The Failure of a Cut Slope on the Tuen Mun Road in Hong Kong

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SYNOPSIS The investigation and analysis of two landslides in a recently-constructed cut slope in weathered granite are described in detail. The failures are attributed partly to complex geological conditions which caused a perched water table to develop in the major scar. Adverse jointing contributed to the minor failure. The original site investigation was not sufficiently detailed to identify the important geological and hydrogeological aspects of the site. Conclusions are drawn with respect to the usefulness of back analysis for identifying failure mechanisms.

INTRODUCTION

Tropical cyclones and low-pressure troughs bring short intense rainstorms and longer periods of heavy rainfall to Hong Kong each year. Within the last 20 years peak rainfall intensities of 157 mm in 1 hour, 550 mm in 24 hours and up to 1025 mm in 5 days have been recorded by the Royal Observatory. The effect of such rainfall is to cause a large number of landslides in the steep hilly terrain, which is extensively covered with a thick mantle of colluvium and deeply weathered rock (So, 1971; Lumb, 1975).

During the afternoon of 28th May and early morning of 29th May 1982 up to 500 mm of rain fell in parts of Hong Kong and resulted in several hundred landslides in an area of less than 1000 km². Twenty-two people were killed and many roads were partially or completely blocked.

This paper discusses two of these landslides, which occurred on a recently-constructed cut slope overlooking the Tuen Mun Road, a dual carriageway trunk road linking urban Kowloon with the new town of Tuen Mun in the western New Territories. These landslides partially blocked Tuen Mun Road whilst other nearby failures during the same storm blocked the only alternative access road to Tuen Mun. As a result, urgent remedial works were required to reinstate normal traffic flow.

The Geotechnical Control Office (GCO) maintains a 24-hour emergency service during heavy rainfall, which provides advice to other government departments at landslide incidents. Following the Tuen Mun Road incident, a geotechnical engineer was present within 3 hours of the call for assistance and provided immediate advice to Highways Office engineers concerning road clearance and restricted land openings. Follow-up action involved site inspection, geotechnical and topographic mapping and recommendations for remedial works. These involved debris clearance, slope cutting to a flatter profile and improved surface drainage and protection, and were completed within 9 weeks.

CONSTRUCTION HISTORY OF THE TUEN MUN ROAD

The Tuen Mun Road was the first major rural trunk road to be built in Hong Kong. Construction of the 15 km long road began in 1974 and was substantially complete in 1983. The bulk of the road is constructed in mountainous terrain formed in granitic and volcanic rocks. In order to maintain the alignment standards for design speeds between 60 and 80 km/hour, a total of 16 bridges and average bulk excavations in excess of 250,000 m³/km were required (Slinn & Greig, 1976). As a result, many major soil and rock cut slopes up to 50 m high have been formed. Slope design was based on a nine-month site investigation contract which preceded the first phase of construction. Boreholes were located at an average of 100 m spacing along the alignment and the results of the investigation were complemented by geological mapping and monitoring during the construction period.

A number of major cut slope failures occurred during the first construction phase along the northern or highest carriageway, some of which have been documented by Slinn & Greig (1976). The initial slope designs for weathered insitu materials were based on the (then) traditional Hong Kong approach of 10V:6H (59°) individual faces, with 1.5 m wide benches at 7.5 m vertical intervals. The majority of the cut faces in soil were provided with surface protection in the form of chunam (a cement/lime/soil plaster), the prime purpose of which is to prevent surface water erosion.

Most of the large cuttings were completed by 1977 and relatively few problems were encountered in the succeeding 4-5 years. However, the May 1982 rainstorm was the most severe event since June 1972 and resulted in five major landslides along the road, causing considerable traffic disruption.

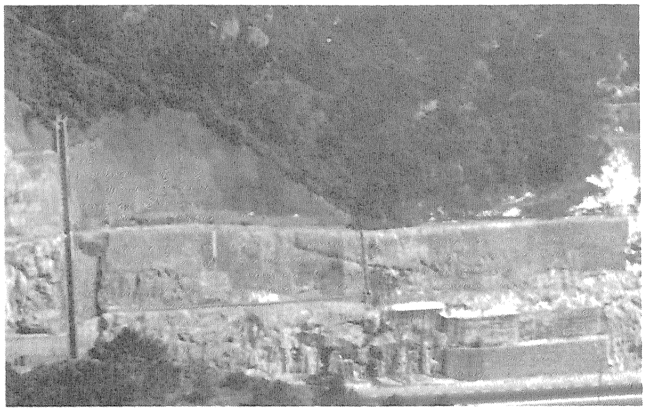

Figure 1 Oblique aerial photograph before failure

Figure 2 Oblique aerial photograph after failure (June 1982)

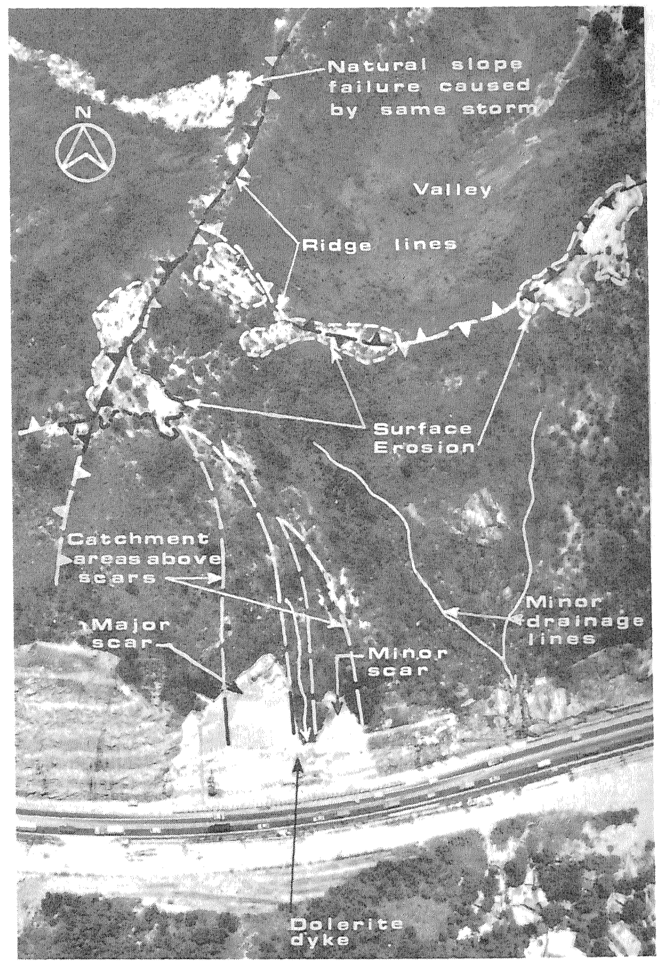

Figure 3 Vertical aerial photograph after completion of remedial works (July, 1982)

1982 FAILURES AT CHAINAGE 6750

Terrain Features

Figures 1 and 2 show the site before and after failure. The landslides occurred in decomposed granite and colluvium in the upper 10 m of the 20-25 m high cuttings. The original cut slope angle was 55° - 60°, with the natural slope above rising steeply at 35° - 40°. Both slides left an arcuate scar on the slope, with steep back walls. The bulk of the slide debris fell to the base of the cut slopes.

Figure 3 shows the site after remedial works had been completed (July 1982). The failures took place on the flank of a major north-south trending ridge line. The rock exposed in the base of the cut slopes was not affected by the landslides. Bare, unvegetated areas near the ridge crest above the failure scars on Figure 3 indicate the susceptibility of the decomposed granite to surface water erosion. No perennial streams occur above the head of the cut slopes but a minor valley falling towards the south separates the two scars. The natural catchment areas are approximately 2200 m² for the major (western) scar and 750 m² for the minor scar.

Site Investigation

A borehole was put down within the area of the major failure scar in 1976 as part of the original site investigation, and a standpipe piezometer installed at that time was found to be still operative in 1982 following the failure.

In order to provide better access to the scars for field description, bamboo ladders were erected as shown in Figure 2. Trial pits were put down at two locations within the failure scars to allow fuller material descriptions as well as to determine the thickness of the landslide debris. Hand-trimmed, large block samples and driven tube samples were taken to the laboratory for testing.

A full topographic survey was carried out at a scale of 1:100, partly using photogrammetric methods. The original geometry and features of the slope were established by reference to 'as constructed' drawings from the road contract, aided by oblique and vertical aerial photographs.

ENGINEERING GEOLOGY

The weathered materials were described fully by

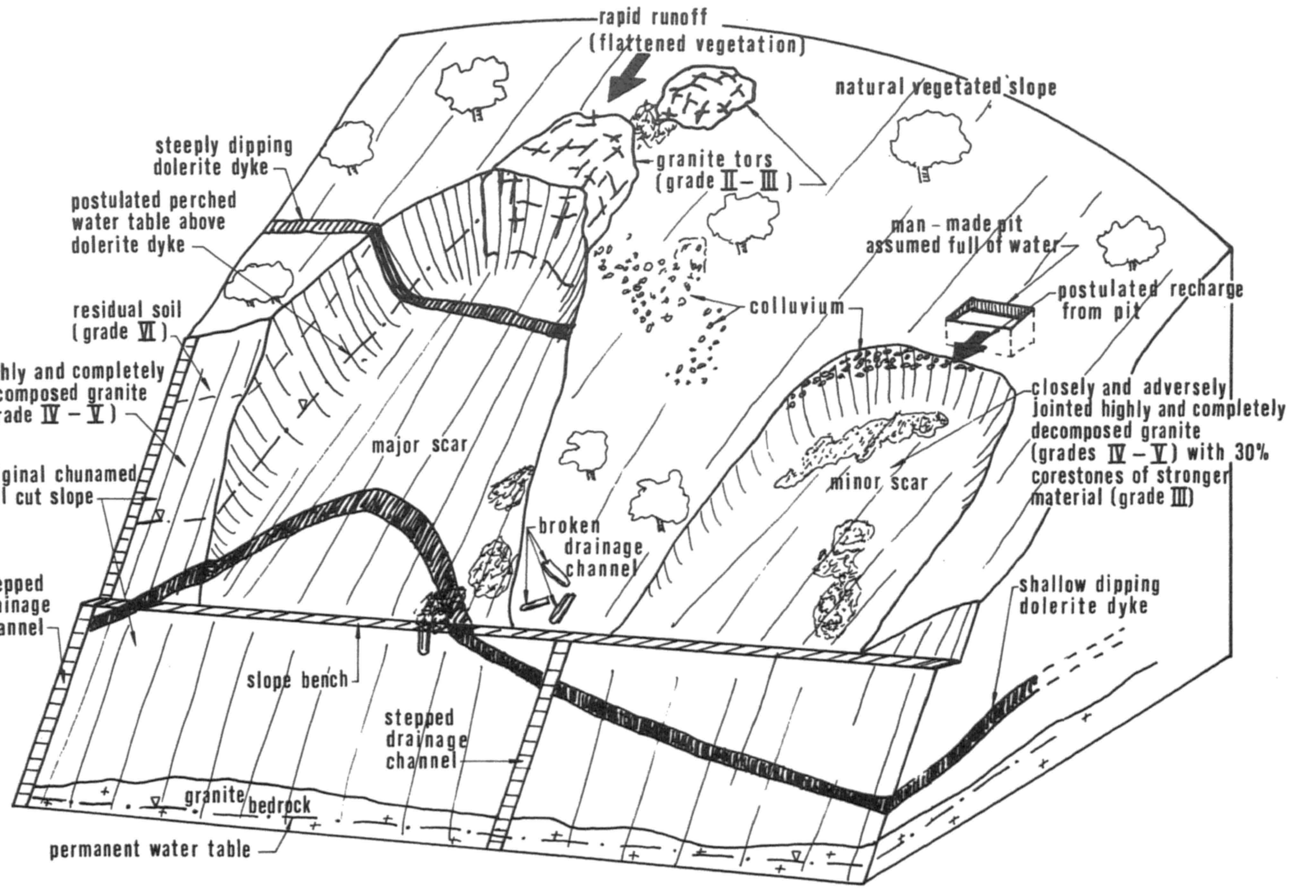

Figure 4 Schematic block diagram showing simplified geology and hydro-geological conditions

the method outlined by Hencher & Martin (1982), the basis of which is the use of field index tests to define the degree of weathering in a uniform specimen. Heterogeneity in terms of variation in material decomposition within a zone or layer is dealt with by assessing the percentage, size and angularity of the subsidiary fractions within the main zonal material and describing these fractions separately.

In simple terms, the two failures occurred in decomposed granite (grades IV and V). In detail, however, it was established that the local geology is far more complex, with the granite being intruded by persistent dolerite dykes and with local variations in material decomposition, resulting in heterogenous engineering properties. The dolerite dykes in particular are considered to have played an important part in the failure by causing a perched water table to develop and this is discussed in more detail below.

The principal geological features are illustrated schematically in Figure 4 and can be recognized in Figure 2.

Granite

The bulk of the decomposed granite exposed in the major failure scar comprises porphyritic, medium-grained, slightly microfractured granite giving very low or zero Schmidt hammer rebound values. Several specimens of the granite slaked when immersed in water. The granite is therefore close to the transition between grade IV and V decomposition as defined by Hencher & Martin (1982). Corestones were not noted.

In the rear scar of the major failure there is a large tor (relict insitu boulders) of grade II and III granite, but it is unlikely that this strong granite played a significant role in the failure as its boundary with the weaker materials is marked by the presence of persistent sub-vertical joints which would have provided a release surface.

In the minor failure scar, up to 70% of the decomposed granite is of a similar decomposition grade to that exposed in the major scar. However, the mass strength would be higher due to the presence of approximately 30% of moderately decomposed (grade III) material. This stronger granite occurs as irregular cobble and boulder-sized corestones between well-defined joints along which the more decomposed material is concentrated.

The decomposed granite in both failures is highly jointed, with the material in the minor failure being closer jointed and more blocky.

Within the failure scar a number of adversely orientated discontinuities were noted, although during field examination these were not considered sufficiently persistent for the landslide to be attributed to simple plane or wedge failure.

Dolerite

As illustrated in Figure 4, two major dolerite dykes up to a metre in thickness intrude through the granite. Within the main failure scar the rock is decomposed to a dark red, uniform silt-sized soil. Similar material sampled from another nearby failure scar was shown to have a permeability approximately one order of magnitude lower than that of the decomposed granite. Therefore it is likely that these dykes restricted seepage through the granite.

LABORATORY TESTING

Shear Strength of Granite

Shear strengths were determined by direct shear and triaxial tests on representative samples of the grade IV and V granite. All triaxial tests were carried out undrained with pore-pressure measurement on back-saturated samples. Specimens for direct shear testing were soaked over-night under load before shearing. Several of the tests were carried out multistage.

The results from these tests in terms of peak strength are given in Figure 5. The data show a great deal of scatter but on closer examination it is found that strength is clearly related to the dry density of the samples, which is indicated beside each data point in Figure 5.

It was also noted that the stronger, high-density samples exhibited pronounced dilatant behaviour which was in turn related to the normal stress (Figure 6).

In order to investigate these results in more detail, stresses were resolved as in rock shear testing (Hencher & Richards, 1982) to account for dilatant or compressive behaviour at peak shear strength. These corrected data are presented together with the original data in Figure 7. This shows that the individual peak strengths can be explained as a basic friction angle, (Ø'=41°) together with a positive or negative component which is due solely to the geometrical deviation of the sample from horizontal during testing. This geometrical behaviour was governed by the original density of the material and the normal stress applied.

This rational explanation for the scatter in the raw data gave confidence for its use in analysis.

Parameters of c'= 5 kN/m², Ø'= 36° were chosen to be representative of the low density granite (1.5 Mg/m³) typical of the major failure scar. Samples taken from the minor scar were of higher density and higher strengths would therefore be appropriate.

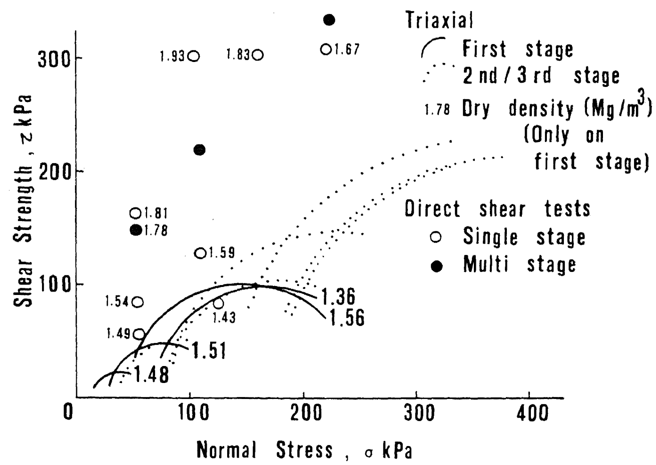

Figure 5 Results of strength tests on decomposed granite (grades IV/V)

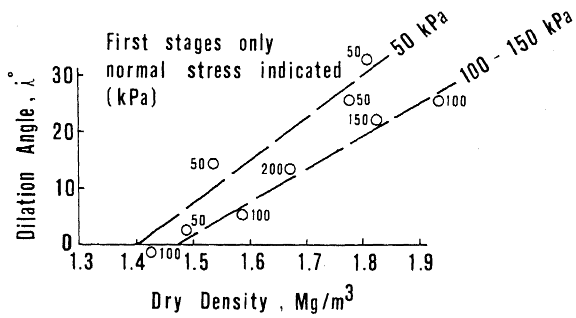

Figure 6 Effects of dry density and normal stress on dilatant behaviour of granite samples in direct shear tests.

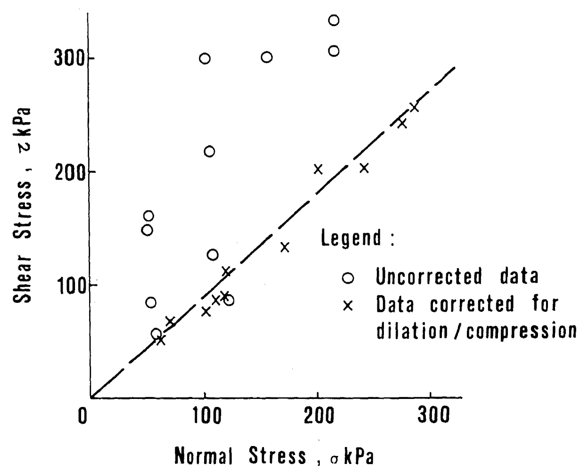

Figure 7 Direct shear test data corrected for geometrical deviation at peak shear strength.

HYDROGEOLOGY

It is clear that the failures on the Tuen Mun Road together with the hundreds of others that occurred in Hong Kong on the same day were caused by the abnormally intense rainfall. Based on Royal Observatory data, it is probable that over 400 mm of rain fell on the Tuen Mun site in only a few hours. Despite this there is no proof that the Tuen Mun road failures were caused by the effects of groundwater.

On the days that the failures were inspected, no seepage was noted issuing from the scars although of course the material was damp. Furthermore, when it was eventually realised, twelve days after failure, that a piezometer which had been installed at the location of the major failure during the original investigations was still functioning, it was found that the water level was 7 m below the failure surface. This agreed with readings taken in 1976. Observations of seepage from the rock face in the road side below the failures confirmed that the 'permanent' water table was well below the failure scar levels.

A number of field observations, however, suggested the possibility of transient perched water tables which might have led to the failures. Firstly it was noted that above the major scar, vegetation had been flattened, apparently by intense surface run-off leading from the natural catchment towards the edge of the cut slope. Secondly, the continuous, low-permeability, decomposed dolerite dykes passing through and just beneath the major failure scar may have acted as aquicludes, leading to critical pore water pressures in the sections of slope that eventually failed.

In the case of the minor failure, a large man-made pit found just above the crest of the slope may have filled with water during the storm, providing a continual and delayed recharge to the already saturated slope. These features are illustrated schematically in Figure 4.

ANALYSIS

The conditions at failure, particularly with respect to pore water pressures, are not sufficiently well-known to allow definitive back analysis to be carried out. Therefore a sensitivity analysis approach was adopted with strength parameters being varied within likely ranges according to laboratory test data and using various trial perched water tables in accordance with the hypotheses discussed earlier.

On the basis of laboratory data, the bulk unit weight of the material in the minor failure was taken as 18.9 kN/m^3 and for the major failure, 21.3 kN/m^3.

Cross-sections of the failure scars are given in Figures 8 and 10. Both sections were analysed using various trial piezometric surfaces as well as for dry conditions. All calculations were carried out using both the Janbu Rigorous and Simplified Methods (Janbu, 1973) programmed for an ICL 2900 computer.

Results and Discussion - Major Failure

Strength parameters are given in Figure 9 for which a factor of safety of 1.0 is calculated for the various trial piezometric levels. The representative strength of the decomposed granite, as discussed above, is indicated by an

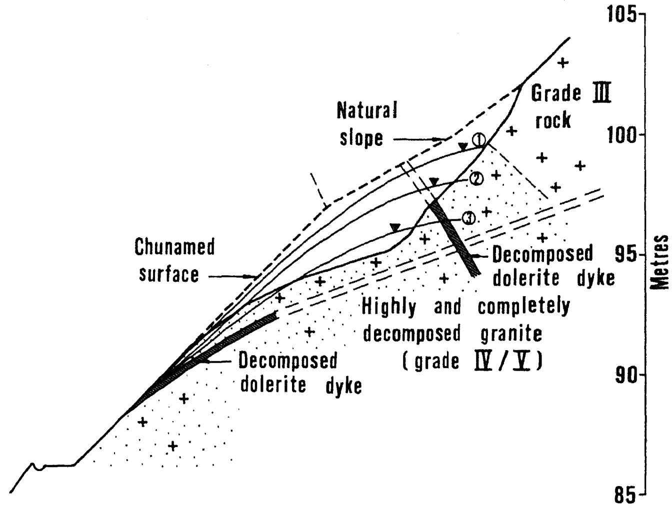

Figure 8 Section through major failure.

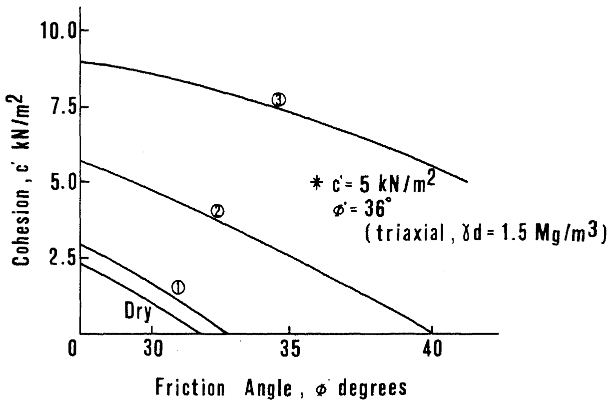

Figure 9 Results of analysis for major failure.

asterisk in Figure 9. Using this strength, it can be seen that the factor of safety would only be reduced to unity for water conditions between those indicated by lines 2 and 3 and it is concluded that these were the probable failure conditions.

For failure to have occurred with no positive pore water pressures, it was calculated that the shear strength of the decomposed granite would have had to be much lower than measured or expected for such material. It is therefore concluded that a perched water table formed above the dolerite dyke and that this led to the failure.

Results and Discussion - Minor Failure

In Figure 11, factors of safety are plotted for various strength parameters on the assumption that the piezometric surface was at ground level.

It can be seen that for this extreme condition, the factor of safety would be greater than one even when using the low strength parameters adopted for the major failure scar. The strength parameters obtained from samples taken from the minor failure scar were much higher than this. Triaxial tests on the weaker material (70%) gave values of c'= 18 kN/m^2,

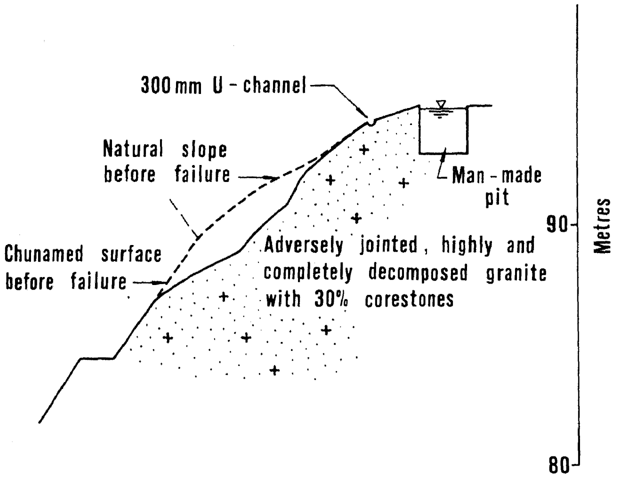

Figure 10 Section through minor failure.

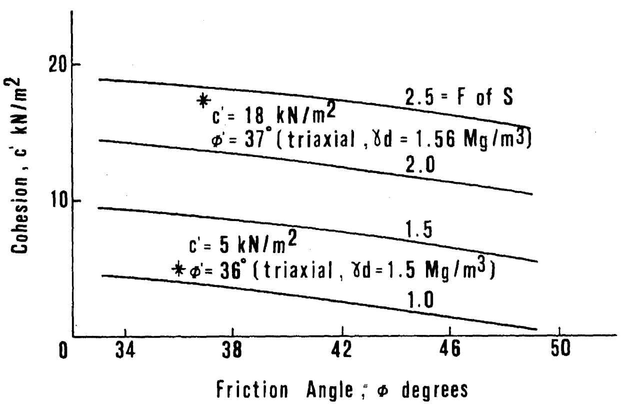

Figure 11 Results for minor failure.

$\phi' = 37°$ and the factor of safety using these parameters is 2.5 even for the worst conceivable ground water conditions. Also the presence of 30% of stronger materials could have only improved the stability. There are two possible conclusions. First, the measured shear strengths were unrepresentative of the intact materials that actually failed. This is not considered likely as the materials were carefully described and the sampling points chosen to be representative. Second, the failure occurred along the complex series of adversely orientated, impersistent joints noted in the failure scar and which are assumed to have lower strengths than the intact granite. This hypothesis appears to be the more feasible of the two.

It should be noted that the original site investigation failed to identify the important features of these failures. The dolerite dyke was not recorded, neither were any details of jointing given on the original borehole log. The piezometer was installed too deeply, so that pore pressures due to perched water could not have been measured even if the piezometer had been installed with maximum-level reading devices or automatic monitoring.

CONCLUSIONS

The two failures at Ch. 6750 on Tuen Mun Road were caused by transient perched water tables which reduced the shear resistance along the potential failure planes. In the case of the larger failure, drainage was restricted by the presence of persistent, dolerite dykes. Ground water in the minor failure scar was probably continually recharged from a man-made pit at the crest of the slope.

The major failure can be explained by using strength parameters derived from coventional triaxial tests on intact low-density granite. The shear strengths derived from direct shear and triaxial tests on intact materials from the minor failure were too high to explain the failure, even allowing for the most extreme water conditions. It is concluded, therefore, that the minor failure was joint-controlled.

The original site investigations for this slope indicated none of the features that were eventually responsible for failure. The importance of accurate identification and description of materials cannot be overemphasised. The location of piezometers installed during site investigations should be selected with care so as to provide relevant data for design.

This paper illustrates the main problem of back-analysis, which is the uncertainty in defining accurately the ground conditions at failure. In this case, even though a piezometer was actually sited within the failure scar, the data were not sufficient to allow calculations of definitive strength parameters. The usefulness of back-analysis is demonstrated however, in that it has been possible to make a rational interpretation of the mechanisms of failure in qualitative terms.

ACKNOWLEDGEMENTS

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