Drilling horizontal drain holes into the face of one of the benches of the Sau Mau Ping slope.

Figure 11: Drilling vertical diamond core holes into the Sau Mau Ping slope. These holes were used for geotechnical investigation purposes and also for the installation of piezometers in the rock mass.

Evaluation of long-term remedial measures

While the short-term threat of instability was considered to be small, the longer-term stability of the slope was considered to be unacceptable and a study was carried out to evaluate various options for stabilising the slope. It was agreed that a factor of safety of 1.5 was required to meet long term requirements. The following alternatives were considered:

1. Reducing the height of the slope.
2. Reducing the angle of the slope face.
3. Drainage of the slope.
4. Reinforcement of the slope.

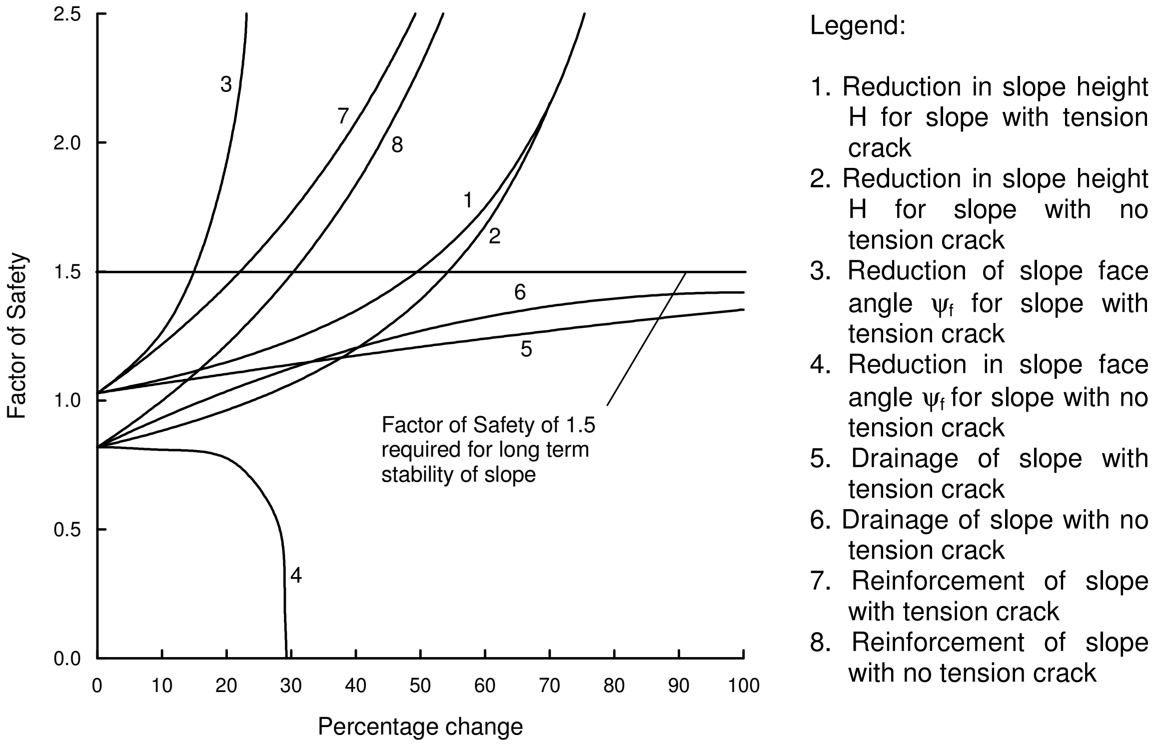


Figure 12: Evaluation of remedial options to increase the stability of the slope

The limit equilibrium models defined in Figure 6 and Figure 7 were used for this evaluation and the results are plotted in Figure 12.

In calculating the factors of safety shown in this figure, the shear strength was maintained constant and was defined by c = 0.10 MPa and ϕ = 35°. Similarly, an earthquake acceleration of α = 0.08 g was used for all the analyses. The percentage change refers to the ratios of slope height, slope angle and water depth to the original dimensions defined in Figure 5.

In the case of the reinforcement options, the percentage change refers to the ratio of anchor force T to the weight of the wedges (24.8 MN for the slope with the tension crack and 28.6 MN for the slope with no tension crack). The anchor inclination was kept constant at $\theta = \phi = 35°$. This anchor inclination gives the minimum anchor load for a dry slope and it can be determined by minimising equations 1 or 5 with respect to θ.

The curves presented in Figure 12 show clearly that some remedial measures are much more effective than others and it is worth examining each of the options in turn.

Curves 1 (slope with tension crack) and 2 (slope without tension crack) show that reduction of the slope height is not an effective solution to the problem. In order to achieve the required factor of safety of 1.5, the slope height would have to be reduced by 50%. If this solution were to be adopted, it would be more practical to excavate the entire slope since most of the volume of the rock to be excavated is contained in the upper half of the slope.

Curve 3 (slope with tension crack) shows that reduction of the slope angle is a very effective remedial measure. The required factor of safety of 1.5 is achieved for a reduction of less than 25% of the slope angle. In other words, a reduction of the overall slope face angle from 50° to 37.5° would achieve the desired result. This finding is generally true and a reduction in the face angle of a slope is usually an effective remedial step. In the case of slopes under construction, using a flatter slope is always one of the prime choices for achieving greater stability.

Curve 4 (slope without tension crack) is an anomaly and demonstrates that calculations can sometimes produce nonsense. The reduction in factor of safety shown by this curve is a result of the reduction in the weight of the sliding block as the face angle is reduced. Since the water pressure on the sliding surface remains constant, the effective stress acting on the sliding surface decreases and hence the frictional component of the resisting forces decreases. When a very thin sliver of rock remains, the water pressure will float it off the slope. The problem with this analysis lies in the assumption that the block is completely impermeable and that the water remains trapped beneath the failure surface. In fact, the block would break up long before it floated and hence the water pressure acting on the failure plane would be dissipated.

Curves 5 and 6 show that drainage is not a very effective option for either of the slope models considered. In neither case is a factor of safety of 1.5 achieved. This is something of a surprise since drainage is usually one of the most effective and economical remedial measures. The reasons for the poor performance of drainage in this case is due to the combination of the geometry of the slope and the shear strength of the failure surface.

Curves 7 and 8 show that, for both slope models considered, slope reinforcement by means of rockbolts or cables can be an effective remedial measure. The anchor force required for a factor of safety of 1.5 would be about 100 tonnes per metre of slope length for the slope with no tension crack.

Final decision on long term remedial works

The two most attractive options for long term remedial works on this slope are reinforcement by means of cables or bolts or reduction of the slope face angle. The first option was finally rejected because of the high cost and because of the uncertainty about the long term corrosion resistance of reinforcement which could be placed in the slope.

This latter concern may not have been justified but, considering the very poor quality of some of the construction in Hong Kong at the time of this study, it was decided that the risk was not worth taking.

The option finally chosen was to reduce the slope face angle down to 35° by excavating the entire block resting on the failure surface and hence removing the problem entirely. Since good quality aggregate is always required in Hong Kong it was decided to work this slope face as a quarry. It took several years to organise this activity and, during this time, the water levels in the slope were monitored by means of piezometers. Although the road was closed twice during this period, no major problems occurred and the slope was finally excavated back to the failure plane.

References

Hoek E. and Bray, J.W. 1974. *Rock Slope Engineering.* London: Instn Min. Metall.