Predicting the performance of residual soil slopes

Prédiction du fonctionnement des talus des sols résiduels

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SYNOPSIS Residual soil slopes are common in many parts of the world, but little attention has been paid to appropriate methods of analysis and design. The basic approaches available for dealing with these heterogeneous materials are reviewed in this Paper and details are given of some recent developments in design methods, with particular reference to the design and performance of residual soil cut slopes in Hong Kong. Examination of the relationship between rainfall and landslides for Hong Kong shows that short-term intensity is almost solely responsible for the many slope failures, antecedent rainfall being of little account. The state-of-the-art with respect to the application of soil mechanics methods to the analysis of residual soil slopes is reviewed, and the main difficulties are examined with reference to a number of Hong Kong case histories. It is concluded that our soil mechanics predictive tools are far from adequate for analysing residual soil slopes, largely because of the difficulties of predicting pore pressures and of modelling geological detail which often controls the mode of slope failure.

1. INTRODUCTION

This Theme Lecture was originally conceived as covering the broad field encompassed by the title "Comparison of Prediction and Performance of Earth Structures". During the preparatory stages, however, the Author became aware that this was too large an undertaking for either the oral or written version of the Lecture. The scope was therefore dramatically reduced to enable concentration on a single aspect of earth structures which is of great importance and which has been much neglected by geotechnical engineers, namely the performance of residual soil slopes.

Residual soils are widespread, and they represent the most common type of material in those parts of the world where slope failures are very prevalent. This is particularly so in tropical countries, where deep weathering of the bedrock has invariably resulted in thick residual soil mantles, and where heavy rainfall is usually the main agent of slope failures. Apart from their economic importance, rain-induced landslides often present a significant hazard to life and property.

Residual soils have hitherto received very little attention from the soil mechanics community. This neglect, and the resulting paucity of relevant literature, stems largely from the fact that the very nature of residual soils makes the application of soil mechanics principles problematical and renders the materials difficult to model for the purposes of engineering analysis and design. The main characteristics of residual soils responsible for this are :

(a) they cannot usually be considered in isolation from the complete weathered rock profile, of which they are one component part,

(b) they are generally very heterogeneous, which makes them difficult to sample and test,

(c) they are nearly always unsaturated, which poses considerable difficulties for shear strength measurements, and

(d) they invariably have high permeabilities, which makes them subject to rapid changes in material properties because of external hydraulic influences.

Experience has shown that engineering problems in residual soils are not usually amenable to the principles of soil mechanics alone, but that these must be combined with the appropriate elements of 'geology', 'geomorphology' and 'hydrology' in much greater measure than is necessary for dealing with sedimented soils. Geotechnical engineering in residual soils therefore spans the narrowly separated fields of soil mechanics, rock mechanics and engineering geology, and the engineering geological approach is generally the most satisfactory one for these materials.

From the point of view of analysis and design, residual soils undoubtedly represent the 'difficult' end of the broad spectrum of engineering soils, as opposed to saturated soft clays, which represent the 'easy' end. Whereas a great deal has been published about our ability to predict the behaviour of soft clays, a negligible amount has been written to enable an evaluation of soil mechanics methods of prediction as applied to residual soils.

Mention must be made of the well-known writings of Lambe (1973, 1975), which have given us a framework by which to judge our soil mechanics predictions, and which have done so much to focus attention on the need for more good measurements of performance. Lambe's predictive efforts have for the most part, however, been concentrated on

the 'easy' soils, and even for these, it has been found that the careful application of soil mechanics methods does not always lead to accurate predictions of field behaviour. A great deal of experience and engineering judgement is therefore often necessary for making economically viable and acceptably safe predictions where less predictable soils are involved. In this context, the 'observational' philosophy continually expounded by Peck (1969, 1975, 1980) is central to good geotechnical engineering.

The material in this Paper is based on the Author's experiences with the performance of residual soil slopes in Hong Kong, which is prone to landslides that occur in the steep granitic and volcanic residual soil slopes during times of heavy rainfall. These failures are of major social and economic significance. Some severe rainfall events have in the past resulted in many failures which have caused large numbers of casualties and widespread damage. In recent years, therefore, a great deal of effort has been devoted to the prediction and prevention of landslides.

In part, this Paper draws heavily on two previous publications by the Author (Brand, 1982, 1985a) which reviewed the various approaches to analysis and design in residual soils with particular emphasis on slope stability.

2. RESIDUAL SOILS

2.1 Definition

No universally accepted definition of 'residual soils' exists. These materials are products of insitu weathering of rocks, the degree of weathering and extent to which the original structure of the rock mass is destroyed varying with depth from the ground surface. This process gives rise to weathering profiles which contain material 'grades' from fresh rock to completely weathered material, the latter usually being described by geotechnical engineers as 'soil'. For engineering purposes, it is difficult to separate the 'soil' from the rest of the weathering profile, and the whole profile is therefore best treated as a single entity.

For the purposes of tropically weathered profiles, the earth materials are sometimes categorised simply as 'laterite', 'saprolite' and 'rock'. The engineering behaviours of laterite and saprolite are usually considered to be governed by the principles of soil mechanics, and they are therefore the materials of most importance here.

Large deposits of colluvium often exist in conjunction with residual materials, particularly as colluvial fans on footslopes of hillsides. Colluvium is material derived from the weathering of any parent rock which has been transported downhill by the agencies of gravity and water. It can range in general composition from a collection of matrixless boulders at one extreme to a fine slopewash material at the other. It possesses many of the same general characteristics as residual soil, particularly in the context of engineering behaviour. Because it is commonly found as slope cover over weathered rock profiles, it is sometimes difficult to distinguish between colluvium and the insitu material, particularly if only drillhole samples are available for examination. For geotechnical engineering purposes, colluvium can therefore be grouped with residual soils.

A recent publication contained a summary of the types of material regarded as 'residual soil' in eighteen countries, as revealed by a collection of review papers (Brand & Phillipson, 1985). No universally accepted definition of 'residual soil' emerged, but the following general statements can be made :

(a) residual soils can result from the weathering of any parent rock type,

(b) under the category of 'residual soil' are included materials which are not completely weathered and which retain the original structure of the parent rock (saprolite), and

(c) colluvium is often categorised as 'residual soil' for engineering purposes.

For the purpose of this Paper, therefore, 'residual soil' will be defined as that broad group of materials, formed by the insitu weathering of any rock type, which exhibits engineering behaviour that is considered to be governed mainly by the principles of soil mechanics. In terms of the six-grade weathering classification system given below (Table I), these materials are grades IV, V & VI. Colluvium is included with these.

2.2 Published Literature

There is only a small amount of published literature on the engineering properties and behaviour of residual soils. As a starting point, reference should be made to the proceedings of the Specialty Session on Lateritic Soils held in Mexico City (Moh, 1969) and to the book by Gidigasu (1975), which relies heavily on these proceedings. Also of importance are the proceedings of the conferences on residual soils held in Hawaii (ASCE, 1982) and Brazil (ABMS, 1985a).

The proceedings of the ten International Conferences on Soil Mechanics and Foundation Engineering each contain some papers directly relevant to residual soils, as do the proceedings of the many ISSMFE Regional Conferences held to-date; the session on slope stability in residual soils at the Fourth Panamerican Conference is particularly noteworthy (ASCE, 1971). To a lesser extent, there are also specifically relevant papers in the proceedings of the four Congresses of the International Association of Engineering Geology and the five Congresses of the International Society for Rock Mechanics. There is a wealth of residual soil literature in the proceedings of the seven Brazilian Conferences on Soil Mechanics and Foundation Engineering, but unfortunately this is virtually all in Portugese.

A few key publications address themselves to the general philosophy and approach to engineering in residual materials. The first of these is the state-of-the-art report on slope stability in residual soils by Deere & Patton (1971). The others are the review papers by de Mello (1972)

and Brand (1982, 1985a), which all deal with the subject of residual materials in fairly broad terms. In addition, the comprehensive report produced very recently by the ISSMFE Technical Committee on Tropical Soils (ABMS, 1985b) is of importance.

Especially worthwhile contributions to the literature on the engineering properties of residual soils have been made by Vargas (1953), Lumb (1962a, 1965), Sowers (1963), Little (1967, 1969), De Graft-Johnson & Bhatia (1969) and Dearman et al (1976). Only Vaughan & Kwan (1984) and Vaughan (1985a), however, appear to have attempted to provide a theoretical framework for residual soil behaviour.

Noteworthy publications specifically on the slope stability aspects of residual soils have been written by Vargas (1967), Patton & Hendron (1974), Morgenstern & de Matos (1975) and Blight (1977). Good reviews of the landslide problems in these materials have been made by Vargas & Pichler (1957), Da Costa Nunes (1969) and Jones (1973) for Brazil, Page & James (1981) for Colombia, Sowers (1971) for Puerto Rico, Lumb (1975) and Brand (1985b) for Hong Kong, Oyagi (1984) for Japan, and Brand (1984) for Southeast Asia.

Colluvium features hardly at all in the technical literature, even though it is a fairly common engineering soil type. Good geomorphological textbooks, such as that by Young (1972), provide a general description of the formation of colluvium, but few attempts appear to have been made to devise an engineering classification system, nor to investigate its wide range of material properties. Although colluvium features fairly prominently in some published descriptions of mass movements, there are only a few papers which describe engineering designs in this material, the most useful of which is probably that by D'Appolonia et al (1966).

2.3 Weathering Profiles

The accurate logging of weathering profiles is fundamental to successful design and construction in residual profiles. These often contain a whole range of materials from an engineering point of view from 'soil' to 'rock'. The weathering profile is therefore of great importance for the stability of slopes, because it usually controls :

(a) the potential failure surface, and therefore the 'mode' of failure for analysis and design, and

(b) the groundwater hydrology, and therefore the critical pore pressure distribution in the slope.

There is no universally accepted system for describing and classifying the component parts of a weathering profile. The classification systems commonly used in soil mechanics have very limited application to weathered rock profiles. Classification in terms of weathering 'zones' and weathering 'grades' is essential for engineering design, and there have been several major attempts to provide a satisfactory description and classification system for engineering purposes. Deere & Patton (1971) gave a valuable comparative summary of the classification systems available at that time, but there have been some important subsequent developments. The Geological Society of London (1977) produced a Working Party Report on the subject of the description of weathered rock masses for engineering purposes, the main elements of which have now been incorporated into the British Code of Practice for Site Investigation (British Standards Institution, 1981). Other important papers on this general topic are those by Little (1967, 1969), Dearman (1974), IAEG (1979, 1981), Hencher & Martin (1982) and Martin & Hencher (1984).

Any weathering description and classification system must be suitable for the particular geological conditions and engineering purpose to which it is applied. In Hong Kong, where site formation and slope stability are the main geotechnical engineering problems (see Section 3 below), the Geotechnical Control Office (1984) has adopted a system for the granites and volcanic rocks which is based on the original work by Moye (1955) and Ruxton & Berry (1957), but which has several important additions suggested by Hencher & Martin (1982). A profile is logged according to the six material grades given in Table I and the four profile zones described in Table II. It should be noted that weathering zones A and B comprise those materials referred to in some countries as 'laterite' and 'saprolite' respectively.

For the purposes of geotechnical analysis and design, the following should be noted :

(a) grades I to III material are usually treated as 'rock', and grades IV to VI material as 'soil', and

(b) the engineering behaviour of weathering zones A and B is broadly considered to be governed by the principles of soil mechanics.

There exists no engineering description and classification system for colluvium, although one is badly needed. An attempt has been made in Hong Kong to provide a framework for such a system (Huntley & Randall, 1981), but this is entirely descriptive in character and requires a great deal of further development.

2.4 Investigation

For projects in residual soils, site investigation must generally be more extensive and more expansive than for more homogeneous earth materials. The emphasis must be on the engineering geological approach, for which the following are the main elements :

(a) execution of adequate surface and subsurface exploration to establish the site engineering geology, to define the 'engineering' materials involved, and to retrieve good quality samples for laboratory testing,

(b) identification of especially significant geological, geotechnical and hydrological features,

(c) study of existing local and other 'case histories' of similar projects and of sites with similar geology,

(d) routine insitu and laboratory measurement of selected engineering properties of materials to establish lower bound values, and comparison of these with generalised parameters developed from existing data, and

(e) continuous reappraisal of the site investigation results throughout the period of construction.

The importance of the study of good case histories cannot be overemphasized. Particularly important also is the continuous reappraisal process, since site investigations carried out in residual soils before construction commences can in most cases only be regarded as 'initial' investigations, and the geotechnical engineer must be prepared to modify his design as excavations reveal much fuller subsurface information than was available prior to the construction phase. This is the basis of the 'observational method' of design expounded by Peck (1969).

Whereas the use of generalised soil properties is not unsatisfactory where the soils are such that these can be established within a sufficiently sensible range, this is not so for residual soils, because of their variability and inhomogeneity. Generalised parameters for residual soils are difficult to obtain, and insufficient basic research has been carried out on most residual materials to provide even approximate values. This is an unsatisfactory situation which confronts everyone who works with residual soils, and it enhances the importance of the non-analytical methods of design described below.

Drilling and sampling in residual soil profiles can be difficult because of the vertical variability encountered, particularly since it is essential to obtain a high recovery for profile description purposes. A recent international review of the methods used worldwide for the sampling and testing of residual soils (Brand & Phillipson, 1985) revealed that the sampling practice in most countries leaves much to be desired, heavy reliance often being placed on methods developed for sedimentary soils. In this context, the drilling and sampling methods used for good quality site investigations in the weathered granite and volcanic profiles of Hong Kong may be of interest elsewhere (Brand & Phillipson, 1984); triple-tube rotary core barrels are used for sampling, and air-foam is sometimes employed as the flushing medium.

3. INTRODUCTION TO HONG KONG

Since there will be continual reference in this Paper to the slope stability problems in the residual soils of Hong Kong, an introduction to the Territory is appropriate at this stage.

3.1 General Description

The Territory of Hong Kong has a land area of only 1 050 sq. km and a population of nearly six million. It consists of Hong Kong Island, a piece of the Chinese mainland and more than 200 small islands, the largest of which is Lantau (Fig. 1). The population is concentrated in a number of distinct geographical locations

TABLE I

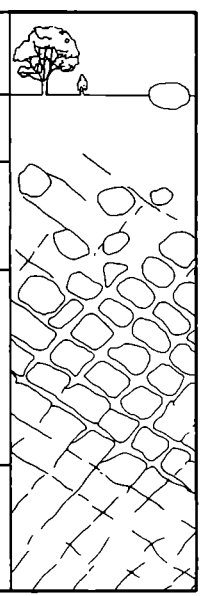
Weathering Grade Classification System Recommended by the Geotechnical Control Office (1984) for Use in Hong Kong

Grade		Typical Characteristics
VI	Residual soil	Soil formed by weathering in place but with original texture of rock completely destroyed
V	Completely decomposed rock	Rock wholly decomposed but rock texture preserved No rebound from N Schmidt hammer Slakes readily in water Geological pick easily indents surface when pushed
IV	Highly decomposed rock	Rock weakened - large pieces can be broken by hand Positive N Schmidt rebound value up to 25 Does not slake readily in water Geological pick cannot be pushed into surface Hand penetrometer strength index >250 kPa Individual grains may be plucked from surface
III	Moderately decomposed rock	Completely discoloured Considerably weathered but possessing strength such that pieces 55 mm diameter cannot be broken by hand N Schmidt rebound value 25 to 45 Rock material not friable
II	Slightly decomposed rock	Discoloured along discontinuities Strength approaches that of fresh rock N Schmidt rebound value greater than 45 More than one blow of hammer to break specimen
I	Fresh rock	No visible signs of weathering; not discoloured

TABLE II

Weathering Zone Classification System Recommended by the Geotechnical Control Office (1984) for Use in Hong Kong

Zone	Description
A	Structureless sand, silt and clay. May have boulder concentration at the surface.
B	Residual material with corestones. Rock percentage is less than 50%, and corestones are rounded and not interlocked.
C	Corestones with residual materials. Rock percentage is 50 to 90%, and corestones are rectangular and interlocked.
D	More than 90% rock. Minor residual material along major structural discontinuities which may be considerably iron stained.



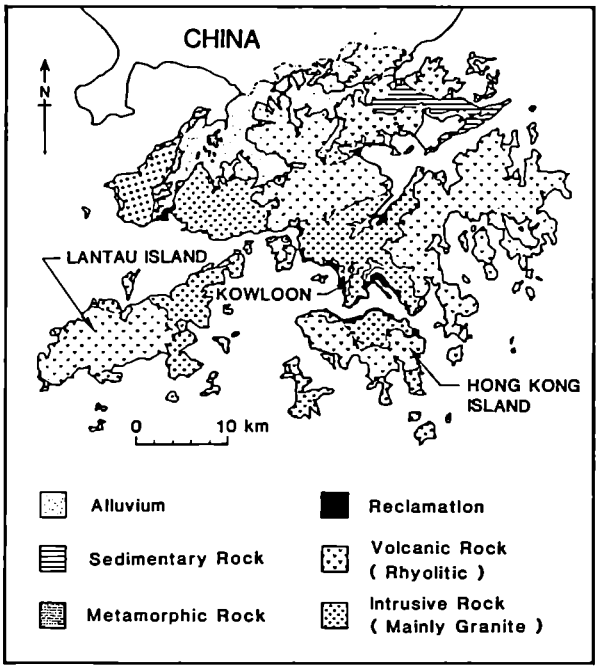


Fig. 1 Map of the Territory of Hong Kong Showing the Geology

dictated to a great extent by the terrain. High concentrations of building development and population exist all along the north side of Hong Kong Island (Fig. 2) and over the entire Kowloon peninsula, as well as in the five new towns which are currently under construction north of Kowloon. Apart from Hong Kong Island, the islands are largely undeveloped and sparsely populated.

The terrain of Hong Kong is very hilly. The land rises from sea level to 550 m on Hong Kong Island in a distance of only 1.5 km. Most of the Kowloon peninsula has now been levelled, along with reclamation of adjacent sea areas, but isolated hills of up to 100 m still exist. Immediately behind Kowloon, hills rise to over 450 m. In the New Territories, very little low-lying land exists, and peaks of over 400 m are common. Natural slopes throughout the Territory are steep, typically with upper slopes steeper than 35 degrees, midslopes of 25 to 30 degrees and footslopes of 15 degrees. Cut slopes formed for roads and site development works are commonly 40 to 80 degrees, and fill slopes are 30 to 35 degrees. Figure 3 gives the distribution of terrain angles for Hong Kong Island and Kowloon.

About two-thirds of the Territory's small land area is considered undevelopable. The usable land is therefore an exceptionally valuable commodity. In the absence of flat land, highrise buildings and other structures are increasingly being built on the mid-slopes and upper slopes of natural hillsides. In these circumstances, very intensive use is made of all developable land.

The rainfall in Hong Kong averages 2 225 mm

Fig. 2 View Looking East along the North Coast of Hong Kong Island Showing the Central District and the Mid-levels Area (Note Po Shan Landslide Scar)

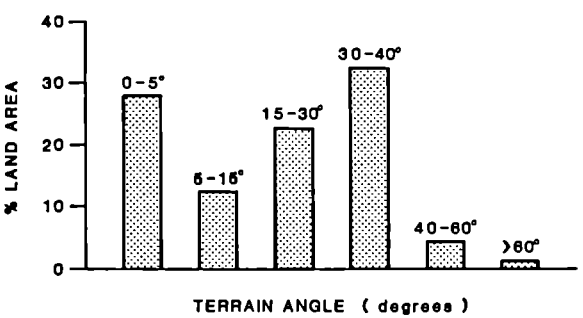


Fig. 3 Distribution of Terrain Angles over Hong Kong Island and Kowloon

annually, and more than 80% of this falls during the period May to September (Fig. 4). Intensities can be high, with 50 mm per hour and 200 mm in 24 hours being not uncommon.

3.2 Geology

The geology of Hong Kong, which is summarised in Fig. 1, has been described by Ruxton (1960) and Allen & Stephens (1971). The main rock types

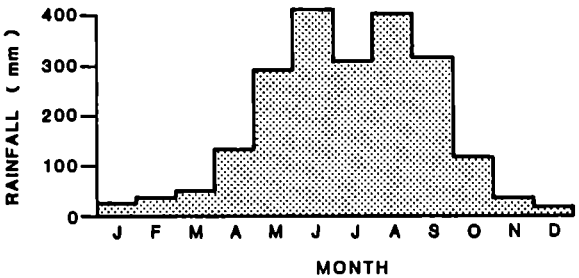


Fig. 4 Average Monthly Rainfalls in Hong Kong

are granite and acid volcanic rocks, which together cover the major portion of the Territory and are by far the most important from an engineering point of view. The small amounts of sedimentary and metamorphic rocks are of much less importance, although some specific landslide problems have been associated with these. Granite predominates in those areas of the Territory where building development is densest.

The granite varies somewhat in colour and composition, but the fresh rock is fairly uniform in its engineering properties. Of major significance for slope stability are the joint patterns and the extent of weathering. Joint spacings are typically 0.5 to 1 m, but they can be as wide as 3 m. In a few locations, sheeting joints occur almost parallel to the natural ground surface. The granite is extensively weathered almost everywhere, with depths of up to 60 m of silty-sandy residual soil, often with large corestones in the matrix or exposed on the surface. Below the residual soil mantle, a considerable depth of differentially decomposed material extends to great depth in a weathered granite profile. The weathering process in Hong Kong granite has been described by Ruxton & Berry (1957).

The volcanic rocks consist mainly of coarse tuffs, fine tuffs and rhyolite, but some ignimbrites and breccias also exist. These rocks are usually fine grained and have a blocky structure, with close joint spacings of generally less than about 0.25 m. They are more resistant to weathering than the granite, the residual soil mantles being only up to 20 m thick. A volcanic profile usually exhibits a steadily decreasing degree of decomposition with depth, the unweathered rock being reached at a much shallower depth than in a granite profile.

Steeply dipping dolerite dykes have been intruded in many places into the granite and volcanic rocks. These dykes, which vary in thickness between about 150 mm and 1.5 m, greatly complicate the engineering geology of many sites and are often a significant feature in landslides (Hencher et al, 1984; Hudson & Hencher, 1984). Granodiorite intrusions also exist in some places.

Of particular engineering geological significance in Hong Kong are the extensive bodies of colluvium which cover about 20% of the total land area. This accumulation of debris from old landslips and mass movements very often carpets the lower slopes of most of the hills. It varies in composition from a boulder field to a fine slope-wash, but it most commonly consists of boulders, cobbles and gravel in a matrix of sand, silt and clay. The colluvium is up to 30 m thick in places. It is sometimes in a loose state, with a high permeability, and it frequently gives rise to perched water-table conditions. It is also prone to the formation of 'pipes' or 'tunnels' as a result of internal erosion (Pierson, 1983; Nash & Dale, 1983; Brand et al, 1986), and these features can be of major significance to the hydrogeology of an area (Leach & Herbert, 1982; Premchitt et al, 1985).

Some engineering properties of the fresh and decomposed granite and volcanic rocks of Hong Kong have been reported by Lumb (1962a, 1962b, 1965, 1975, 1983).

3.3 Slope Stability

Geotechnical engineering practice in Hong Kong tends to be dominated by the slope stability problems brought about by the combination of the high intensity rainfall and the steep terrain. Several hundred failures may occur in a year, most of which are not of great consequence, but severe effects are felt from some of these in terms of casualties and damage. In fact, a significant landslide event, in which a large number of failures occur in one day causing considerable disruption and damage, can statistically be expected to take place in Hong Kong about once every two years (Lumb, 1975; Brand et al, 1984).

The whole range of slope 'features' is prone to landslides in Hong Kong, including natural slopes, soil cut slopes, rock cut slopes, earth fill slopes, retaining walls and boulders. The majority of failures, and usually those with the most severe consequences, take place in man-made features or are triggered by man-made features, particularly cut slopes in soil (weathering grades IV to VI).

Cut slope failures in soil, or in mixed soil and rock, now constitute by far the most common form of landslide. The volcanic rock profiles are more susceptible to failure than the granite profiles, and colluvium is frequently involved. The failures nearly always occur suddenly during intense rain without prior warning (Brand, 1984, 1985b), and most slip surfaces are shallow, the thickness of the failed zone usually being less than 3 m. A typical example is shown in Fig. 5.

4. METHODS OF SLOPE FAILURE PREDICTION

The stability analysis of a slope is ipso facto related directly to the prediction of the conditions under which the slope could fail. Soil mechanics methods of analysis have long been used for this purpose, but it must be realised that the majority of the world's man-made slopes were in fact formed on the basis of experience and precedent. The vast majority of residual soil slopes certainly fall into the latter category.

Apart from the application of sound judgement and experience alone, there are four basic methods

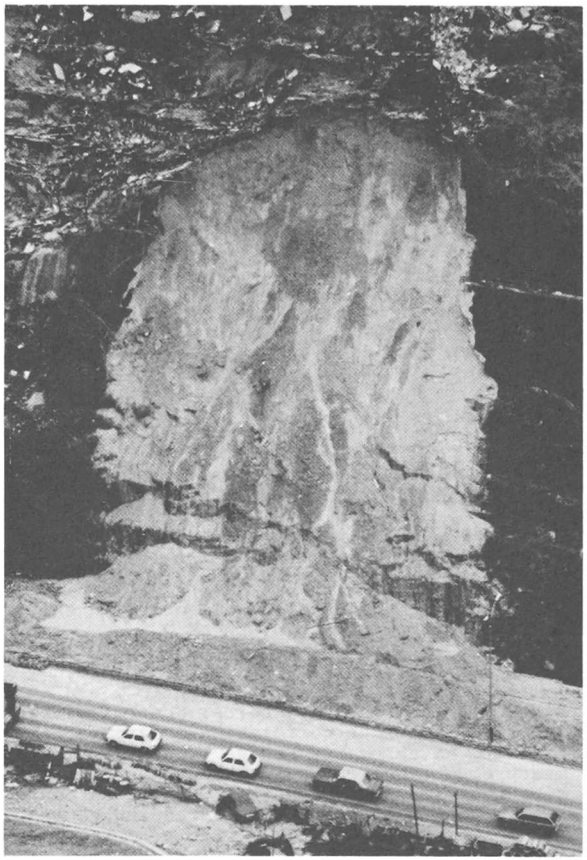


Fig. 5 Typical Hong Kong Soil Cut Slope Failure Caused by Heavy Rainfall

available for the prediction of rain-induced failures in residual soil slopes. These are :

(a) correlations between slope failures and pattern of rainfall,

(b) terrain evaluation, mainly on the basis of geomorphological mapping,

(c) semi-empirical (or modified precedent) approach, which is based on an examination of the geomorphology and geology of stable and unstable slopes, and

(d) soil mechanics analytical methods, usually in the form of limit equilibrium analysis.

The first of these two approaches can be considered to be directly related, since they apply to the stability of a land area in one particular location. The last three methods can be regarded as methods of analysis and design; all three have been used extensively in residual soils. The terrain evaluation and semi-empirical methods are closely related, in that both are based on an explicit assumption that the stability characteristics of a slope can be assessed on the basis of observations of the performance of others with similar characteristics.

Whereas the vast majority of cut slopes in residual soils were not designed on the basis of rigorous soil mechanics methods, the soil mechanics approach is being increasingly adopted even in this difficult soil type. This is certainly true in Hong Kong, where design practice is governed largely by the Geotechnical Manual for Slopes (Geotechnical Control Office, 1984). The degree of safety of a slope can only be quantified on the basis of an analytical method, whereas methods (a), (b) and (c) provide no such quantification.

5. RELATIONSHIP BETWEEN RAINFALL AND LANDSLIDES IN HONG KONG

5.1 Background

The vast majority of failures in residual soil slopes are caused by rainfall. In several countries of the world, studies have been carried out to correlate slope failures with the pattern of rainfall. At their best, such correlations provide a broad basis for predicting widespread slope failures, which can lead to the establishment of a warning system for those whose lives might be endangered. Rainfall-failure correlations might therefore be regarded as 'bulk' predictions.

Simple direct rainfall-failure correlations have been made for Brazil (Barata, 1969; Guidicini & Iwasa, 1977), Italy (Rossetti & Ottone, 1979), Japan (Onodera et al, 1974; Fukuoka, 1980) and the United States (Campbell, 1975; Nilsen et al, 1976), whereas more sophisticated correlation attempts have been undertaken in New Zealand (Crozier, 1969; Eyles et al, 1978; Eyles, 1979; Crozier & Eyles, 1980) and Hong Kong (Lumb, 1975, 1979; Brand et al, 1984).

It is very rare for detailed information to be available anywhere in the world on the geographical distribution and short-term intensities of rainfall, measurements of rainfall commonly being made on a daily basis at stations which are far apart. It is also rare for the precise times to be known for the occurrence of landslides. All previously published analyses of landslide data have therefore been based upon regional correlations of landslides with one-day or longer duration rainfalls, the more sophisticated correlations taking account of the antecedent rainfalls for periods of up to several weeks.

Relationships between rainfall and landslides were first established for Hong Kong by Lumb (1975, 1979), who classified landslide events into four categories, defined as follows :

(a) 'Disastrous event' - Territory-wide damage, with more than 50 individual failures recorded in one day,

(b) 'Severe event' - widespread damage, with between 10 and 50 failures in one day,

(c) 'Minor event' - localised damage, with less than 10 failures in one day, and

(d) 'Isolated event' - a single individual failure.

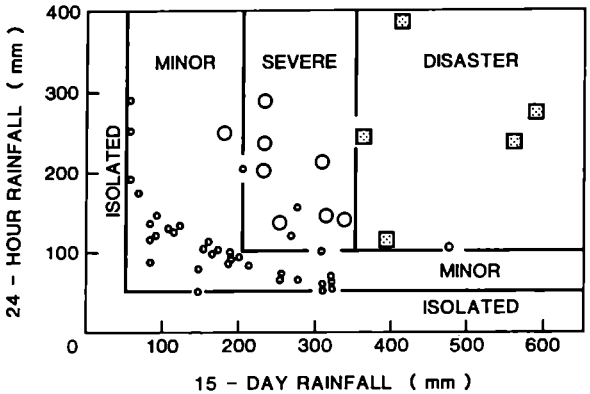


Fig. 6 Relationship between Rainfall and Landslides Established by Lumb (1975)

On this basis, Lumb correlated the landslide events for the period 1950 to 1973 with the one-day rainfalls and the 15-day antecedent rainfalls to obtain the interesting results shown in Fig. 6.

In incorporating antecedent rainfall into his correlations, Lumb (1975) in effect took account of the well-known phenomenon of soil moisture deficit and its relationship to shear strength. His one-day rainfalls, however, were taken as being the calendar-day rainfalls measured at the Royal Observatory (see below), and it is now known that better correlations are possible if account is taken of the localised variations in rainfall. A further correlation study was therefore undertaken recently in Hong Kong (Brand et al, 1984) which had the benefit of extremely detailed data not previously available. The results of this have greatly aided our understanding of rain-induced slope failures, and may have application elsewhere.

The rainfall-landslide correlation programme, carried out by the Geotechnical Control Office, was based on the six main parameters :

(a) one-hour rainfall,

(b) 24-hour rainfall,

(c) antecedent rainfall for periods of up to 30 days,

(d) number of landslides which caused casualties,

(e) number of casualties, and

(f) times of landslide occurrence.

5.2 Rainstorm and Landslide Data

Excellent rainfall data is available for Hong Kong. Rainfall records have been kept by the Royal Observatory (RO) since 1884. Rainfall statistics are referenced to the 'principal' gauge situated at the Observatory on the Kowloon peninsula (Fig. 7), at which hourly measurements are made. In addition, several autographic

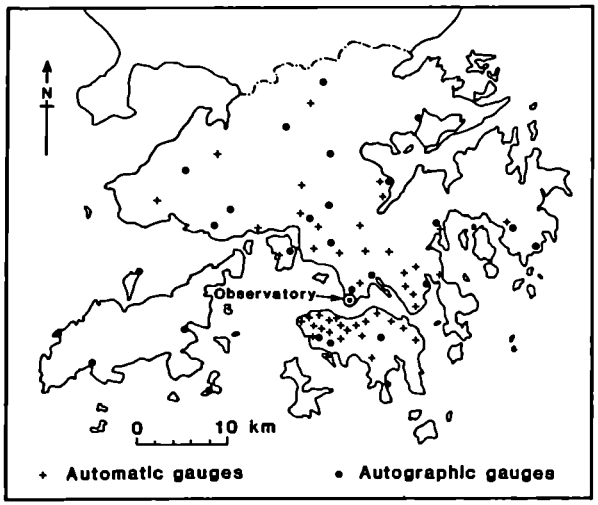


Fig. 7 Locations of Continually Recording Raingauges in Hong Kong, July 1985

gauges have been installed elsewhere in the Territory for many years.

Since 1978, a sophisticated system of automatically recording raingauges has come into use to provide data for the Geotechnical Control Office's rainfall-landslide correlation programme. At present, 46 automatic gauges transmit rainfall measurements continuously through telephone lines to a central micro-computer. The locations of these, and other continuously reading gauges, are shown in Fig. 7.

Large geographical variations in rainfall occur over the Territory during any rainstorm event, as dramatically illustrated by rainfall records shown in Fig. 8 for three automatic gauges during a very recent rainstorm. These variations are attributable to the effects of topography. A common feature of a Hong Kong rainstorm is that the highest rainfalls occur near the tops of

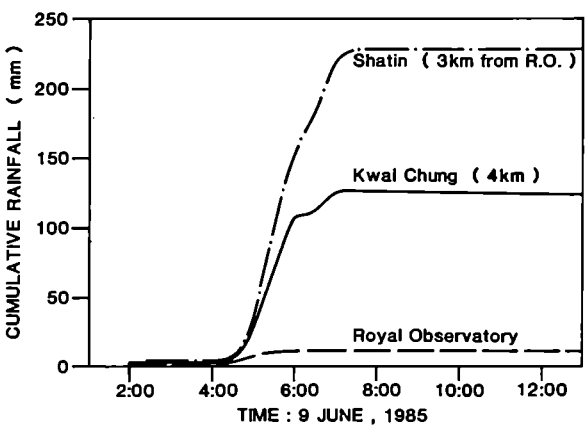


Fig. 8 Example of Large Variations in Rainfall over Short Distances in Hong Kong

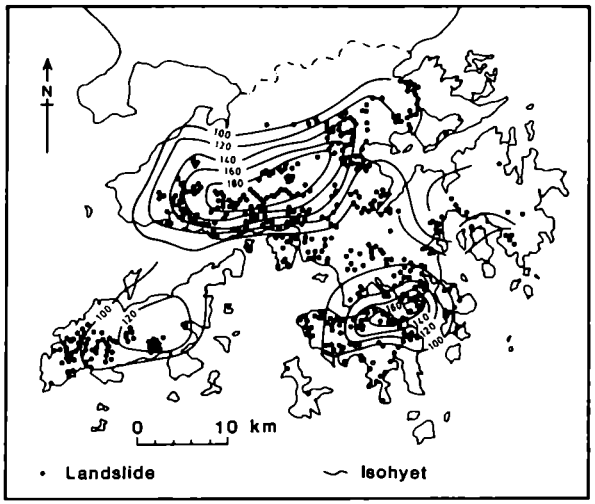


Fig. 9 Locations of Landslides in Relation to the 3-hour Isohyets in the May 1982 Rainstorm

large hills, and that the rainfall isohyets tend to follow the ground elevation contour lines. These features are illustrated in Fig. 9, which shows the contours of two separate three-hour rainfalls during a severe storm in May 1982.

If meaningful rainfall-landslide correlations are to be made for any geographical location, accurate information is needed on the number of landslides and their times of occurrence. Information on precise times of failure, however, is invariably difficult to obtain. In Hong Kong, there is often a delay of several hours between a failure occurring and an engineer being called out to inspect the failure; landslides in rural areas are sometimes not inspected for several days. Information on times of failure is therefore often not reliable from this source.

In collecting data for its landslide correlation study, the Geotechnical Control Office found the Fire Services Department to be an extremely reliable source of information on the occurrence of landslides in Hong Kong over the past twenty years. Although the FSD is involved with only a small proportion of the total number of landslides that occur, these comprise all the more serious incidents, and their excellent records can be considered to be representative of the landslide events in Hong Kong as a whole. Their data which relates to times of call-out is particularly good, and the fact that they are usually called to a serious incident immediately after its occurrence enables the time of failure to be fixed fairly accurately. The correlation studies were therefore undertaken largely on the basis of Fire Services data.

The Geotechnical Control Office's rainfall-landslide correlation study was based on data available for the 20-year period 1963 to 1983, for which the 13 severest rainstorm events are listed in Table III, together with their human consequences. Seven of the rainstorms were associated with troughs of low pressure and six were caused by tropical cyclones. For the majority of these rainstorms, there are no adequate records of the actual numbers of landslides, but the small numbers of Fire Services reports are indicative of the numbers of particularly serious incidents.

TABLE III

The Thirteen Major Rainstorm Events in Hong Kong and Their Consequences during the Period 1963-1983

Date of Rainstorm	Type of Storm	Maximum Rainfall, mm				Landslide Consequences			
		Observatory		Other Location		No. Failures Reported in Newspapers*	No. Fire Services Reports	No. People Killed or Injured	No. People Permanently Evacuated
		24-hour	1-hour	24-hour	1-hour				
24-25 August 1976	STS Ellen	416	52	500	82	314	23	57	2400
12 June 1966	trough	401	108	525	157	100	30	35	8500
29 May 1982	trough	394	44	430	111	498	15	48	8000
16-17 October 1978	STS Nina	380	37	380	38	15	1	1	no record
16 August 1982	STS Dot	362	68	370	95	62	6	9	1500
17 June 1983	trough	347	69	460	101	114	5	2	600
27 September 1965	TS Agnes	333	47	333	47	9	4	4	200
17 August 1971	Typhoon Rose	328	63	328	63	10	5	7	no record
12-13 October 1964	Typhoon Dot	304	60	375	94	8	10	39	8000
12-13 June 1968	trough	287	100	343	143	10	7	27	200
16-17 June 1972	trough	280	36	560	71	>15	15	21	7800
17-18 June 1972	trough	275	99	300	98	"dozens"	14	229	
17 May 1972	trough	271	79	377	92	"dozens"	2	0	6000

TS - Tropical Storm
STS - Severe Tropical Storm

* The numbers of landslides reported in newspapers are for comparison only - they represent the lower limits of the numbers of landslides that actually occurred.

However, detailed landslide information was obtained by the Geotechnical Control Office for the May and August 1982 rainstorms, which caused over 1 500 slope failures (observed from aerial photographs), and the June 1983 rainstorm, which caused more than 150 failures. While the numbers of landslides reported in newspapers are well below the actual figures, they give some relative measure of the number of failures that occurred during each storm.

It should be noted from Table III that the number of people killed and injured was not necessarily proportional to the severity of a particular rainfall event, since a few major individual landslides were responsible for large numbers of casualties. In particular, the June 1972 storm resulted in 224 casualties from the two failures that occurred at Po Shan Road on Hong Kong Island and Sau Mau Ping in Kowloon (Government of Hong Kong, 1972a, 1972b; Vail. 1984). Also, the vast majority of the people who were permanently evacuated from their homes in the 20-year period as a result of rainstorms were people who occupied temporary squatter dwellings on unformed hillside sites.

5.3 Effect of Rainfall Intensity

The available rainfall and landslide data was examined in detail for all thirteen rainstorm events listed in Table III. Figure 10 shows the patterns of rainfall for five of the most serious rainstorms (in terms of consequence). It will be seen that these storms generally lasted for a few days, but the majority of the rain fell in a few hours in each case. All these storms were therefore associated with short duration, high intensity rainfalls. Where sufficient data was available, a definite pattern emerged from a close examination of this fact for each of the thirteen cases.

The storms of June 1966, October 1978, May 1982 and August 1982 will be used as examples. The last two are of particular importance, because the Geotechnical Control Office obtained detailed information on these two severe storms, which

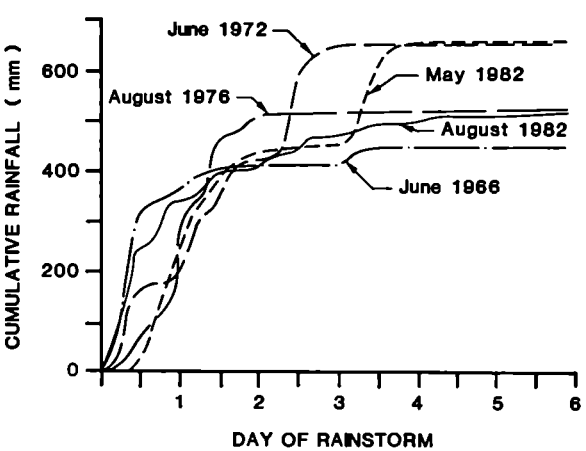


Fig. 10 Hong Kong's Five Severest Rainstorms in 1963-83

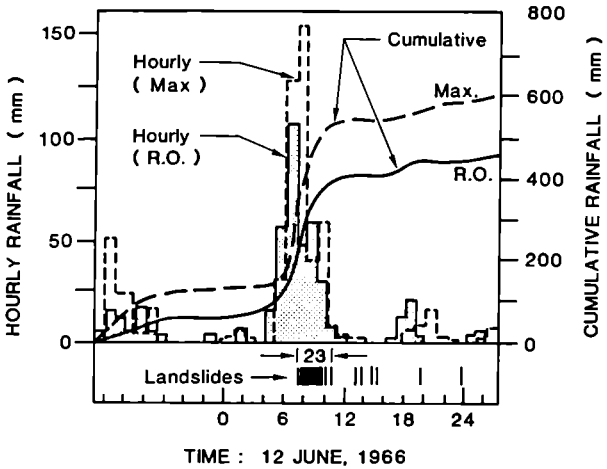


Fig. 11 Occurrence of Landslides in Relation to Rainfall at the Observatory and at Any Location during the June 1966 Rainstorm

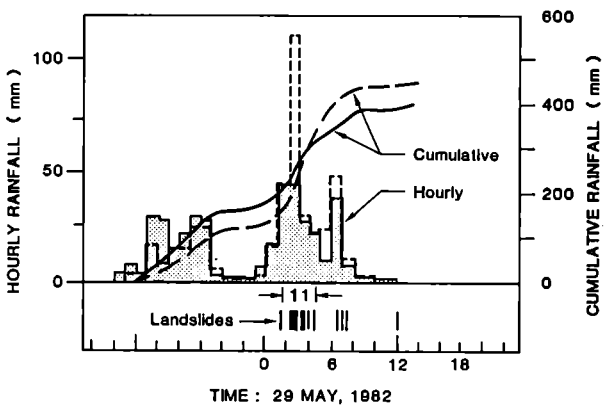


Fig. 12 Occurrence of Landslides in Relation to Rainfall at the Observatory and at Any Location during the May 1982 Rainstorm

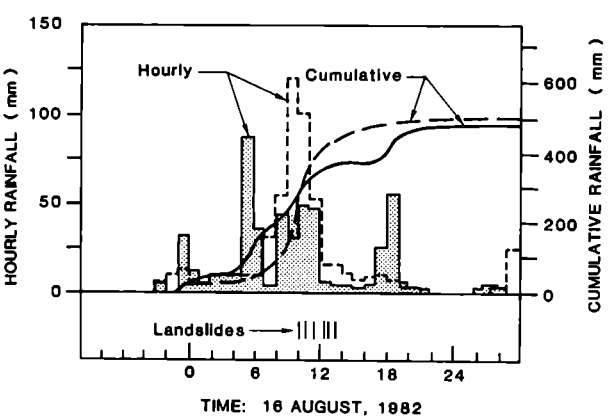


Fig. 13 Occurrence of Landslides in Relation to Rainfall at the Observatory and at Any Location during the August 1982 Rainstorm

resulted in the loss of 27 lives, and which ranked third and fifth of the last 20 years' rainstorm events in terms of 24-hour rainfall (Table III).

In June 1966, the heaviest rainfall in memory fell on Hong Kong Island, and the highest rainfall to-date was recorded at the Observatory. Widespread landslides and flooding occurred throughout the Territory. For this rainstorm, Fig. 11 shows the hourly cumulative rainfall at the Observatory and at the location of measured maximum rainfall in each case, compared with the occurrence of landslides as recorded by the Fire Services Department. The pattern of cumulative rainfall measured at the Observatory is quite similar to that measured at the location of maximum rainfall, although it is inevitably somewhat lower. In contrast, the hourly maximum rainfalls are much higher than the corresponding ones measured at the Observatory. Landslide timings for the rainstorm show very good correlations with hourly maximum rainfalls, which suggests that high intensity rainfall of short duration is a major triggering factor for landslides.

For the May 1982 rainstorm, Fig. 12 shows the rainfall and Fire Services landslide information. It can be seen that virtually all the recorded landslides occurred at the time of the maximum rainfall intensity. Also, although the maximum hourly rainfalls are dramatically higher than those measured at the Observatory, the pattern of cumulative rainfall is very similar in each case.

For the August 1982 rainstorm, the data plotted in Fig. 13 shows essentially the same pattern depicted in Figs. 11 & 12.

In Fig. 14 is shown the data for the October 1978 rainstorm. Although the 24-hour rainfall of 380 mm was the fourth highest in the 20-year period (and is similar to that for the May 1982 rainstorm, Fig. 12), only one landslide and one casualty were reported for this event, for which the maximum hourly rainfall was only 38 mm. This lends confirmation to the dependence of landslide occurrence on the short duration rainfall intensity.

During 1982, 3 248 mm of rain fell in Hong Kong, the highest annual rainfall on record. The distributions of all rainy days and of days with three or more landslides are shown in Fig. 15, plotted against the maximum hourly rainfall anywhere on each day. When the maximum hourly rainfall was less than 20 mm, no slope failures usually occurred. In contrast, failures nearly always occurred on the few days on which the maximum hourly rainfall intensity exceeded 40 mm per hour. The failures that occurred were also found to be in the vicinity of the area that experienced the highest rainfall intensity.

For the 20-year period from 1963 to 1983, an analysis of all the times of occurrence of landslides was carried out. The results presented in Fig. 16 show that the majority of the reported landslides occurred within four hours of peak rainfall intensity, with only 10% taking more than 16 hours to occur.

The dependence of landslide occurrence and

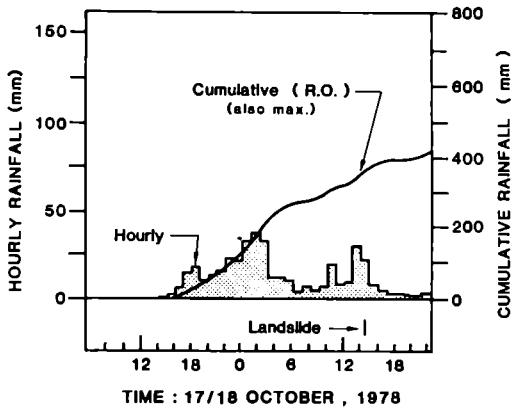


Fig. 14 Occurrence of Landslides in Relation to Rainfall at the Observatory at Any Location during the October 1978 Rainstorm

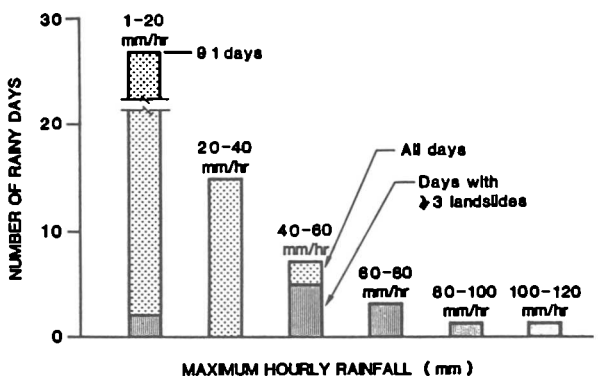


Fig. 15 Correlation of Landslide Occurrence during 1982 with the Maximum Hourly Rainfall Intensity Anywhere

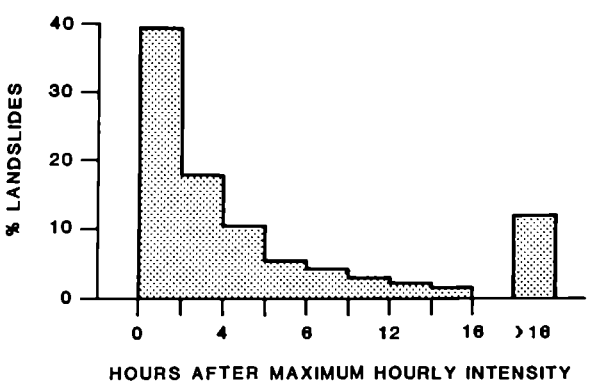


Fig. 16 Times of Occurrence of Major Landslides as Related to the Maximum Hourly Rainfall Intensity, during the Period 1963-83

casualties on hourly rainfall intensity is clearly shown in Fig. 17. Throughout the twenty-year period, the number of landslides that caused casualties was very small unless the maximum hourly rainfall anywhere in the Territory approached 70 mm. Above this figure, the number of landslides with casualties increased sharply with increasing maximum hourly intensity. The actual number of casualties caused by landslides shows a similar but even more dramatic correlation with hourly intensity.

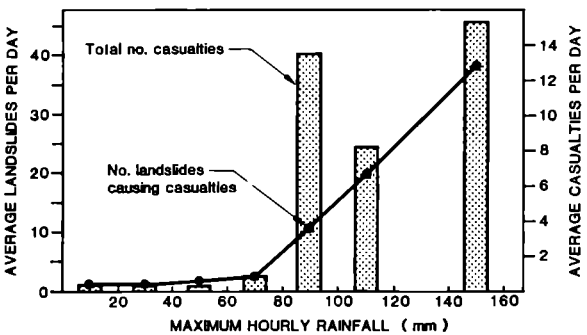


Fig. 17 Correlation of Landslides Causing Casualties and Number of Casualties during 1963-83 with Maximum Hourly Rainfall Anywhere

5.4 Correlations with 24-hour Rainfalls

Although clear evidence has been presented that Hong Kong landslides are dependent on short duration rainfall intensity, it is worth examining whether any correlations exist between landslides and 24-hour rainfalls.

For the May 1982 rainstorm, Fig. 18 shows the daily rainfalls at the Observatory for the event, together with the occurrence of all landslides for which the dates of failure are known. This shows that the large majority of landslides occurred on the 29th May, the day of the heaviest rainfall. The few failures that occurred up to a week later were probably caused bv delayed

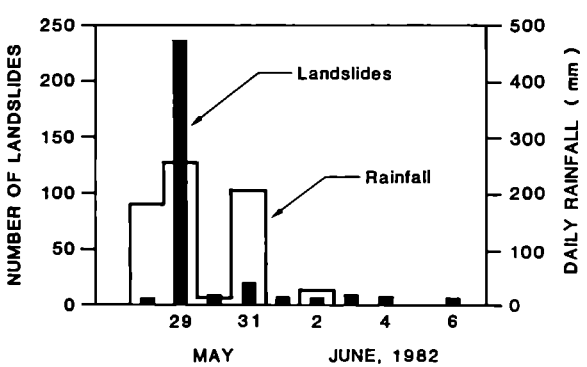


Fig. 18 Relationship between the Daily Rainfall at the Observatory and the Occurrence of Landslides during the May 1982 Rainstorm

groundwater build up (Hudson & Hencher, 1984).

The often large short-term differences in rainfall from place to place throughout Hong Kong tend to even out over the longer rainfall periods. This is demonstrated in Figs. 11 to 14, where the 24-hour rainfalls at the Observatory are generally not greatly different from the maximum values measured anywhere. It is therefore useful to examine the relationship between landslides in Hong Kong and the Observatory 24-hour rainfall.

Correlations between landslides in Hong Kong and the Observatory 24-hour rainfall are shown in Fig. 19 on the basis of two categories of event. A 'minor' event is one for which there are less than ten recorded landslides in one day, and a 'major' event is one for which there are more than ten landslides in one day. The 'minor' category is identical to that used by Lumb (1975), while the 'major' category corresponds to his combined 'severe' and 'disastrous' categories. Figure 19 demonstrates clearly that, where the 24-hour rainfall at the Observatory is below 100 mm, only a few minor events and no major events occur. With increasing 24-hour rainfall, the proportion of landslide days increases, with major events increasingly dominating at higher rainfalls. When more than 270 mm of rain falls in 24 hours, every rainstorm event results in a major landslide event.

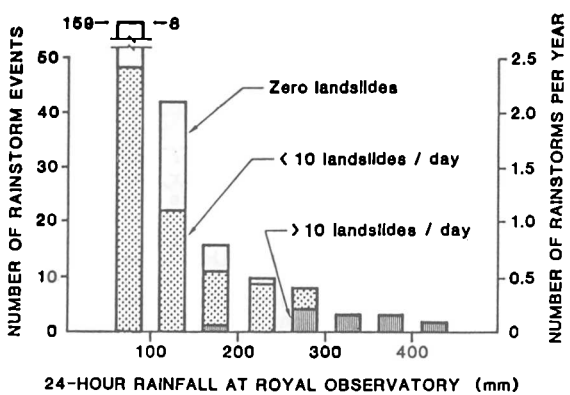


Fig. 19 Correlation of Number of Landslides with the 24-hour Rainfall at the Observatory during the Period 1963-83

The relationship shown in Fig. 19 is not unlike the distribution in Fig. 15 of landslide days in relation to maximum hourly rainfall for 1982. This is to be expected, since the heavier the 24-hour rainfall, the more likely it is that it will include a short duration rainfall of high intensity. The main importance of the correlation between landslides and the 24-hour rainfall, however, is its usefulness as a landslide warning criterion, since the accumulation of rainfall to approach a 'trigger' value can be sensibly anticipated a few hours in advance, whereas the short duration intensity cannot possibly be predicted in advance with any precision. The use of the 24-hour rainfall also has the advantage that measurements from the Obser-

vatory gauge alone can be used to represent the situation throughout the whole Territory.

5.5 Effect of Antecedent Rainfall

The evidence presented here suggests that the antecedent rainfall is not a significant factor in the occurrence of major landslide events in Hong Kong, despite the fact that it has been found to be of major significance elsewhere. This was confirmed in the study by the poor correlations obtained between landslides and antecedent rainfalls for periods of up to 30 days. In Hong Kong, failures appear to be directly related to short duration rainfall intensity. It is common, however, for such high intensity rainfalls of short duration to occur during prolonged rainfall events. Where limited information is available on rainfall intensities and times of landslides, it is therefore not surprising that a relationship can be found to exist between antecedent rainfall and the occurrence of a large number of landslides.

Of interest is the fact that the three- or four-day antecedent rainfall appears to influence the occurrence of minor landslide events in Hong Kong when short duration intensities do not dominate. This indicates that soil moisture deficit might well be a factor in rain-induced landslides in soil slopes, but that its effects are secondary in situations of high intensity rainfall.

5.6 Conclusions

From the rainfall-landslide correlation study in Hong Kong, the following main conclusions can be drawn :

(a) The large majority of landslides are induced by localised short duration rainfalls of high intensity, and these landslides take place at about the same time as the peak hourly rainfall.

(b) Antecedent rainfall is not a major factor in landslide occurrence, except in cases of minor landslide events which take place under relatively low intensity rainfalls of short duration. In these circumstances, only a few days antecedent rainfall appears to be significant.

(c) A rainfall intensity of about 70 mm/hour appears to be the threshold value above which landslides occur. The number of landslides and the severity of the consequences increase dramatically as hourly intensity increases above this level.

(d) The 24-hour rainfall usually reflects short duration rainfalls of high intensity, and this can therefore be used as an indicator of the likelihood of landslides. A 24-hour rainfall of less than 100 mm is very unlikely to result in a major landslide event.

The results of the rainfall-landslide correlation study are summarised broadly in Fig. 20, which shows the approximate frequency of landslide events (as defined by Lumb, 1975) in terms of

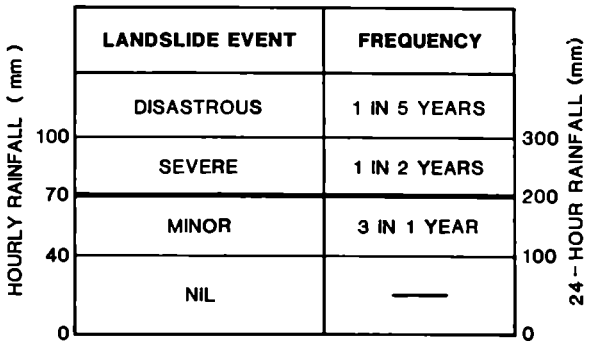


Fig. 20 Approximate Relationship between Rainfall Intensity and Landslide Events in Hong Kong

one-hour and 24-hour rainfalls. This forms the basis of a landslide warning system currently used in Hong Kong for hillside squatters.

6. TERRAIN EVALUATION

6.1 Background

Terrain evaluation is a powerful technique for categorising terrain for stability purposes. It has hitherto been somewhat neglected by geotechnical engineers, who could make much more use of the technique for planning and development studies, and to provide input for land use management purposes.

The evaluation of terrain is based on a suitable 'terrain classification' system,which provides a means for classifying terrain into homogeneous land units, largely on the basis of surface features. For this purpose, suitable geomorphological 'attributes' must be chosen. Attributes for stability purposes commonly include slope angle, land form, vegetation, surface hydrology and erosion, each of which is divided into a number of 'classes'. Once classified in this way, the terrain evaluation is carried out by sorting the land units into categories on the basis of predetermined combinations of the attribute classes.

An excellent example of the development of a comprehensive system of terrain evaluation for engineering purposes is the Australian PUCE system described by Grant (1975a, 1975b). This has been developed continuously over 20 years into a highly complex system which requires the use of a large storage capacity computer (Grant & Finlayson, 1978; Finlayson, 1984).

In addition to general engineering geological assessment maps, 'risk' or 'hazard' maps can be constructed from terrain evaluation data. This has been done in some landslide-prone areas of a few countries, notably France (Porcher & Guillope, 1979), Italy (Carrara et al, 1978; Carrara, 1983) and the United States (Nilsen & Brabb, 1977; Nilsen et al, 1979). Good reviews of the various systems used have been made recently by Brabb (1984), Varnes & Keaton (1984) and Varnes (1985).

The most advanced form of the landslide hazard

map is the digitized form. The methodology used is no different in principle from that for manually-compiled maps, but the terrain classification is digitized to facilitate ease of storage, retrieval and sorting. The computer-stored data also acts as a data bank of information that can be called upon at will. Hazard maps have been prepared in this way in a few countries, the most advanced system probably being that developed by Carrara et al (1978) and Carrara (1983) in Italy.

6.2 The GASP System in Hong Kong

The Geotechnical Control Office in Hong Kong has for several years placed considerable emphasis on its Geotechnical Area Studies Programme (GASP) to provide systematic geotechnical input for land use management and development planning purposes. This programme and some aspects of the system of terrain evaluation employed have been described in several publications (Brand et al, 1982a, 1982b; Burnett & Styles, 1982; Burnett et al, 1985).

GASP was designed to be carried out in the following three phases :

(a) Regional Studies - Initial geotechnical assessments (at a scale of 1:20 000), based entirely on aerial photograph interpretation, site reconnaissance and existing geotechnical information.

(b) District Studies: Stage 1 - Initial geotechnical assessments (at a scale of 1:2 500), based entirely on aerial photograph interpretation, site reconnaissance and existing geotechnical information, to give more detailed assessments of specific areas identified from Regional Studies.

(c) District Studies: Stage 2 - Expanded geotechnical assessments, based on the results of Stage 1 Studies together with data obtained from planned programmes of site investigation.

Regional Studies, each of which covers an area of 50 to 100 sq. km, provide geotechnical input for outline and strategic planning within the Territory. In addition, they are designed to provide a comprehensive physical land resource inventory through a computer-based Geotechnical Terrain Classification System (GEOTECS). All eleven Regional Studies have now been completed or are in their final stages.

District Studies are typically concerned with areas of 2 to 4 sq. km. They provide information suitable for local planning needs and are of assistance in the basic layout planning of large sites. For most localised areas, Stage 1 District Studies provide adequate information, a Stage 2 Study being required only if very significant geotechnical constraints are revealed by Stage 1. Nine Stage 1 Studies have so far been completed. As yet, no Stage 2 Studies have been undertaken as an integral part of GASP, but a related Stage 2 area study was completed several years ago (Geotechnical Control Office, 1982; Rodin et al, 1982).

The Geotechnical Control Office realised at an early stage that no existing system of terrain evaluation was ideally suited to Hong Kong's peculiar conditions. The GASP approach differs in several important aspects from any other known terrain evaluation or hazard mapping system. It combines elements of both of these approaches, but the zonation framework employed is based on the overall geotechnical assessment of the land units, not only on the identification of 'hazards' from a stability viewpoint. As might be expected, however, considerations of stability are dominant.

A comprehensive report is produced for each Area Study, an essential part of which is a base map and a series of transparent overlay maps.

For a Regional Study (1:20 000), the overlay maps consist of :

(a) Terrain Classification Map,

(b) Landform Map,

(c) Erosion Map,

(d) Physical Constraints Map,

(e) Engineering Geology Map, and

(f) Geotechnical Land Use Map (GLUM).

For a District Study (1:2 500), the overlay maps are :

(a) Terrain Classification Map,

(b) Surface Hydrology Map,

(c) Vegetation Map,

(d) Engineering Data Sheet,

(e) Engineering Geology Map, and

(f) Geotechnical Land Use Map (GLUM).

The Engineering Geology Map indicates the broad pattern of the geological materials and their general engineering characteristics, and it identifies features which are of engineering relevance. One of the major purposes of this map is to present the geomorphological and engineering geological constraints that influence the allocations of GLUM classes (see below). It is designed for use by engineers and engineering geologists who require explanations for the nature of the geotechnical limitations which affect the terrain.

For a Regional Study, the Physical Constraints Map summarises the physical constraints to regional development planning. For a District Study, every item of available engineering geological information (including every borehole location) is plotted and referenced on the Engineering Data Sheet to avoid confusion on the Engineering Geology Map; this information is shown in a much more general way on the Engineering Geology Map of a Regional Study. The Engineering Geology Map and Engineering Data Sheet have been described in some detail by Burnett & Styles (1982).

The main findings of a Geotechnical Area Study are engendered in the Geotechnical Land Use Map (GLUM). In this interpretative map, land units are classified into four GLUM 'classes' on the basis of combinations of attributes from the terrain classification, with due account being taken of the other data collected during the

TABLE IV

The Four-class Classification System Used for the Geotechnical Land Use Map (GLUM) in Hong Kong

Characteristics of GLUM Classes	Class I	Class II	Class III	Class IV
Geotechnical Limitations	Low	Moderate	High	Extreme
Suitability for Development	High	Moderate	Low	Probably unsuitable
Engineering Cost for Development	Low	Normal	High	Very high
Intensity of Site Investigation Required	Normal	Normal	Intensive	Very intensive
Typical Terrain Characteristics (Some, but not necessarily all, of the stated characteristics will occur in the respective GLUM class)	Insitu terrain with gentle slopes (0-15°) without severe erosion or instability. Cut platforms in insitu terrain.	Insitu terrain with slopes between 15° & 30° without instability. Insitu terrain of gentle slopes associated with drainage but no instability. Colluvial terrain with gentle slopes (0-15°) without severe erosion or instability.	Insitu terrain with slopes between 30° & 60° without severe erosion or instability. Insitu terrain less than 15° with history of landslips. Colluvial terrain less than 15° with evidence of instability. High to moderate fill slopes.	Very steep insitu slopes (>60°) and cliffs. Steep to very steep insitu and colluvial slopes with history of instability. Colluvial terrain with gentle slopes, but associated with instability and drainage.

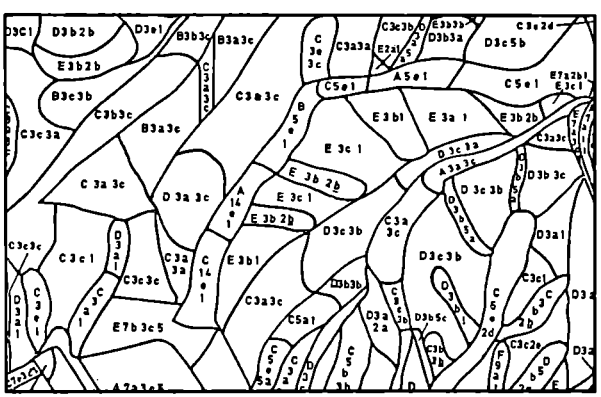


Fig. 21 Example of a Terrain Classification Map (Alpha-numerics denote attribute classes)

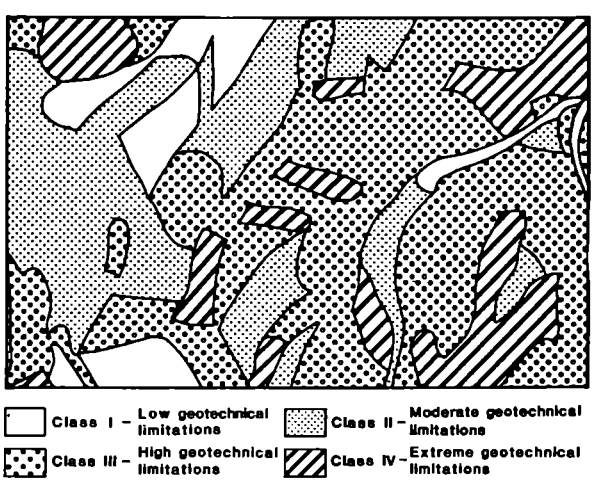


Fig. 22 Example of a Geotechnical Land Use Map (GLUM) for the Area Shown in Fig. 21

Study. Examples of a GASP terrain classification map and a GLUM are given in Figs. 21 & 22. The GLUM class is assigned to a unit on the basis of the likely geotechnical limitations on the land unit posed by the combination of its categories of the six attributes - slope gradient, terrain component, terrain morphology, erosion (which includes instability), slope condition and hydrology. The four GLUM classes are summarised in Table IV.

The results of GASP are intended for use largely by planners, but they also provide valuable data for engineering appraisal and feasibility studies. A recent study carried out to compare the GLUM class assessments in the Mid-levels area with the results of engineering stability analyses

(Styles et al, 1984) indicated clearly that the GASP system provides a reliable means of rapidly delineating localised areas of potential instability.

7. SEMI-EMPIRICAL DESIGN METHODS

7.1 Background

The majority of existing cut slopes in residual soils were not designed in the engineering sense but were formed on the basis of judgement or precedent. In most cases, the precedents were established essentially on the basis of common sense and were then gradually modified in the light of experience with the performance of the slopes. In a few instances, this 'precedent' approach has evolved into a formulated semi-empirical design method in the form of a set of design rules.

A particular set of slope design rules can obviously only be applied to a specific geological formation at one geographical location, because these rules have evolved from the topography, geology and climate of that location. It is true, however, that design rules developed for one location may provide a good guide for the development of rules for a similar geological formation elsewhere.

Semi-empirical design rules must be based on the examination of stable and unstable slopes of similar geology. This can in some cases result in the establishment of simple relationships between slope height and slope angle. At the other extreme, very complicated statistical relationships can sometimes be devised among a large number of geological and geomorphological parameters.

An example of the simplest type of semi-empirical design rules is illustrated in Fig. 23, which shows the recommended slope angles for different heights of cut slopes on the basis of the degree of weathering of Wellington greywacke (Grant-Taylor, 1964; Taylor et al, 1977). Earlier, Lane (1961) had correlated slope height and slope angle for slopes in clay shales in Montana, North Dakota and Kansas, to produce the plot shown in Fig. 24. The work by Shuk (1965, 1968), summarised in Fig. 25, is of particular note,

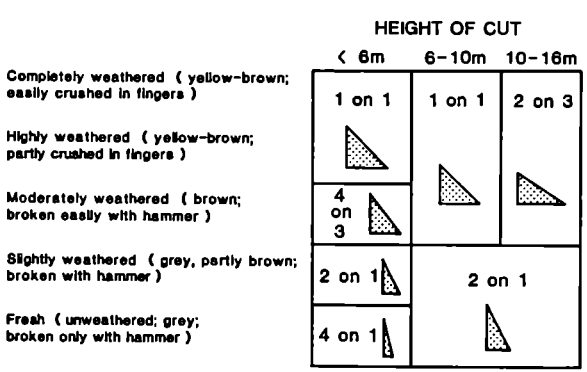


Fig. 23 Simple Empirical Design Rules Developed for Wellington Greywacke by Grant-Taylor (1964)

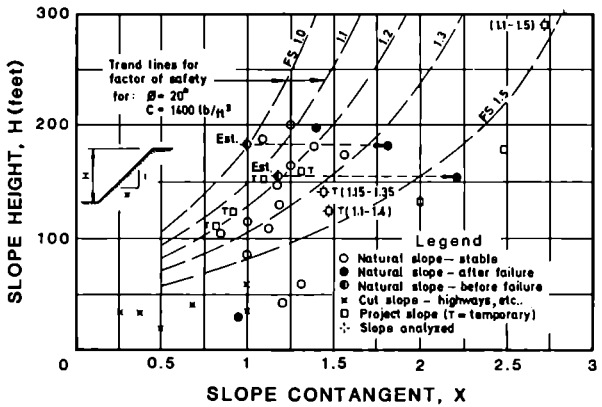


Fig. 24 Design Guidelines Developed by Lane (1961) on the Basis of the Performance of Clay Shale Slopes in Northern USA

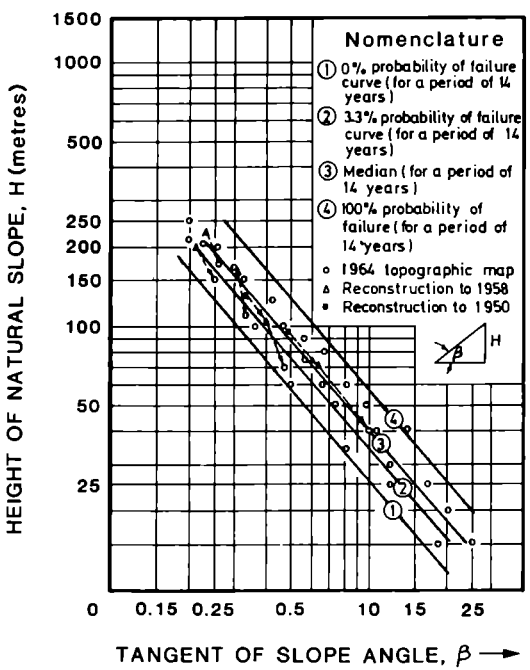


Fig. 25 Design Guidelines Developed by Shuk (1965, 1968) for Clay Shale Slopes near Bogota

since he combined a probability of failure in his log-log plot of height versus slope angle for natural shale slopes near Bogota, Colombia. Shuk's approach was later used by McMahon (1976) for data obtained on 233 slopes, for which he found that the log-log straight line relationship applied for any rock type.

Morphometric models have been used to establish the attributes common to landslides. The work by Blong (1973) and Crozier (1973) are good examples of this approach in its more descriptive

Fig. 26 Typical Stable Hong Kong Cut Slope Included in the CHASE Study (Brand & Hudson, 1982)

form. A more complex model was devised by Neuland (1976) using 31 variables, which included morphometric, geological, stratification and soil mechanics attributes. Equally sophisticated are the terrain models being used in parts of Italy (Carrara et al, 1977, 1978; Carrara, 1983).

7.2 The CHASE Study in Hong Kong

A major Study (CHASE) was carried out in Hong Kong a few years ago (Brand, 1982; Brand & Hudson, 1982) to examine the possibility of establishing semi-empirical guidelines for the design of cut slopes in the residual soil profiles. Detailed examinations were conducted on 177 stable and failed slopes, and the data was analysed statistically in an attempt to obtain simple correlations among the geological, hydrological and geometrical factors which control cut slope stability. More than 200 items of data were collected for each slope to ensure that any factors which influenced its stability were recorded.

A typical CHASE slope is shown in Fig. 26. This is 45 m high and is cut at an angle of about 60° into a steep natural hillside covered with dense vegetation. The material of the slope comprises 5.5 m of colluvium overlying decomposed granite,

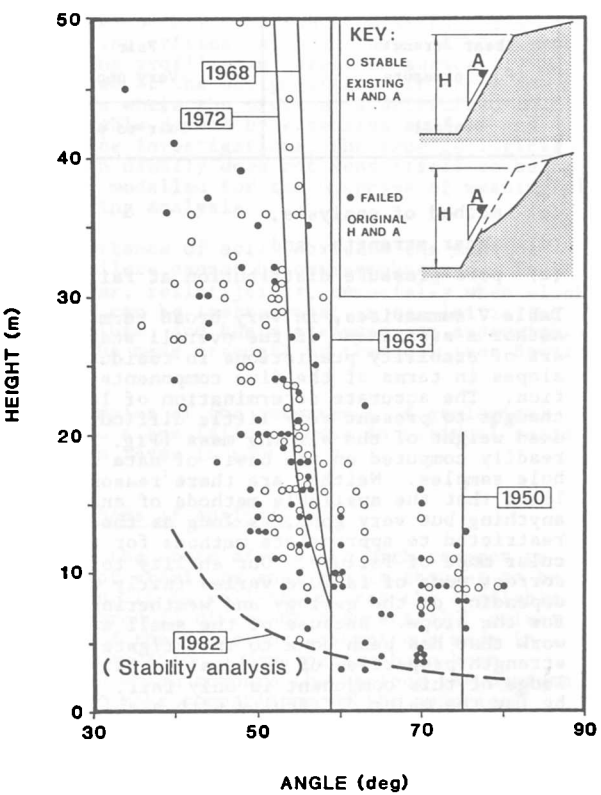


Fig. 27 Relationship between Height and Angle for the 177 Hong Kong Cut Slopes Included in the CHASE Study (Brand & Hudson, 1982)

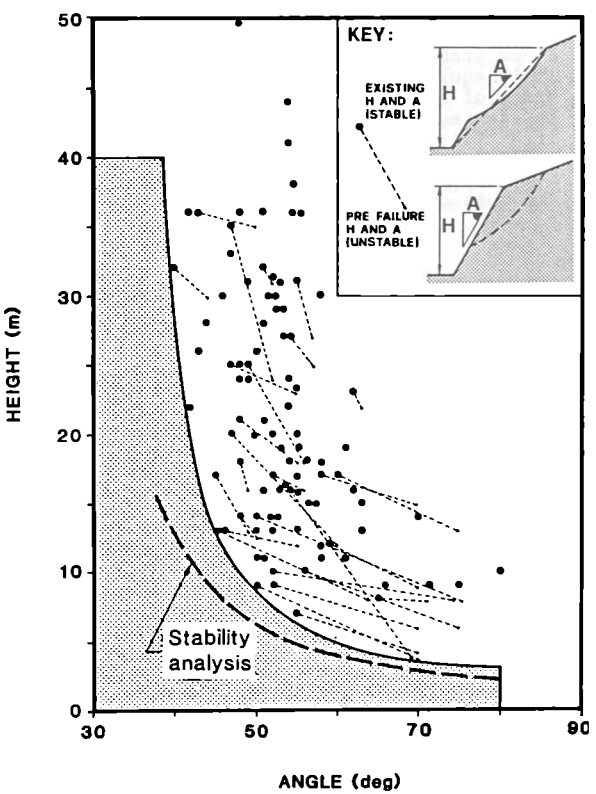


Fig. 28 Relationship between Height and Angle for the 117 Hong Kong Granite Soil Cut Slopes Included in the CHASE Study (Brand & Hudson, 1982)

which varies from grade V at the top to grade IV at road level. The surface is covered with a protective coating of 'chunam' (lime stabilised soil), which is commonly used in Hong Kong to prevent surface erosion and to minimise infiltration.

Despite the vast amount of data collected, and the extensive statistical analyses undertaken, the CHASE Study did not reveal the simple relationships hoped for, but it did result in the establishment of some lower bound envelopes for the relationships between height and slope angle of the kind shown in Figs. 27 & 28. These have proved to be useful for the initial design of new cut slopes, and for assessments of the likelihood of failure of existing slopes.

Figure 27 is of interest for the information it provides about empirical design rules adopted at various times in Hong Kong's history. It can be seen that none of these was meaningful in the context of the results of the CHASE Study. It is also evident that classical stability analyses are probably very conservative for Hong Kong conditions. The same conclusion is well supported by Fig. 28, where the changes in slope height and angle are shown for a large number of granitic soil cut slopes which had previously failed.

8. SOIL MECHANICS PREDICTIONS

8.1 Summary

Slope stability analysis is usually based on the limit equilibrium approach, for which the equilibrium of a sliding mass is examined (Fig. 29). The degree of stability is quantified in terms of a 'factor of safety', which is most commonly defined as the ratio between the average shear resistance and the average shear stress along the most critical slip surface, i.e. :

$$F = \frac{S_a}{\tau_a} \qquad (1)$$

The determination of the F-value for a particular slope therefore requires :

(a) prediction of the correct mode of failure (i.e. selection of the critical slip surface),

(b) prediction of the distribution of shear stress over the critical slip surface, and

(c) prediction of the distribution of shear resistance over the critical slip surface.

The distribution of shear stress along the critical slip surface is dependent on the loading and the method of analysis employed. The shear resistance along the slip surface is governed by the effective shear strengths of the materials of the slope and the normal effective stress distribution. The normal effective stress distribution is, in turn, a function of the pore pressure distribution at failure and the method of analysis. The five 'components' of stability prediction are therefore :

(a) mode of failure,

(b) loading,

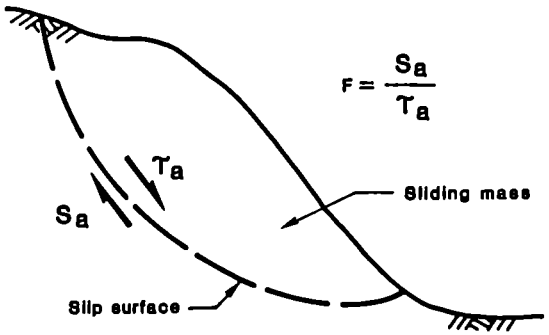


Fig. 29 Basis of the Limit Equilibrium Method of Stability Analysis for Slopes

TABLE V

Assessed State-of-the-Art of Stability Predictions for Residual Soil Slopes

Component	State-of-the-Art
Mode of failure	Good to poor
Loading	Very good
Method of analysis	Very good
Shear strength	Fair
Porè pressure	Very poor
Overall	Fair to poor

(c) method of analysis,

(d) shear strength, and

(e) pore pressure distribution at failure

Table V summarises, in very broad terms, the Author's assessment of the overall state-of-the-art of stability predictions in residual soil slopes in terms of the five components of prediction. The accurate determination of loads is thought to present very little difficulty, the dead weight of the sliding mass (Fig. 29) being readily computed on the basis of data from borehole samples. Neither are there reasons to believe that the available methods of analysis are anything but very good, as long as these are restricted to appropriate methods for the particular mode of failure. Our ability to select a correct mode of failure varies fairly widely depending on the geology and weathering profile for the slope. Because of the small amount of work that has been done to investigate the shear strength properties of residual soils, our knowledge of this component is only fair. However, by far the major difficulty with stability predictions in residual soil slopes is the poor state-of-the-art with respect to the prediction of pore pressure distribution at failure.

8.2 Mode of Failure

The selection of a suitable mode of failure is

crucial to accurate stability predictions in all soil types. It is particularly crucial for slopes in residual profiles, where the potential failure surface is often governed by geological detail. Some guidance on the accurate assessment of failure modes for geologically complex conditions was given many years ago by Terzaghi (1950). More recent important publications in this context are those by Skempton & Hutchinson (1969), Deere & Patton (1971), Patton & Hendron (1974) and Coates (1977).

Whereas the pre-failure geometry of a slope is easily defined, it is sometimes difficult to decide upon the critical potential failure surface for design. Occasionally, even a post-failure surface cannot readily be determined because of the multiple-failure nature of some landslides in residual materials.

As a general rule, the shear strength of a residual soil increases with depth, and slope failures can therefore be expected to occur on relatively shallow slip surfaces. These surfaces are largely controlled by the weathering profile. Failures most frequently occur along surfaces dictated largely by relict joints or by boundaries between weathering zones. By the very nature of residual weathering profiles, non-circular failure surfaces are the most common, and these are often almost planar over a major proportion of their length.

Geological detail is often crucial to the location of the critical slip surface. A complex weathering profile can rarely be adequately investigated at the design stage. In those few instances where the profile is determined in considerable detail by extensive surface and subsurface investigations, the true geological situation usually does not lend itself to being properly modelled for the purposes of meaningful engineering analysis.

The importance of soil fabric to the mode of slope failure cannot be over-emphasized. In particular, relict joints, especially when slickensided, can be instrumental in the initiation of a failure, and their presence can appreciably reduce the mass strength of the soil (see Section 8.4).

Some examples of the importance of geological detail to slope failure modes in residual soils are given later in Section 9.

8.3 Methods of Analysis

Although the state-of-the-art with respect to methods of stability analysis is thought to be very good, it is well-known that the available methods produce different theoretical F-values because of the different assumptions made in their formulation. Many papers have been published which compare the various assumptions and which make comparisons of the numerical factors of safety obtained for certain specific slope situations. For the up-to-date position, reference should be made to the recent publications by Fredlund et al (1981), Fredlund (1984) and Ching & Fredlund (1984).

For residual soil slopes, methods of stability analysis which employ circular slip surfaces are usually inappropriate, and methods which apply to any shape of surface must be used. In this category, are the well-known methods by Janbu (1954, 1973), Morgenstern & Price (1965) and Sarma (1973, 1979). The first two of these are in very common usage. Although Morgenstern & Price made more satisfactory assumptions than Janbu, the latter's method is much easier to programme and requires much smaller computer capacity. For these reasons, it is more widely used for routine stability calculations. Sarma's method is as rigorous as that of Morgenstern & Price, but it requires less iterations with consequent less computer time.

All the above methods of slope stability analysis are based on a two-dimensional failure mode. In practice, however, slope failures are nearly always three-dimensional, but it is rare for account to be taken of this in design or stability assessment, or even during the back analyses of failures that have occurred. Three-dimensional effects on slope stability analysis have been examined by Baligh & Azzouz (1975), Azzouz et al (1981), Lovell (1984) and Leshchinsky et al (1985), but more work on this needs to be done.

A great deal of effort has been put into refining the details of methods of stability analysis over the years, without sufficient attention being paid to the way in which these methods are used. A small study carried out in Hong Kong (Lumsdaine & Tang, 1981, 1982) is revealing in this respect. A large number of organisations intimately involved with slope stability assessment in Hong Kong on a regular basis were asked to compute the factors of safety for a number of problem situations by means of the well-known methods of analysis for non-circular slip surfaces. Many incorrect answers were received, and this reflected the fact that many of the computer programs being used had never been properly checked or documented. In addition, an alarming number of errors had resulted from careless data input.

8.4 Shear Strength Measurement

Despite the obvious objections to the measurement of shear strengths on residual soils by means of laboratory tests, these still comprise the most satisfactory means of establishing the likely range of shear strengths on the softer materials (grades V & VI). The effects of corestones and other large-sized particles, however, cannot be determined, and there is no doubt that laboratory strength tests carried out on the 'matrix' material of residual soils and colluvium will usually underestimate the shear strength of the insitu material because of the neglect of the boulder content.

Residual soils and colluvium are invariably unsaturated and are of relatively high permeability. Stability computations must therefore always be made in terms of effective stresses; analysis on the basis of undrained strengths has no relevance.

Because of their high permeabilities (usually 10^{-4} to 10^{-6} m/sec.), rainwater infiltrates with ease into most residual soils and colluvium, and it is thought likely that saturation conditions will be approached at shallow depths in the field during the life of a slope. It is therefore generally felt to be appropriate to measure

strength parameters on the basis of shear tests carried out on saturated soil specimens. Although this may often be a more severe condition than that experienced by the soil insitu, it remains the only certain means by which a 'base' shear strength envelope can be established.

The triaxial test is the most widely used method for shear strength measurement on residual soils (Brand & Phillipson, 1985). Test specimens should be as large as possible, full-diameter lengths of drillhole sample being ideal for routine work, and they are usually saturated by the application of a sufficiently high back pressure prior to shear. Either drained tests (CD) or consolidated undrained tests with pore pressure measurement ($\overline{CU}$) can be used, but the latter are much to be preferred, because they are quicker and provide much more information about the stress-strain behaviour of the soil.

Cell pressures used for triaxial testing must relate to the correct insitu stress range if the measured strengths are to be meaningful. Critical slip surfaces in residual soil slopes are most commonly shallow, and the effective stresses on these are therefore low (typically 30 to 200 kPa). At such low effective cell pressures, however, triaxial tests are difficult to control satisfactorily, and this stress range is not recommended for routine use. There is some evidence to suggest that the strength envelopes for some residual soils are curved at low effective stresses, and that the straight-line projection of strengths measured at high stresses underestimates the strengths in the low stress range (Fig. 30). It is almost certain that this is part of the explanation for why many stable residual soil slopes have theoretical factors of safety of less than 1.0.

As discussed in Section 8.5 below, failures of residual soil slopes are invariably caused solely by pore pressure increases. In its simplest form, the mechanism of failure is therefore that the soil suction (i.e. negative pore pressure) decreases (i.e. pore pressure increases), thereby reducing the shear strength of the soil, as dictated by the effective stress principle. Failure will occur when the average shear resistance on the critical slip surface has decreased

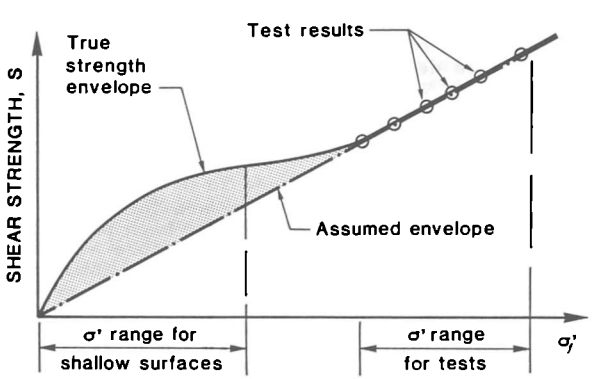


Fig. 30 Underestimate of Shear Strengths because of Incorrect Strength Envelope Deduced from Data Obtained at Too High Normal Stresses

to the value of the average shear stress on that surface. Rain-induced slope failures, therefore, take place under conditions of almost constant total stress and increasing pore pressure (Brand, 1981).

The triaxial test is most commonly conducted by increasing the axial stress, σ_1, to failure while the cell pressure, σ_3, is kept constant. This is carried out either as a drained test ($\Delta u = 0$), or as an undrained test (or constant water content test) with pore pressure measured throughout the shearing process. In contrast, for rain-induced failure, σ_1 and σ_3 are almost constant, and the pore pressure increases to failure. The comparative stress paths for the common triaxial tests and for the real field situation are shown in Fig. 31, in which p' & q are defined as :

$$p' = \frac{\sigma'_1 + \sigma'_3}{2} \quad ; \quad q = \frac{\sigma_1 - \sigma_3}{2} \qquad (2)$$

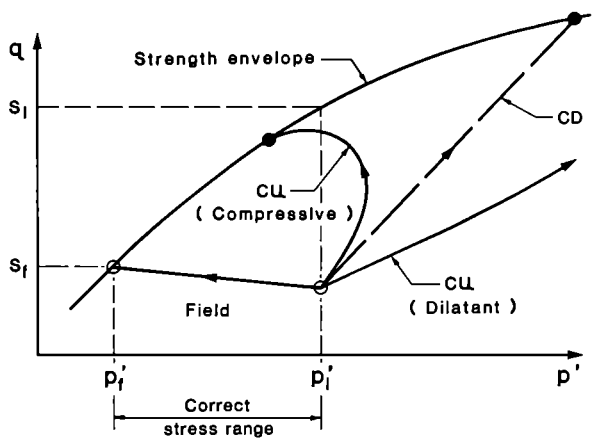


Fig. 31 Comparison between Stress Paths for Rain-induced Slope Failure and Those for Triaxial Tests (Brand, 1981)

Figure 31 illustrates that the stress paths commonly followed in the triaxial test are quite different from that which pertains in the field, and that the stress ranges over which triaxial tests are usually conducted are generally not appropriate to the field stress conditions. Where the strength envelope of a soil is markedly curved, this disparity will result in an appreciable underestimate of the correct shear strength for stability assessment (Fig. 30). It is possible to follow the correct stress path in the laboratory simply by decreasing the cell pressure, but this does not simulate the correct mechanism of failure in the field. This can only be done by means of a constant load test in which the pore pressure is increased from an initial negative value until failure occurs. Tests of this kind have been carried out by the Geotechnical Control Office for some time, but the experimental difficulties are considerable, and this test procedure cannot be recommended for routine use.

Direct shear (shear box) testing is deservedly

becoming more commonly used for the strength assessment of residual soils. Apart from its relative simplicity, this method has obvious merits over the triaxial test when determinations of strengths along relict joints are required, because orientated specimens can be prepared. However, there is no certain method of saturation, and pore pressures cannot be measured during shear. Despite its lack of theoretical 'purity', the direct shear test is seen by some as providing a means of obtaining shear strength data under conditions which model those in the field more closely than the triaxial test. It is also readily adaptable for insitu measurements of shear strength on relatively large masses of material. The equipment shown in Fig. 32 has been used successfully for testing 300 x 300 mm soil and soft rock specimens in Hong Kong (Brand et al, 1983b). A typical test specimen of grade V granitic soil is shown in Fig. 33.

Relict joints and other such discontinuities are worthy of special mention, because they frequently play a major part in slope instability (Hencher et al, 1984). These are often slickensided and sometimes coated with thin deposits of very weak material. The significance of smooth slickensides in decomposed igneous and metamorphic rocks was fully discussed in a paper by St John et al (1969). There have been some reported failures of quite flat slopes attributable to extremely low relict joint strengths. One such well-documented Hong Kong case (Hunt, 1982), which was of major economic significance, led to an extensive investigation into the shear strength of jointed sedimentary volcanic material (Koo, 1982a, 1982b). Persistent joint surfaces were coated with a thin black-brown deposit believed to be the precipitation of iron and manganese oxide products that filled the joints in the parent rock during the course of weathering. The results of laboratory direct shear and triaxial tests showed that the effective strength parameters on the joints were much lower than those measured on the intact material.

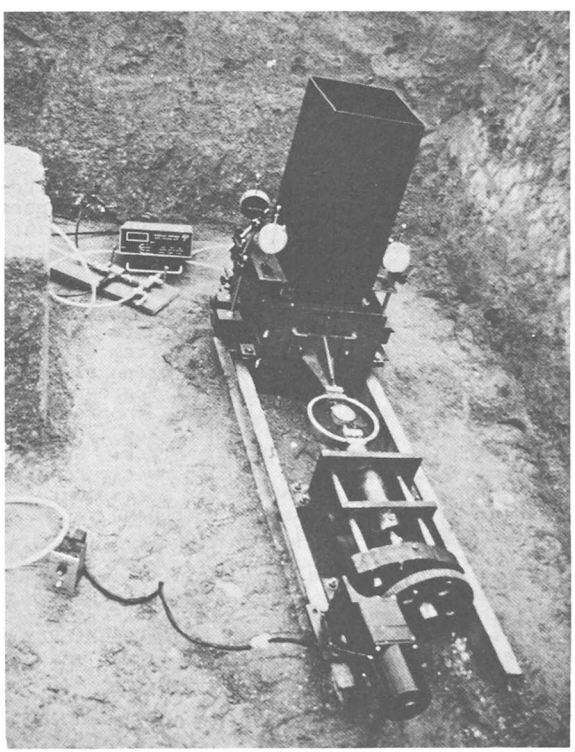


Fig. 32 Insitu Direct Shear Test Apparatus Used on Hong Kong Residual Soil (Brand et al, 1983b)

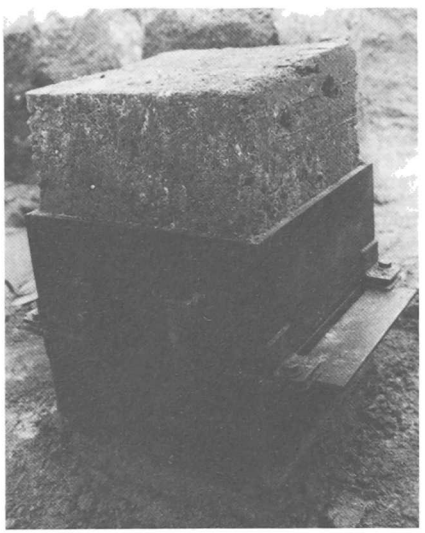


Fig. 33 Block Sample of Grade V Hong Kong Granite Used in the Apparatus Shown in Fig. 32

8.5 Pore Pressure Prediction

For a residual soil slope, the prediction of the pore pressure distribution is by far the most critical factor for stability analysis. This is particularly so since most failures in residual soil slopes are caused by rainfall.

The hydrological effects of rainfall on a permeable slope are depicted in Fig. 34. Some of the water runs off the slope and may cause surface erosion if there is inadequate surface protection. Because of the high soil permeability, however, the majority of the water infiltrates. This causes the water table to rise, or it may cause a perched water table to be formed at some less permeable boundary, usually dictated by the weathering profile. Above the water table, the degree of saturation of the soil increases, and the soil suction (i.e. negative pore pressure) therefore decreases.

Failures in residual cut slopes are thought to be caused mostly by the 'wetting-up' process by which the soil suction (and hence the soil strength) is decreased, but there is some evidence to suggest that transient rises in ground-

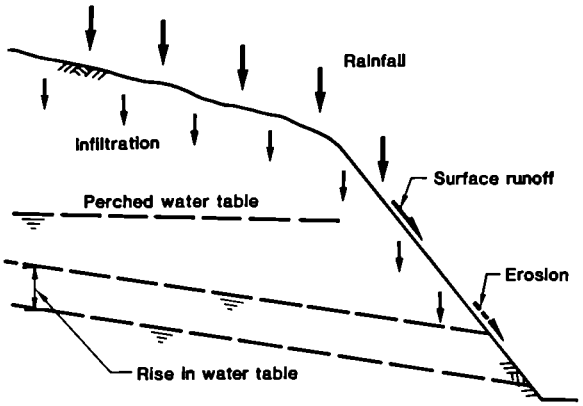


Fig. 34 Diagrammetic Representation of the Effects of Rainwater on a Slope

water tables are responsible for some rain-induced landslides (Premchitt et al, 1985). Rain-induced slope stability failures thus occur as a direct result of pore pressure increases (see Section 8.4), and pore pressure distribution is therefore the variable of most concern.

The pore pressure distribution in a residual soil is dependent on the pattern of rainfall and the hydrogeology of the slope. Pore pressure is therefore a variable which is independent of soil mechanics considerations, being imposed upon a slope by external influences. For this reason, it is extremely difficult to predict the appropriate pore pressures for slope design or stability assessment. This fact is vividly illustrated in Fig. 35, which shows the rapid variation in piezometric head recorded in a Hong Kong slope during a rainstorm in June 1982.

For the infiltration of water into the horizontal surface of a porous medium, Lumb (1962b) derived an expression for the advance of the 'wetting front' (Fig. 36). On the assumption that diffusion is negligible at the end of an intensive

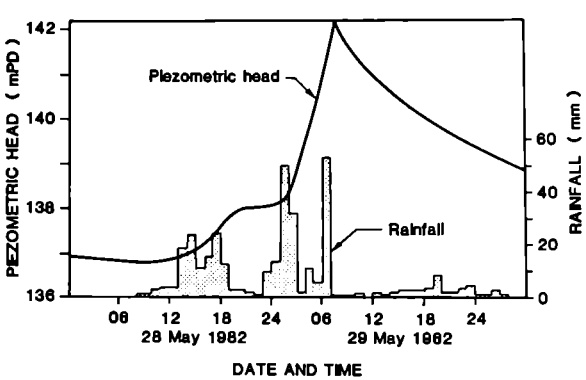


Fig. 35 Rapid Changes in the Groundwater Table Measured in a Hong Kong Slope

rainfall, Lumb (1975) later showed that the thickness, h, of the wetting front after time t could be approximated by :

$$h = \frac{kt}{\eta(S_f - S_o)} \qquad (3)$$

where k is the saturated permeability, η is the porosity, and S_o and S_f are the initial and final degrees of saturation.

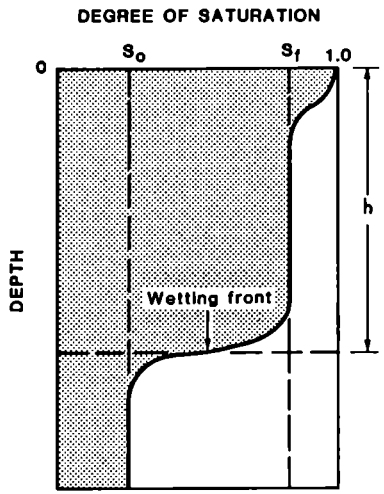


Fig. 36 Representation of the Advance of the Wetting Front as Water Infiltrates into Soil (Lumb, 1962b, 1975)

Despite its shortcomings, equation (3) is frequently used for the prediction of design pore pressures in Hong Kong (Beattie & Chau, 1976; Beattie & Attewill, 1977; Geotechnical Control Office, 1984) by the superimposition of the depth of wetting front onto the groundwater table level at the end of the wet season. For this purpose, a rainfall return period of ten years is commonly employed. This approach suffers from the fact that the value of h is directly proportional to S_f-S_o, and the initial and final degrees of saturation are somewhat speculative. In practice, the Hong Kong wetting band thickness is usually calculated as being about two metres. Such predictions will generally lead to overly conservative slope design and stability assessments, especially if it is assumed that no soil suction exists anywhere in the soil profile.

For the analysis of slope stability in residual soils and colluvium, measured pore pressure data is much to be preferred to the application of uncertain predictive methods. For such data to be meaningful, however, pore pressures must be monitored for a sufficiently long period of time, and piezometers must be installed at the appropriate depths at sufficient locations on the slope. In addition, an appropriate type of instrument must be used which can respond rapidly to pore pressure changes. In Hong Kong, numerical modelling techniques have been applied in a few cases (Leach & Herbert, 1982), but simple

direct correlation and extrapolation methods are more commonly resorted to (Koo & Lumb, 1981; Endicott, 1982).

Because of the rapid changes in pore pressure that occur with rainfall in many residual soil slopes, the critical pore pressures are rarely obtained from normal piezometer measurements. In order to enable standpipe piezometers to record the maximum water levels attained during a given period, Halcrow 'buckets' (British Patent No. 1538487)(Fig. 37) have become popular in Hong Kong. These simple plastic devices are threaded onto a weighted nylon string at selected depth intervals above the normal base water level in a standpipe. When the string of buckets is withdrawn, the highest transient water level is indicated by the upper limit of water-filled buckets. This system is cheap and simple, and it can be used to provide important design information when expensive automatic piezometer systems, of the type described below, cannot be justified.

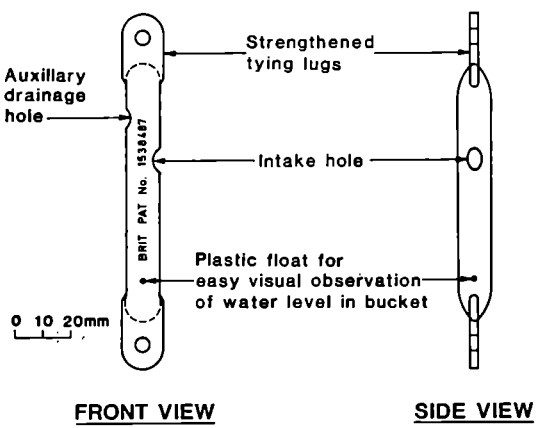


Fig. 37 Halcrow Plastic 'Buckets' Used for Recording the Maximum Water Elevation Attained in a Standpipe Piezometer

During most of the year, suctions exist in many residual soil slopes. These suctions, which can be of high magnitudes, are reduced dramatically by the process of water infiltration during rainfall. There are almost certainly situations where even the heaviest rainfall does not completely destroy the soil suctions, and these continue to contribute to the stability of a slope. Whereas slope design is commonly based on predictions of the maximum positive pore pressures that are likely to develop in the slope, no account is usually taken of negative pore pressures that might be sustained under the worst rainfall conditions. This neglect of the contribution of soil suction to the shear strength of the soil might be the main reason why stable residual soil slopes often have theoretical factors of safety of less than unity.

Programmes of laboratory testing carried out over many years on Hong Kong residual soils have proved that suction pressures act as 'modified' effective stresses, a matric suction of $u_a - u_w$ increasing the shear strength by $(u_a-u_w)\tan \phi^b$, where ϕ^b is the angle of internal friction with respect to matric suction (Fredlund, 1981; Ho & Fredlund, 1982; Fredlund & Rahardjo, 1985). It has been suggested (Sweeney & Robertson, 1982; Boonsinsuk & Yong, 1982) that this increase in soil strength should be permissible for the design of Hong Kong slopes as long as insitu measurements are available to substantiate that suctions continue to exist in the slopes throughout the year.

Much valuable data on soil suctions has been obtained on some Hong Kong slopes over several years. It has been found that the suctions on relatively shallow potential slip surfaces decrease everywhere close to zero during a wet season, and that suction cannot therefore normally be relied upon for slope stability purposes. However, some measurements made at considerable depth in Hong Kong granite (Sweeney, 1982; Sweeney & Robertson, 1982) suggest that not all soil suction is destroyed by infiltration. The extent to which suctions play a significant part in slope stability in residual soil profiles elsewhere in the world clearly depends on the pattern of rainfall and the infiltration characteristics of the material concerned.

Much valuable data on soil suctions can be obtained for shallow depths by means of simple agricultural tensiometers (Anderson, 1984; Greenway et al, 1984). These instruments, which consist of a small porous ceramic tip connected to a perspex tube and a vacuum gauge (Fig. 38), have been found to perform satisfactorily. They suffer from the serious disadvantage, however, that they must be read manually, and critical readings during intensive rainfall are frequently unable to be obtained.

Some sophisticated, automatic reading equipment has been developed to measure both positive and negative pore pressures in Hong Kong conditions

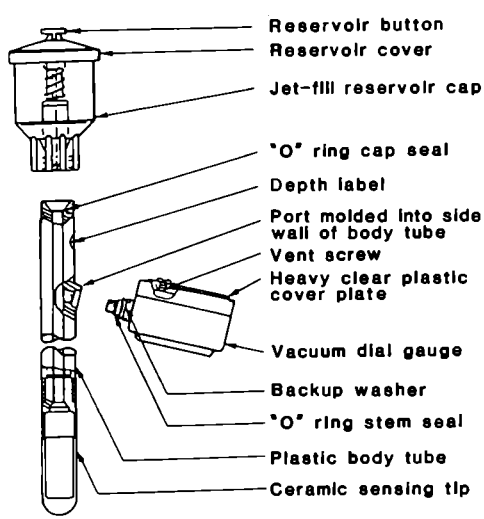


Fig. 38 Agricultural Tensiometer for the Measurement of Insitu Soil Suction

(Pope et al, 1982). This is based on similar equipment designed in the United Kingdom (Anderson & Burt, 1977; Burt, 1978). Small diameter tensiometer tips of high air-entry ceramic are each connected by twin water-filled flexible nylon tubes to a single transducer through a fluid scanning switch (Fig. 39). The switch is analogous to an electrical wafer switch, in that it allows sequential measurement of a number of fluid pressure inputs. A rotating metal valve inside the water-filled wafer (Scanivalve) enables the pressure transducer to be connected to 24 different inputs in turn, two of which are used as reference reservoirs for calibration purposes. The measured pore pressures are recorded on a variety of types of strip-chart recorder, the pressure-sensitive paper type now being regarded as the most reliable. The system is powered by a twelve volt battery, and all measuring and recording components are housed in a water-tight metal box. Although installation and maintenance costs are high, the systems have been found to be reliable for periods of up to three years.

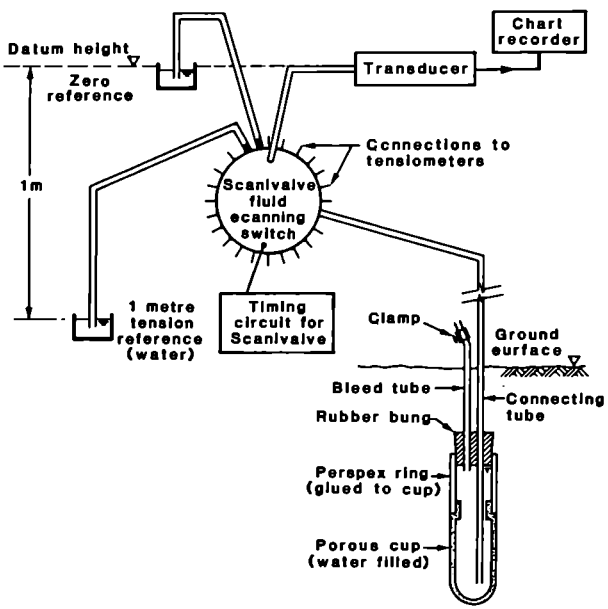


Fig. 39 Automatic Scanivalve System Used in Hong Kong for the Measurement of Positive and Negative Pore Pressures (Pope et al, 1982)

In the context of pore pressure prediction, mention must be made of soil 'pipes' (erosion tunnels) which frequently occur in residual and colluvial terrain. These can vary in size from a few millimetres to perhaps a metre in diameter, and they are capable of very rapidly conveying large volumes of water downslope (Fig. 35). There is no doubt that they are frequently an important factor in slope failures (Pierson, 1983). In Hong Kong, pipes can be of major significance to the hydrogeology of an area (Leach & Herbert, 1982; Nash & Dale, 1983; Premchitt et al, 1985).

8.6 Factors of Safety

Residual soil slopes commonly occur in steep natural terrain, and it is often necessary to work to very low factors of safety in order to avoid the excessive costs of adopting higher values. In these circumstances, it is logical to relate design safety factors to the consequences of failure in terms of risk of casualties and risk of damage to property and services.

Slope design practice in Hong Kong is governed by the Geotechnical Manual for Slopes (Geotechnical Control Office, 1984), which specifies factors of safety. Table VI shows the recommended factors against loss of life and against economic risk for new soil and rock cut slopes. These values are for groundwater conditions resulting from a ten-year return period rainfall. There are three risk categories in each case - 'negligible', 'low' and 'high'. The risk-to-life category reflects the likelihood of loss of life in the event of failure, while the economic risk category reflects the likely magnitude of economic loss. Typical of high risk-to-life slopes are high cut slopes adjacent to occupied buildings. An example of a negligible risk-to-life slope is one which threatens only a lightly trafficked secondary road. Other examples of the two risk categories for Hong Kong conditions are given in Tables VII & VIII.

TABLE VI

Factors of Safety Specified for New Cut Slopes in Hong Kong (Geotechnical Control Office, 1984)

Economic Risk \ Risk to Life	High	Low	Negligible
High	1.4	1.4	1.4
Low	1.4	1.2	1.2
Negligible	1.4	1.2	>1.0

Note: these factors of safety are for a rainfall return period of ten years

Hong Kong's Geotechnical Manual for Slopes also gives guidance on preventive and remedial works to existing slopes. When analysing an existing slope to determine the extent of any preventive (i.e. before failure) or remedial (i.e. after failure) works required, the performance history of that slope can be of considerable assistance to the designer. There is, for example, an opportunity to examine the geology of the slope more closely than for an undeveloped site, and to obtain more realistic information on groundwater. The Manual maintains that the designer is therefore able to adopt with confidence factors of safety for proposed preventive or remedial works that are slightly lower than those specified in Table VI for new works, as long as rigorous geological and geotechnical investigations are conducted (which include a thorough

TABLE VII

Examples of Risk-to-Life Categories for Slope Failures in Hong Kong (Geotechnical Control Office, 1984)

Example	Risk to Life		
	Neg.	Low	High
(1) Failures affecting country parks and lightly used open-air recreation areas.	*		
(2) Failures affecting roads with low traffic density.	*		
(3) Failures affecting storage compounds (non-dangerous goods).	*		
(4) Failures affecting densely used open spaces and recreation facilities (e.g. sitting-out areas, playgrounds, car parks).		*	
(5) Failures affecting roads with high vehicular or pedestrian traffic density.		*	
(6) Failures affecting public waiting areas (e.g. railway platforms, bus stops, petrol stations).		*	
(7) Failures affecting occupied buildings (e.g. residential, educational, commerical, industrial).			*
(8) Failures affecting buildings storing dangerous goods.			*

TABLE VIII

Examples of Economic Risk Categories for Slope Failures in Hong Kong (Geotechnical Control Office, 1984)

Example	Economic Risk		
	Neg.	Low	High
(1) Failures affecting country parks.	*		
(2) Failures affecting rural (B), feeder, district distributor and local distributor roads which are not sole accesses.	*		
(3) Failures affecting open-air car parks.	*		
(4) Failures affecting rural (A) or primary distributor roads which are not sole accesses.		*	
(5) Failures affecting essential services, which could cause loss of that service for a temporary period (e.g. power, water and gas mains).		*	
(6) Failures affecting rural or urban trunk roads or roads of strategic importance.			*
(7) Failures affecting essential services, which could cause loss of that service for an extended period.			*
(8) Failures affecting buildings, which could cause excessive structural damage.			*

examination of slope maintenance history, groundwater records, rainfall records and any slope monitoring records). These reduced factors of safety are shown in Table IX.

8.7 Conclusions

It is clear from the preceding brief review that the application of rigorous soil mechanics methods of slope stability analysis to residual profiles is extremely difficult. It is prudent to adopt this approach only in conjunction with a thorough engineering geological assessment and with the liberal application of sound judgement. Analytical solutions alone cannot be relied upon, as succinctly stated by Peck (1975), thus :

> " Analytical procedures have been developed for calculating the factors of safety of a slope under various conditions ... The theories and their applications have met with some successes and more failures. Most of the failures are a consequence of oversimplification.
>
> Even the most complex theories are necessarily oversimplifications of nature; they are general rather than specific. A theory that would take into account all the significant variables at a given location would be far too complex for use. Furthermore the parameters that must be evaluated to use the theory are not simple invariant quantities, few in number, but are instead complex, highly variable, and usually not constant with respect to time. Frequently the range of variability of the parameters remains extraordinarily large even after completion of exploration and testing that costs as much as the project can possibly afford. "

Experience with the application of soil mechanics methods of analysis to slope failures in Hong Kong enables a more detailed state-of-the-art assessment to be made than that contained in Table V. This has been done for Hong Kong's residual soil slopes by Hencher et al (1984) (Table X), who also suggested where advances could be made in the two particularly poorly understood areas of shear strength and pore pressure prediction (Table XI).

TABLE IX

Factors of Safety Specified for Existing Cut Slopes in Hong Kong (Geotechnical Control Office, 1984)

Risk to Life	High	Low	Negligible
Factor of Safety	1.2	1.1	>1.0

Valuable information for future design purposes can often be obtained from back analyses of slope failures. Some thorough analyses have been carried out for some Hong Kong landslides (Hencher et al, 1984). Such analyses can only be meaningful, however, in circumstances where the majority of factors which contributed to the failure can be positively evaluated, so that the results of the calculations provide an unambiguous measure of shear strength at failure. All too often, back-analyses are conducted without adequate information being available on the mode of failure or on the pore pressures that existed at the time of failure. It is suggested that unsatisfactory isolated analyses of this kind can be harmful to our understanding of slope failure mechanisms, since they purport to provide 'evidence' in support of a design method about which

TABLE X

Assessment of the State-of-Knowledge of the Various Aspects of Slope Stability Predictions for Hong Kong Conditions (Updated from Hencher et al, 1984)

Aspect	Current State-of-Knowledge for Hong Kong Conditions	Overall Rating of Knowledge
Methods of Stability Analysis	Janbu (1954, 1973) method of analysis for non-linear surfaces thought satisfactory. Recommended factors of safety of 1.2 to 1.4 are satisfactory (GCO, 1984). Computational data is often poorly handled (Lumsdaine & Tang, 1981, 1982).	Very good
Geometry of Failure	Pre-failure geometry is easily defined. Often difficult to decide critical potential failure surface for design, especially where geology is complex (Hencher et al, 1984; Hencher & Martin, 1984; Hudson & Hencher, 1984).	Good to Very good
Geology	Site investigation procedures are adequate, but descriptions often poor. Complex weathering profiles are difficult to describe (Hencher & Martin, 1982). Understanding of influence of geological details on hydrogeology is poor.	Fair
Shear Strength	Mass strength as distinct from sample strength is poorly understood. Laboratory tests are commonly used to determine saturated strengths of samples in terms of effective stress, but doubt exists about applicability of test results (Brand, 1981, 1982). Limited amount of insitu strength testing carried out (Brand et al, 1983a). Weakening effect of relict joints recognised (Koo, 1982a, 1982b). Effects of boulder and corestone content unknown (Hencher & Martin, 1982).	Fair to Poor
Groundwater and Pore Pressures	Useful correlations available between landslides and rainfall (Lumb, 1975; Brand et al, 1984). Rapid changes in pore pressure with rainfall are very difficult to predict for design (Anderson et al, 1983; Premchitt et al, 1985). Only limited attempts made to model groundwater (Leach & Herbert, 1982). Extrapolation of insitu measurements seems best design approach (Koo & Lumb, 1981; Endicott, 1982). Some progress made with field instrumentation (Pope et al, 1982; Brand et al, 1983a). Erosion pipes are important in transmitting water (Nash & Dale, 1983; Brand et al, 1986).	Poor

TABLE XI

Suggested Advances which Can Be Made in the State-of-Knowledge of Slope Stability Predictions for Hong Kong Conditions (Updated from Hencher et al, 1984)

Methods / Key Areas	Field Observations	Testing		Theoretical Studies	Back Analysis
		Field	Laboratory		
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	Considerable advances can be made in our understanding of the shear strength behaviour of unsaturated residual soils	Advances can be made in establishing models for the mass behaviour of materials	This is the only means of checking validity of relationships between properties
Groundwater and Pore Pressure	These are useful for checking theories and methods of prediction	Case studies relating subsurface profiles to hydrogeology and infiltration characteristics can lead to improved predictions	Not applicable	Advances are possible, e.g. development of more sophisticated infiltration and hydrogeological models	This is the only means for checking that the methods of incorporating groundwater into design are satisfactory

the analyst often has preconceived ideas. It is important to guard against these dangers inherent in the back-analysis of failures, as emphasized by Leroueil & Tavernas (1981). There is no doubt, however, that a large number of case histories of failures in a particular location provides a good basis for future design, in that they can be used as the input to a semi-empirical method (see Section 7).

9. SOME HONG KONG CASES

9.1 Introduction

In establishing his framework by which to judge soil mechanics predictions, Lambe (1973) recognised three main categories of prediction, namely: type A predictions (made before the event), type B predictions (made during the event), and type C predictions (made after the event). For obvious reasons, type A predictions are thought to be the only true predictions, and these are certainly the most valuable kind for assessing the reliability of our predictive methods. Type C predictions are regrettably the most common type reported in the published literature, and alone they provide unconvincing confirmation of the accuracy of our predictive tools. However, these 'post-mortem' predictions can be extremely valuable where they are accompanied by full details of the engineering design carried out before construction.

Mention has been made in Section 8.7 of the detailed back analyses carried out on some slope failures in Hong Kong (Hencher et al, 1984). These were part of a continuing programme to evaluate the engineering design and analysis methods currently in use and specified in the Geotechnical Manual for Slopes (Geotechnical Control Office, 1984). This continuing evaluation process is vitally important in Hong Kong, where the design factors of safety are necessarily very low (Table VI).

In order to illustrate the state-of-the-art with respect to predicting the performance of residual soil slopes, details will be given here of the performance of six selected Hong Kong cut slopes. These fall into the two categories :

(a) slopes which have remained stable even though the present theoretical factors of safety are less than 1.0 (Cases 1, 2 & 3), and

(b) slopes which have failed, even though the 'design' factors of safety were above 1.0 (Cases 4, 5 & 6).

The slopes represented by Cases 1 to 3 were formed some years ago on the basis of precedent, no engineering design methods having been used. Cases 4 to 6, on the other hand, involved the latest analysis and design procedures. All six cases depict common types of cut slope in Hong Kong, and they together represent a small proportion of the examples that could be used to illustrate the same points.

9.2 Stable Slopes with F <1.0

The three stable slopes (Cases 1 to 3) were studied as part of a Territory-wide appraisal of existing cut slopes carried out between 1979 and 1981. Each slope was the subject of detailed assessment, which included a review of the slope's history, a detailed site survey, and an appropriate site investigation and testing programme. Holes were drilled to supplement the surface geological examinations and to obtain samples, and piezometers were installed to monitor pore pressures throughout one or two wet seasons. Triaxial tests were carried out in the laboratory to obtain good quality shear strength data.

The three slopes were all analysed using the Janbu routine method (Janbu, 1954, 1973) applied to a large number of slip surfaces chosen on the basis of the site geology as revealed by the field studies. The minimum calculated factor of safety was in each case significantly lower than 1.0. In May 1982, August 1982 and June 1983, severe rainstorms occurred (Table III) which between them caused well over 2 000 slope failures, but the three slopes in question did not fail, and they remain stable to this day.

Case 1 slope is shown in the simplified cross-section in Fig. 40. It has a height of about 36 m from the flat open space below. Deep excavations into the original ground surface left a relatively thin layer of granitic soil (grades IV, V & VI) over the granite bedrock. The slope surface is covered partly by vegetation and partly by lime-stabilised soil ('chunam'). The water table, as determined by piezometer installations, is low even during the wet season, no positive pore pressures having been measured in the soil.

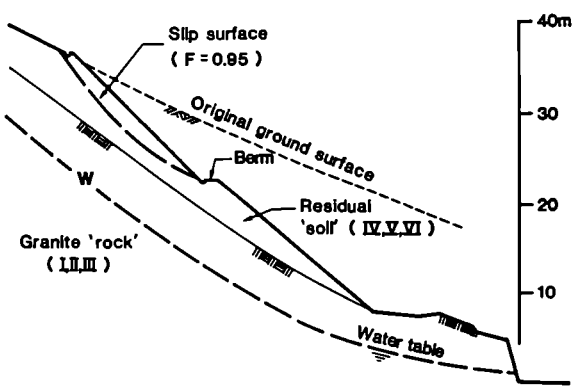


Fig. 40 Case 1: Stable Cut Slope with F <1.0

The measured shear strength parameters for the material of the Case 1 slope vary between c' = 14 kPa and ϕ' = 45° for the grade IV material, and c' = 0 and ϕ' = 42° for the grade VI material. On the basis of these parameters, the factor of safety for the steep upper part of the slope, above the berm, is calculated to be 0.95, with the critical slip surface being very shallow and passing largely through the grade V material. The detailed cross-section of the 'unstable' part of the slope (Fig. 41) clearly illustrates the importance of the weathering profile on stability predictions for slopes of this kind.

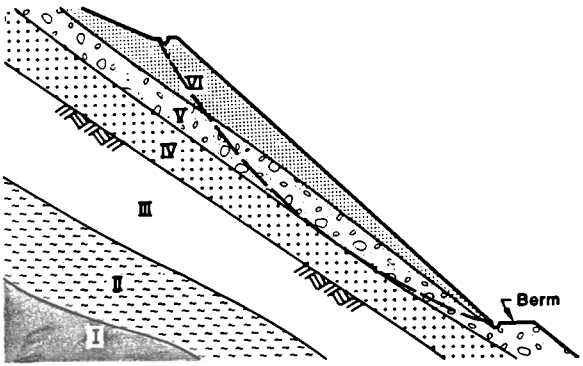


Fig. 41 Detailed Weathering Profile for the Case 1 Slope Showing the Grades IV, V & VI Soil

Case 2 involves a cut slope in granitic soil which was formed as part of a combined site formation for a housing block and a road (Fig. 42). The steep cut slope, which is about 18 m high and coated with chunam, is retained at its toe by a 3 m concrete retaining wall. The very thick deposit of granitic soil is predominantly of grade V, with shear strength parameters of c' = 5 kPa and ϕ' = 35°. The water table remains low throughout the wet season. The calculated factor of safety on the deep critical slip surface shown in Fig. 42 is 0.82

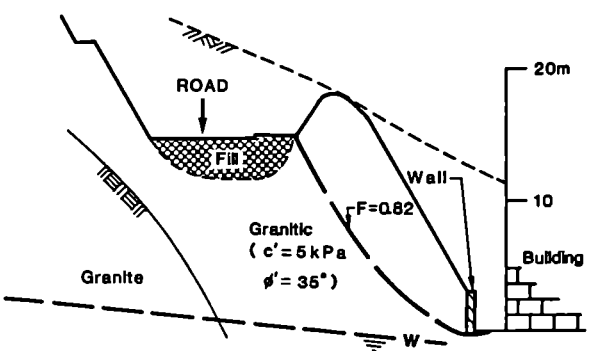


Fig. 42 Case 2: Stable Cut Slope with F <1.0

Case 3 involves a 30 m high slope (Fig. 43) cut in granitic soil with a thin covering of colluvium over its lower half. At the toe of the slope is a 5 m high masonry retaining wall, and above the slope is a road. The surface of the slope is protected partly by chunam and partly by vegetation. The granitic soil varies from grade IV to grade VI, the vast majority being of grade V with strength parameters of c' = 2 kPa and ϕ' = 39°. At its highest, the water table is located many metres below the top of the slope, but it is close to the surface of the bottom part of the slope. The results of the stability analyses are depicted in Fig. 43. Several slip surfaces give factors of safety which are less than 1.0, the critical surface, which is very

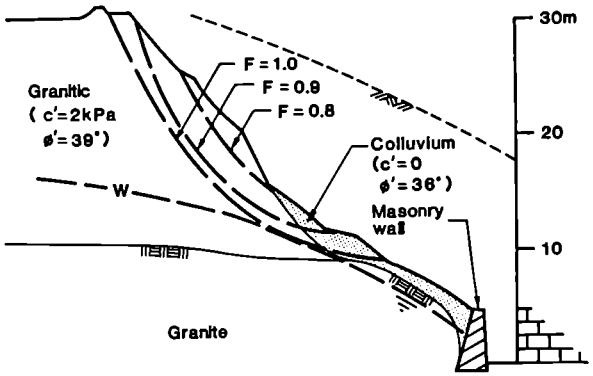


Fig. 43 Case 3: Stable Cut Slope with F <1.0

shallow, having a factor of safety of only 0.80.

9.3 Unstable Slopes with F >1.0

Examples will be given of three cut slopes which were designed to be adequately stable and which subsequently failed during the severe rainstorms of 1982 and 1983 (Table III). These will be referred to as Cases 4, 5 & 6.

Case 4 cut slope, shown in Fig. 44, was designed and constructed in 1980-81 as part of a new road scheme. What was thought to be adequate site investigation was carried out at the time in the form of drilling, sampling and strength testing. This indicated that the granite rock was overlain by a relatively thin layer of grades IV, V & VI materials. Piezometers installed in drillholes showed the water table to be quite close to the ground surface in the upper part of the slope, and two rows of horizontal drains were included at mid-slope as part of the design. On the assumption that the horizontal drains would control the maximum level of the water table, the critical slip surface was found to be as shown in the failure. The measured shear strengths of c' = 10 kPa and ϕ' = 35° yielded a design factor of safety of 1.24, which was considered adequate for the risk category concerned.

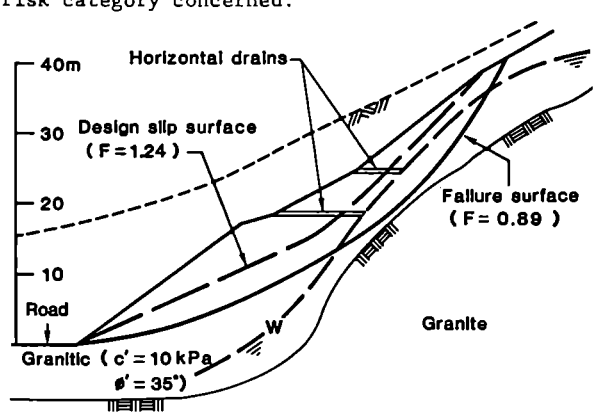


Fig. 44 Case 4: Unstable Cut Slope Previously Predicted as Being Stable (F = 1.24 ?)

After the heavy rainstorm at the end of May 1982, failure of a large portion of the Case 4 slope occurred, as can be seen in the aerial photograph in Fig. 45. The failed mass measured approximately 70 m along the slope and 40 m from top to bottom, the deduced failure surface being as shown in Fig. 44. It should be noted that, unlike the vast majority of Hong Kong landslides, failure did not occur suddenly and dramatically, but took place over several days, and the large mass of failed material remained on the slope. The failure was extensively investigated, many boreholes being drilled to obtain a much more detailed geological profile than that on which the design was based. Reanalysis of the slope on the basis of the actual failure surface, which was much deeper than the design critical surface (Fig. 44), gave a range of F-values from 0.82 to 1.0 depending on the water table level, with the most probable value being F = 0.89. The crucial design error, therefore, was a misinterpretation of the weathering profile because of insufficient borehole data.

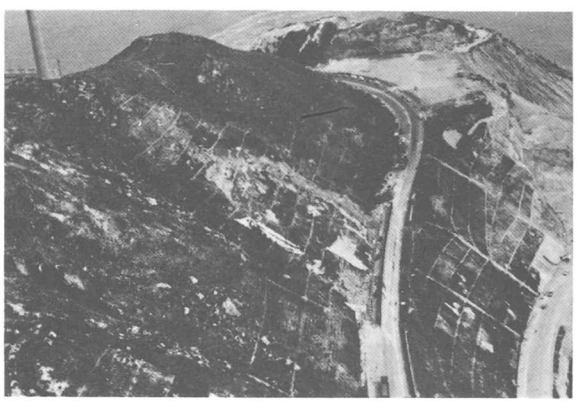


Fig. 45 Aerial View of Case 4 Slope after Failure

Case 5 concerns a very large cut slope formed in the late 1960s for the construction of a road. At its highest point, the crest of the slope is 90 m above the road level. The slope was originally cut to a 'standard' angle without the benefit of investigation or design. A number of shallow failures occurred at various points on this long slope during 1973, 1974 and 1976, as a result of which a limited amount of investigation was conducted to enable remedial works to be designed and carried out. These took the form of cutting back together with the installation of some horizontal drains in the failed areas.

Figure 46 shows a section of the upper portion of the Case 5 slope as it was in 1981. Several metres of colluvium overlay about 12 m of grades V & VI granitic soil. A comprehensive site investigation on the slope was carried out in 1981-82 to reassess its stability. This included the installation of piezometers and a thorough programme of laboratory shear strength testing. The strength parameters were measured as c' = 4 kPa and ϕ' = 36° for the colluvium, c' = 6 kPa and ϕ' = 40° for the grade VI material, and c' = 20 kPa

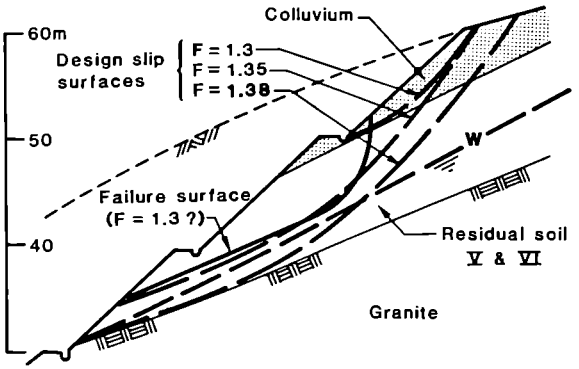


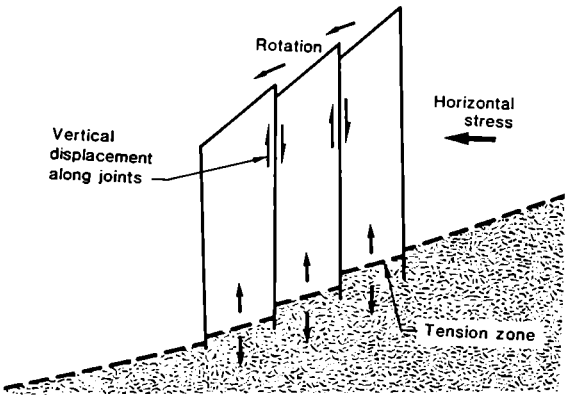
Fig. 46 Case 5: Unstable Cut Slope Previously Predicted as Being Stable (F = 1.3 ?)

and ϕ' = 40° for the grade V material. On the basis of this data and the maximum groundwater elevation recorded during 1981-82, stability analyses gave a 'design' factor of safety in excess of 1.3.

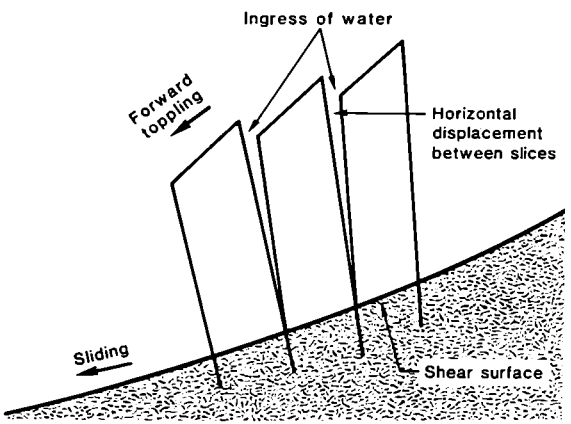
A portion of the slope, 30 m high and 15 m long, failed during the rainstorm of August 1982, and a further study was then carried out to investigate the cause. The failure surface was accurately determined to be as shown in Fig. 46, for which the calculated factor of safety was 1.3, unless considerably higher pore pressures than those measured were assumed in the calculations. Of major importance was the presence of almost vertical slickensided relict joints visible at the failure scarp, which could have resulted in the joint-controlled failure mechanism depicted in Fig. 47. In addition, soil pipes (see Section 8.5) discovered in the vicinity could have transmitted large quantities of water to the slip surface to bring about localised transient pore pressures higher than those measured by the piezometers. It is therefore difficult to envisage how this slope failure could possibly have been predicted on the basis of available soil mechanics methods.

Case 6 is a 70 m high cut slope in volcanic soil constructed in 1974-77 as part of the Tuen Mun Highway, which features in Section 10 below. No proper investigation or stability analysis was carried out before construction. Failure of the slope first occurred in 1975, and this led to remedial works, which included cutting back the slope and the installation of some horizontal drains, to leave the slope as shown in the cross-section in Fig. 48. On the basis of site investigation data collected for the remedial works, a full stability analysis was later carried out in 1978, to give F = 1.3 on the 'design' critical slip surface shown in the figure. In September 1983, an extremely large failure, about 10 000 sq.m in area (Fig. 49), occurred on the slope.

A very detailed geotechnical study was carried out on the Case 6 failure. The many boreholes drilled (Fig. 49) indicated that the bedrock (grades III or stronger) was much deeper over most of the slope than assumed in the 1978 analysis. Inspite of this, borehole data and



(a) Rotation and Displacement of Joints



(b) Ingress of Water through Open Joints

Fig. 47 Possible Failure Mechanism of Case 5 and Case 6 Slopes

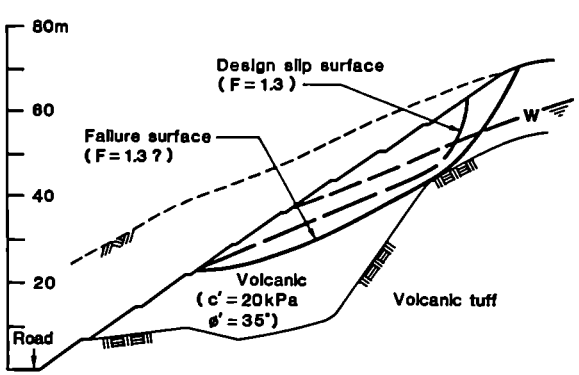


Fig. 48 Case 6: Unstable Cut Slope Previously Predicted as Being Stable (F = 1.3 ?)

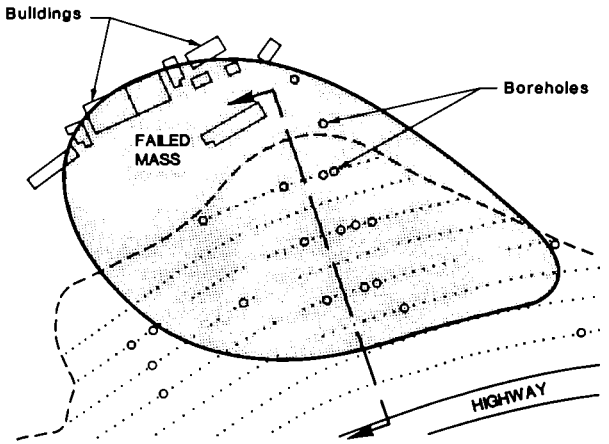


Fig. 49 Plan Showing the Extent of the Case 6 Failure

inclinometer readings determined the failure surface as being fairly close to that predicted in 1978 (Fig. 48), although the back scarp was appreciably steeper. This fact appeared to be related to the subvertical relict joints in the decomposed volcanic soil which were crucial in dictating the mode of failure. The computed F-value for the actual failure surface, however, was still about 1.3 on the basis of strength data obtained from good quality laboratory triaxial tests. Like Case 5, it is thought that gradual slope 'deterioration' caused by the ingress of water along the relict joints could have been the main cause of failure. Piezometers installed for the back analyses gave measured groundwater levels very close to those assumed in the original 1978 stability calculations.

9.4 Discussion

The six Hong Kong examples described above illustrate our apparent inability to accurately predict the failure of residual soil slopes. On the one hand, the classical soil mechanics methods of stability analysis would seem to be conservative, as evidenced by the Cases 1 to 3 slopes which have remained stable with factors of safety of less than 1.0. On the other hand, Cases 4 to 6 suggest that the same methods overestimate the F-values for some slopes which are in fact unstable.

There are two possible contributory factors to the incorrect predictions that the Cases 1 to 3 slopes should have failed, namely :

(a) the insitu mass shear strengths are higher than those measured in the laboratory, for the reasons discussed earlier in Section 8.4, and

(b) soil suctions are sustained in the slopes throughout the heaviest rainstorms which are sufficient to render them stable.

The usually dominant part played in slope stability calculations by positive pore pressures

cannot in any way be the cause of error in these three cases, since the pore pressures were zero everywhere on the determined critical slip surfaces.

For soil suction alone to be the reason why the three slopes have remained stable (i.e. F >1.0), suction values greater than about 2 kPa, 16 kPa and 8 kPa would need to be sustained in the three slopes respectively throughout severe rainstorms. In order for the slopes to possess factors of safety of 1.4, as specified in Hong Kong for high risk situations, the soil suctions required are about 8 kPa, 45 kPa and 28 kPa respectively. As mentioned earlier, it is unlikely that suctions of these large magnitudes could exist at such shallow depths in these slopes during severe rainstorms. However, it is probable that the soil never becomes fully saturated insitu, in contrast to the full saturation state usually imposed for laboratory shear measurements.

Where slopes fail with 'design' factors of safety which exceed 1.0 (Cases 4 to 6), the role of geological detail appears to be paramount, and this cannot be readily accounted for in our routine methods of slope analysis and design. It is not uncommon in weathered profiles for the geological complexity to be such that our soil mechanics approach can only be applied successfully with hindsight. An example of such a situation is shown in Fig. 50, for which back analyses have been carried out by Hencher & Martin (1984).

It is noteworthy that none of the three slopes represented by Cases 4, 5 & 6 failed dramatically, but all involved slow and relatively small displacements of slope material which in each case remained on the slope surface. This suggests that a calculated value of 1.3 might be the lower bound F-value to ensure long-term stability in all Hong Kong's large soil cut slopes.

Of importance is the fact that there are no recorded failures of Hong Kong slopes for which the F-values predicted on the basis of state-of-the-art soil mechanics methods have exceeded 1.4. This gives good support to the use of a minimum factor of safety for high risk situations of 1.4, as recommended by the Geotechnical Manual for Slopes (Geotechnical Control Office, 1984) (Table VI).

10. PROBABILITY OF SLOPE FAILURE

10.1 Prediction Quality

The calculated factor of safety for a slope is a way of representing the probability of failure. Engineering design aims to produce a low probability of failure combined with a high degree of economy. A perfect prediction method would result in the failure of all slopes with F <1.0 and no failures of slopes with F >1.0. In other words, the probability of failure for F <1.0 should ideally be 1.0, and the probability of failure for F >1.0 should be 0. This ideal situation is

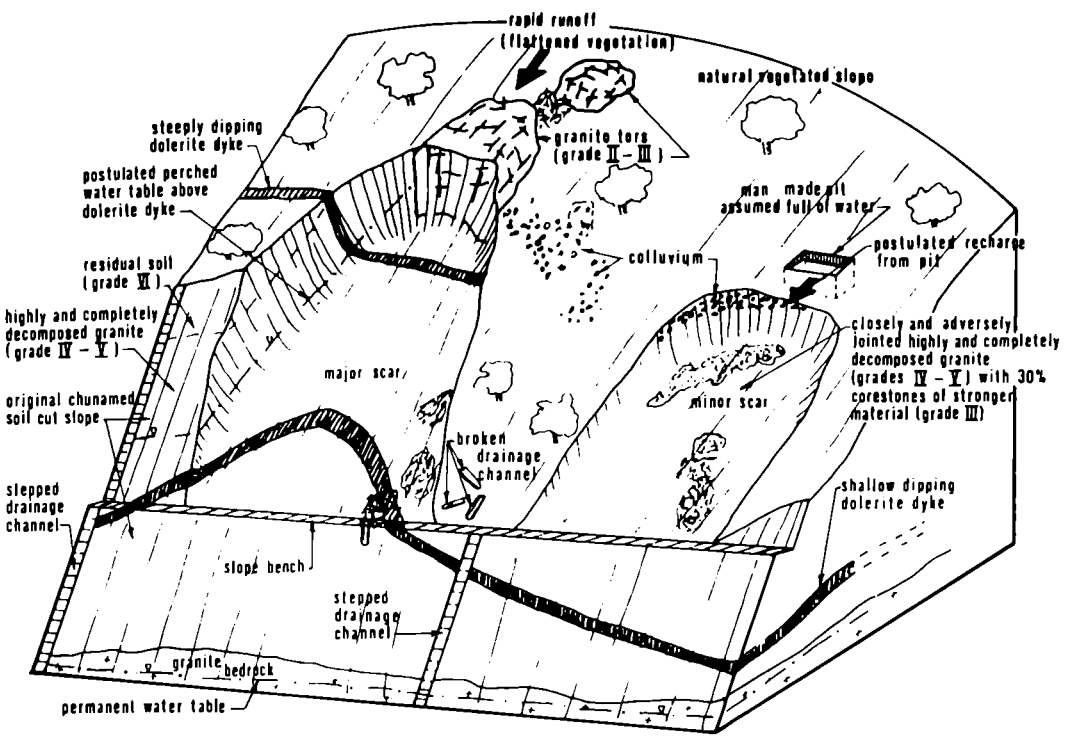
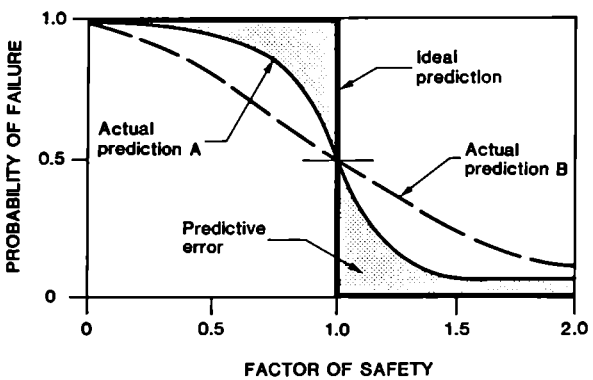
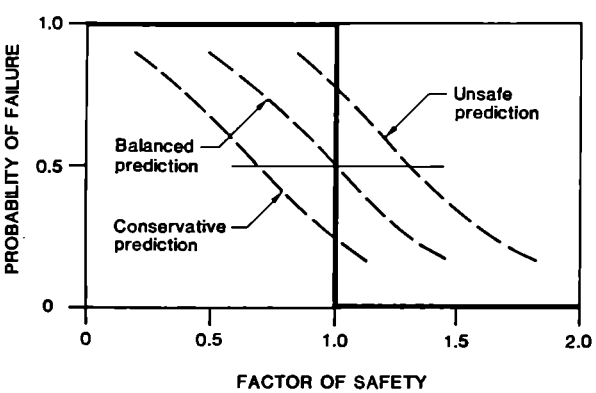


Fig. 50 Diagram Showing the Complex Geological Conditions at the Failure Site Studied by Hencher & Martin (1984)

depicted in Fig. 51(a), which shows the probability of failure as a function of theoretical factor of safety. In practice, the ideal prediction is not achievable, but predictions based on the best slope design methods could be expected to yield results represented by the probability curves A & B. While prediction A is clearly superior to prediction B, both are 'balanced' predictions, in the sense that each gives a probability of failure of 0.5 for F = 1.0.



(a) Ideal and Actual Balanced Predictions



(b) Conservative and Unsafe Predictions

Fig. 51 Reliability of Slope Stability Predictions in terms of Probability of Failure and Theoretical Factor of Safety

Where our predictive tools are not adequate to enable a balanced prediction to be made, the probability of failure for a designed slope could be as shown in Fig. 51(b). The balanced prediction curve moves to the left for a conservative prediction, and to the right for an unsafe prediction. A conservative prediction is one for which the probability of failure for a slope with F = 1.0 is less than 0.5; an unsafe prediction is one for which the probability of failure for F = 1.0 is greater than 0.5. As our prediction capabilities improve, the prediction becomes steeper and approaches the ideal prediction situation depicted in Fig. 51(a).

10.2 A Hong Kong Example

In order to evaluate, in a general way, our ability to predict slope failures in residual soils, the probability of slope failure will be examined for the Tuen Mun Highway in Hong Kong. This major highway, which was constructed over a number of years commencing in 1974, runs for about 15 km along the southwest coast of the mainland portion of the Territory. A large number of cut slopes were required to be formed in steep natural hillsides, and this was done on the basis of the standard 'design' geometry of 5 vertical to 3 horizontal (about 60°), with 1.5 m wide berms at 7.5 m vertical intervals. Provision was made for any slope to be flattened if it proved to be unstable during construction.

The majority of the Tuen Mun Highway slopes are cut in weathered granite, but a few at the western end are in weathered volcanic rocks. Most of the slopes have exposed rock (grades I to III) at the toe with residual soil (grades IV to VI) above. Of the rest, a few are cut entirely in rock, while the others are composed of residual soil throughout. While the slopes can therefore be expected to vary considerably in their stability, they have all been in existence for the same length of time and they have all experienced the same rainfall history.

The Territory-wide slope survey carried out in 1977-78 recorded 78 cut slopes higher than 5 m along the Tuen Mun Highway. Fourteen of these were selected for detailed geotechnical studies, which included full stability analyses. The distribution of the calculated factors of safety for these is shown in Fig. 52. Half of the slopes had factors of safety greater than 1.4, while the others had F-values which varied from 1.1 to 1.4. No stability analyses were carried out on the other 64 slopes along the Highway, but the crude assumption will be made here that the distribution of F-values for these slopes would be similar to the distribution for the fourteen shown in Fig. 52.

The Tuen Mun Highway slopes have endured six major rainstorm events since they were constructed (Table III). The rainstorms of 1982 and 1983 were particularly severe tests of their stability. Four slopes failed in the storm of May 1982 and another (Case 6, Section 9.3) in September 1983,

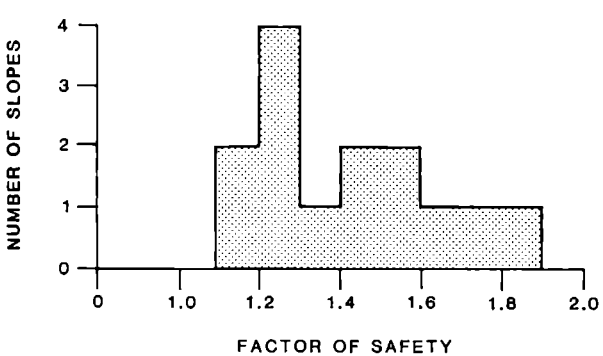


Fig. 52 Distribution of Factors of Safety for the 14 Tuen Mun Highway Slopes Studied in Detail

which all caused disruption to traffic for many days. During the highway's first seven years of life, therefore, five major failures occurred in the 70 slopes, which gives a rough annual probability of failure of about 1 in 100 for each slope.

The rough annual probability of slope failure on the Tuen Mun Highway can be compared with the probabilities quoted by Whitman (1984) for other types of geotechnical failure. Figure 53 shows Whitman's plot of probability of failure against consequence of failure for the situation in the United States. To this has been added a zone to represent the group of Tuen Mun Highway slopes. Whitman's acceptability criteria, as represented by straight lines in the figure, suggest that the Tuen Mun Highway situation would represent an acceptable risk in the USA and would be little different from the risk accepted for foundation failure.

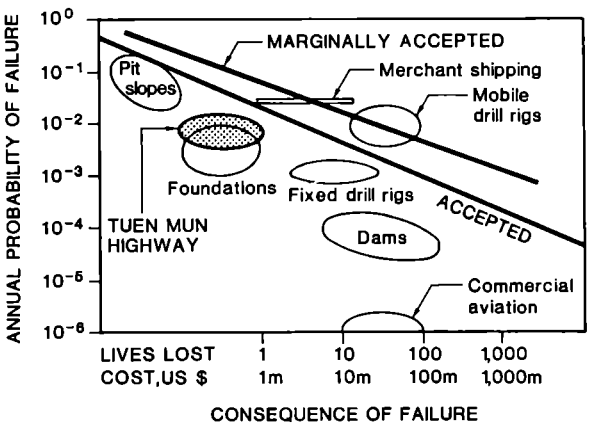


Fig. 53 Annual Probabilities and Consequences of Geotechnical Failures in the USA (Whitman, 1984) Compared with Those on the Tuen Mun Highway

The Geotechnical Control Office has established a comprehensive catalogue of approximately 10 000 large cut slopes in Hong Kong, the majority of which were constructed before engineering design methods were adopted. If the probability of failure of 1 in 100 for each Tuen Mun Highway slope is representative of the slope stability situation overall in Hong Kong, then approximately 100 slope failures can be expected in an average year. This figure is in fact not far in excess of the average number of significant failures recorded annually over the last 20-year period (Table III). The situation is improving dramatically, however, because of the high standard of geotechnical control which is now applied to the design and construction of all new cut slopes, which are required to comply with the Geotechnical Manual for Slopes (Geotechnical Control Office, 1984). In time, it is envisaged that the overall probability of failure will tend towards a reduction by an order of magnitude (from 10^{-2} to 10^{-3}), in much the same way as Peck (1980) hoped for a reduction from 10^{-4} to 10^{-5} in the annual probability of failure for the world's dams.

11. CONCLUSIONS

It has been shown that there are several basic approaches that can be used for the design and stability assessment of residual soil slopes. For most residual soil problems, the traditional division between the methods developed separately for soil mechanics, rock mechanics and engineering geology is not meaningful, and a unified approach is necessary for good engineering design. Because of the nature of residual materials, the application of soil mechanics methods alone will generally not be adequate. The selected use of terrain evaluation has much to recommend it, especially for planning and feasibility studies of relatively large land areas. In addition, the establishment of semi-empirical design guidelines will often be invaluable where there is adequate information available on the performance of existing cut slopes over a sufficiently long period of time.

The state-of-the-art with respect to the application of soil mechanics methods to the analysis of residual soil slopes is generally poor. Whereas some simple situations readily lend themselves to solution by analytical means, the often complex geological conditions which exist in residual soil slopes usually preclude the sensible application of the well-known methods of stability analysis. Major difficulties invariably arise with the selection of a suitable mode of failure, with the meaningful measurement of shear strengths and, particularly, with the prediction of the most unfavourable pore pressures which are likely to occur during the life of the slope.

There is considerable scope for improvements to the application of analytical methods for the prediction of slope performance. Long-term programmes of research and development are required into the shear strength behaviour of residual soils and into improved methods for predicting the pore pressures induced by rainfall. Most importantly, good case histories are badly needed which give detailed comparisons of stability analyses and slope performance. There is no doubt that the available analytical methods have far out-striped our ability to observe, measure and interpret data. As Peck (1975, 1980) has commented, analytical methods of design are usually too heavily relied upon by the engineer at the expense of the application of engineering judgement based on sound experience.

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