Geotechnical Study of a Large Hillside Area in Hong Kong

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This paper describes part of a major study into the stability of the lower slopes of Victoria Peak, which consist of deeply weathered granitic and volcanic soils covered by colluvium. The depth of colluvium and residual soil ranges up to 90 m.

The study has involved an extensive field and laboratory research programme, since current site investigation techniques and state-of-the-art knowledge on the behaviour of saturated and unsaturated residual soils does not satisfactorily explain phenomena observed in the field. The paper describes the methodology of the following particular topics:

(a) A field study of the hydrogeological behaviour of the multi-layer system of colluvium, residual soil and bedrock, in which ground water can respond very rapidly to rainfall and in a different mode in each of the layers. Automatic and continuously monitored piezometers were installed in order to measure the rise and decay of piezometric pressures during the sequence of rainfall events in the wet season.

(b) A field study of soil suction and its variation with local topography and rainfall. The suction measurement systems are partly automatic and they continuously monitor large arrays of tensiometers installed in hillside spur and hollows to depths of 10 m.

(c) A laboratory study of the shear strength of the residual soils and colluvium in both saturated and unsaturated conditions.

INTRODUCTION

In May 1979, following a survey of all slopes and retaining walls in Hong Kong, a temporary ban was placed on further developments in an area, 1.5 km x 1 km, lying on the northern slopes of Victoria Peak. The ban was made for geotechnical reasons, as the area had a history of landslides. The lower slopes had already been intensively developed with multi-storey residential property, but there was still potential for considerable redevelopment.

A detailed geotechnical study was immediately put in hand to assess the stability of the slopes and to provide geotechnical guidelines for future developments in this area. The area covered by the study is shown in Figure 1. A little over two years was allowed for the study. Because of the limitations of current site investigation techniques and state-of-the-art knowledge concerning unsaturated residual soils, the study has involved extensive field and laboratory research work.

CLIMATE

The mean annual rainfall is approximately 2 225 mm, of which about 1 800 mm falls between May and September. Periods of heavy rain may continue over several successive days, with total rainfalls of several hundred millimetres. Thus, during the hot, wet season the soils on slopes undergo successive cycles of saturation and drying.

GEOLOGY

Figure 2 shows two typical cross-sections for the eastern and western sides of the study area. The natural slopes are inclined at approximately 40° in the steep upper slopes but flatten out to 25° and less in the developed lower slopes. Failures in natural slopes and cuttings have always been a common occurrence in Hong Kong during periods of heavy rainfall. Within the study area, two major landslides occurred in 1925 and 1972, respectively, involving the loss of about 70 lives in each case. However, most failures in the soils are typically shallow (Lumb, 1975).

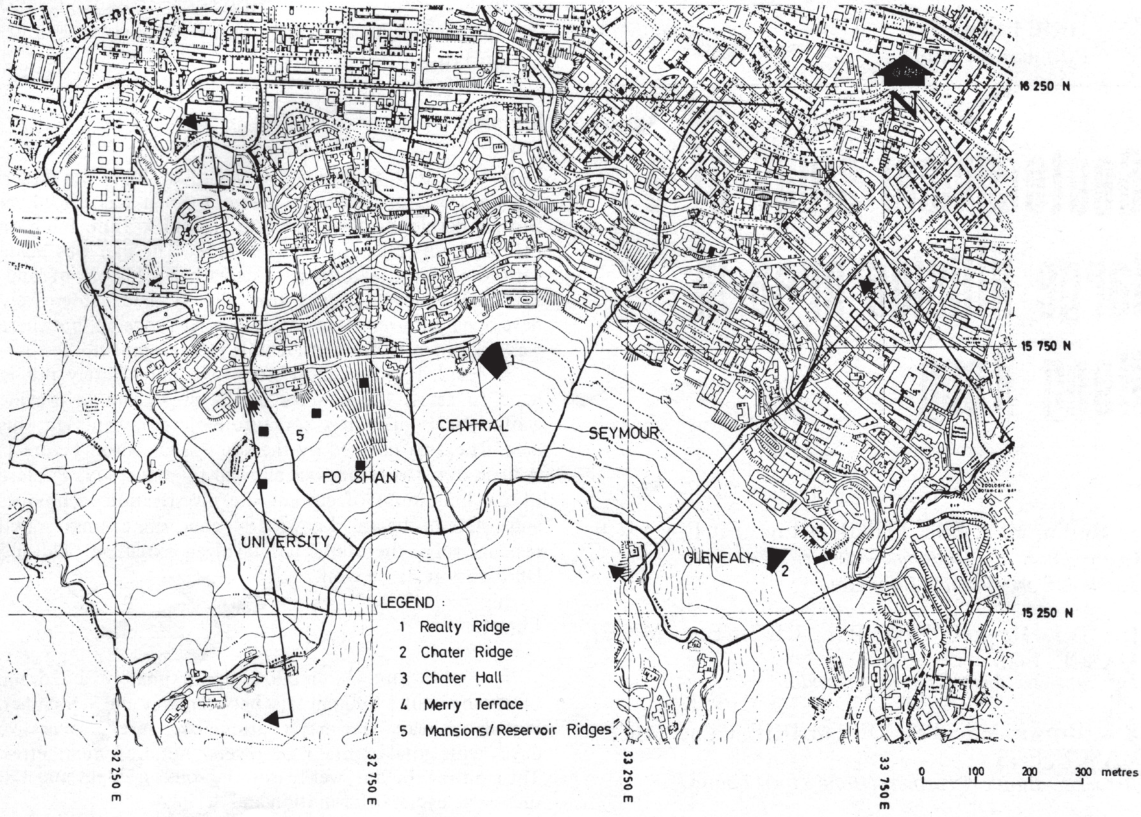

Figure 1. Study area

The rock consists of volcanic ash deposits which were folded and later intruded by a large body of granite. Most of the granite has been eroded leaving the more resistant volcanic rocks capping the hills. The granite is very uniform in its pink colour and its grain size. In contrast, a variety of volcanic rock types is present.

The depth of decomposed rock varies considerably due to faults and changes in rock types, but generally increases downslope from virtually none on the upper slopes to a thickness of up to 90 m beneath the developed area.

The degradation of Victoria Peak is a continuing process, and products of erosion of the upper slopes (colluvium) now mantle the lower slopes. This erosion takes the form of either falls of boulders from exposed rock cliffs or landslides of soil and rock mixtures. The colluvium generally consists of cobbles and boulders set in a matrix of gravelly or sandy silt. Its thickness varies considerably but is generally between five and 20 m. In places, streams have cut through the colluvium and form the present natural drainage lines.

The colluvium was found to be very variable, but three broad classes were designated based on the matrix density, colour and state of decomposition of cobbles and boulders. The definitions are as follows:

Class 1 (oldest)

Matrix - stiff to very stiff, mottled dark red and yellow brown slightly clayey sandy silt, with some gravel in the east.

Cobbles and Boulders - commonly comprising 75 to 100 per cent but may be as low as 25 per cent, subrounded minor angular, mainly highly to completely decomposed, some moderately decomposed, no patinas.

Class 2 (intermediate age)

Matrix - firm to stiff, mottled dark red and yellowish brown, clayey sandy silt, with some gravel in the east.

Cobbles and Boulders - commonly comprising 0 to 50 percent and up to 80 per cent, subangular to subrounded, mainly moderately to highly decomposed, occasionally slightly decomposed. Patinas commonly 30 to 60 mm thick and may be up to 100 mm thick.

Class 3 (youngest)

Matrix - soft to firm, uniform pale brown to yellowish brown, slightly clayey sandy silt, with some gravel in the east.

Cobbles and Boulders - commonly comprising 25 to 50 per cent in the west, 50 to 75 per cent and up to 100 per cent in the east, angular to subrounded, mainly slightly to moderately decomposed with patinas generally a few millimetres and up to 10 mm.

SOIL SAMPLING TECHNIQUES

The geology at each borehole location was obtained using conventional techniques of water flush HMLC triple tube or a combination of Mazier triple tube and Craelius double tube rock coring barrels. To obtain quality core samples of soft ground and decomposed rock it is the common practice in Hong Kong to use triple tube core barrels with retractor shoes. A serious problem with core drilling in soft ground is contamination and scour of the core by the drilling fluid, even with the protection of a retractor shoe.

In this study it was essential to obtain high quality samples for shear strength testing in the laboratory. Drilling trials were carried out using various combinations of core barrel types and sizes and flushing media. These trials are described by Phillipson and Chipp (1981). As a result of these trials, air or foam flush using large diameter (4C or 3C-MLC) triple tube core barrels was adopted for the high quality coring to a depth of about 30 m. Deeper holes were drilled by water flush with H or N size triple tube (MLC) core barrels or Mazier triple tube barrels and T2-101 mm double tube core barrels.

A detailed drilling specification based on the drilling trials together with full time supervision of the drilling operations by geologists or engineers ensured good quality drilling and sampling.

In addition, a large number of block and tube samples were obtained from 2 m diameter hand dug inspection shafts and trial pits. The block samples were 300 mm cube, from which 76 mm diameter triaxial test specimens were obtained by hand trimming in the laboratory.

HYDROGEOLOGY

The history of landslides in Hong Kong shows that critical conditions are produced by heavy intense rainfall particularly if preceded by a short wet period which has almost saturated the ground. Failures occur during or within hours following critical storms and little time is available to warn local inhabitants or to observe the mode of failure. For shallow slips on steep slopes simple sensitivity calculations can demonstrate that a rise in pore water pressure of a metre has as much effect on stability as a five degree drop in effective angle of internal friction.

About 350 piezometers had already been installed within the study area for previous routine site investigations of local stability. It had been the common practice to install such piezometers in the bottom of routine site investigation holes on the premise of a common ground water response in the soil and rock. For the study, an additional 124 piezometers were installed. The philosophy adopted was that in a multi-layered system, the most useful information would be obtained from a detailed knowledge of the geology and pore water pressure distribution on a few vertical profiles, rather than at many spot locations. Piezometers were of the standard Casagrande type, one or sometimes two being installed at carefully selected points in each drillhole. Four drillholes were needed at each investigation point to obtain the necessary good quality information on geology, pore water pressure and soil samples. Most piezometers were dipped manually but where possible those which responded rapidly to rainfall were read automatically.

The piezometer observations showed that over much of the area perched water tables develop in the colluvium and that in some areas these rise appreciably during storms. The depths to the perched water tables vary quite rapidly on plan and reflect the complex interaction of the topography and the geology.

Three methods were used to analyse the observed piezometric levels and to predict conditions that would develop during rare rainfall events:

a) a statistical correlation of ground water response with rainfall,
b) ground water modelling of vertical two dimensional section, and
c) extrapolation of observed piezometer responses.

Only the ground water modelling considered the lag in pore water pressure response to storm and seasonal rainfall.

The study adopted a return period of one in ten years as a design standard for rare events because piezometric data was available over only one or two years, and extrapolation to a longer return period was not considered to be reasonable. The return period of rainstorms and induced piezometric levels were assumed to correspond.

Different patterns of piezometer response could be recognised, varying from sharp changes in level within a matter of minutes of the rainfall to seasonal

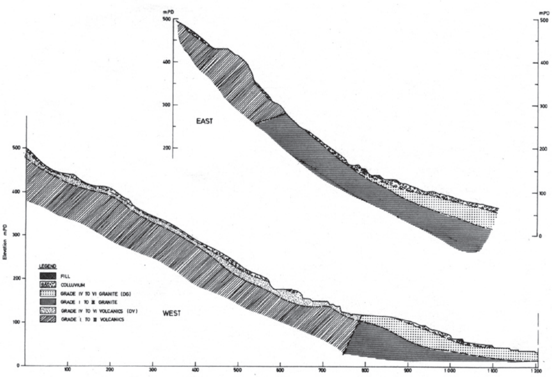

Figure 2. Typical geological cross sections

fluctuations where peak levels were reached four to six months after the wettest months. Each piezometer has a critical rainfall pattern which would give the highest response.

(i) Statistical correlation

Forty eight piezometers, with readings over four to 15 months, were selected for statistical correlation analysis. All response types were represented except very fast storm responses. Sixteen of the piezometers were automatically monitored from July 1980, and daily or more frequent data were available.

Depth to piezometric level was regressed by least squares onto a variety of measures of rainfall. These included cumulative rainfall over various durations, lagged cumulative rainfall for various lags and durations, antecedent precipitation indices (APIs) and lagged APIs. All of the relationships tested imply a linear relation between total rainfall and piezometric rise. Usually a one variable equation was used; in a quarter of the cases where a reasonable relationship could be found, a two variable equation was used. In almost all cases an API variable was found to explain most variance. Thirty-five of the final regression equations have an explained variance of 0.7 or better (correlation coefficient > 0.84) and these were used to estimate annual maximum levels for 1949-80 by running daily rainfalls through the models. Probability analyses were made of the resulting 32 year records and in all cases normal distributions were fitted by calculation. The fitted distributions were then available to estimate events of various return periods.

Only 12 of the statistical model cases were sufficiently accurate to meet four acceptance criteria that were set and this was not sufficient to develop estimates of piezometric levels throughout the area.

(ii) Ground water models

One section through the Glenealy/Seymour area (Figure 2, East) has been modelled using a finite difference model programmed for an ICL 2900 computer (Leach and Herbert - 1982). The model was first calibrated against the observed piezometer responses during 1980. Subsequently, the ground water response was calculated for several extreme rainfall years (1966, 1972, 1973 and 1976) and compared with the one in ten year predictions.

The calibration primarily involved the assessment of the appropriate aquifer properties and aquifer recharge. To obtain aquifer recharge from the upper slopes the rainfall was modified as follows:

Recharge = Rainfall - Runoff
- Change in soil moisture deficit
(including actual evapotranspiration)

If the difference between rainfall and run-off was insufficient to satisfy the soil moisture deficit then no recharge occurred.

During the initial calibration, the recharge to the lower slopes required from bedrock was considered to be unreasonably large compared with the annual rainfall for the catchment, and the aquifer properties were modified so that less recharge was required to reproduce observed ground water conditions. Subsequently it was discovered that leakage from services could be providing significant recharge to the developed area (see section *(iv)*, last paragraph).

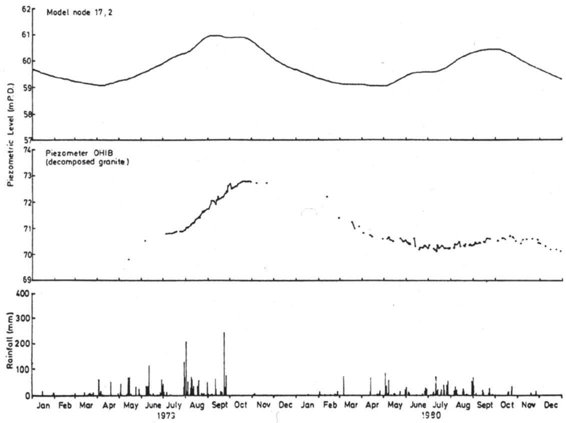

Figure 3. Comparison of groundwater model with Piezometer OH1B

A section in the Po Shan/University area (Figure 2, West) was also modelled. The model was similar in type to that used in the Glenealy/Seymour area, but was programmed for an in-house desk-top computer using a backward difference method of solution of successive over-relaxation. The model was first calibrated against observed piezometer levels and responses measured in 1979 and then used to predict responses for 1980. Results for one piezometer are shown in Figure 3. The patterns of response were well matched, but the model predicted too great a seasonal response in 1980.

The model studies confirmed that:

a) Ground water response is greatest near the foot of the steep vegetated undeveloped slopes.
b) Ground water response to storm in the covered developed lower slopes is relatively small.
c) Leakage from services within the developed area has a significant effect.
d) The predicted one in ten year levels are not substantially different from the highest water levels during the extreme rainfall years considered.

(iii) Extrapolation of observed responses

Neither the statistical method nor ground water modelling was sufficient on its own or in combination to predict the ground water pattern accurately enough over the whole area. The statistical method gave too few results and the model could not allow for local variability in ground properties and pore water pressure responses. However, each method gave an insight into the behaviour of the ground water regime with storms of different duration and intensity. Therefore, data was extrapolated from individual piezometers to obtain estimates of one in ten year levels. The results showed patterns that fitted the understanding of the hydrogeology and thus permitted interpolation in areas where there were no piezometers.

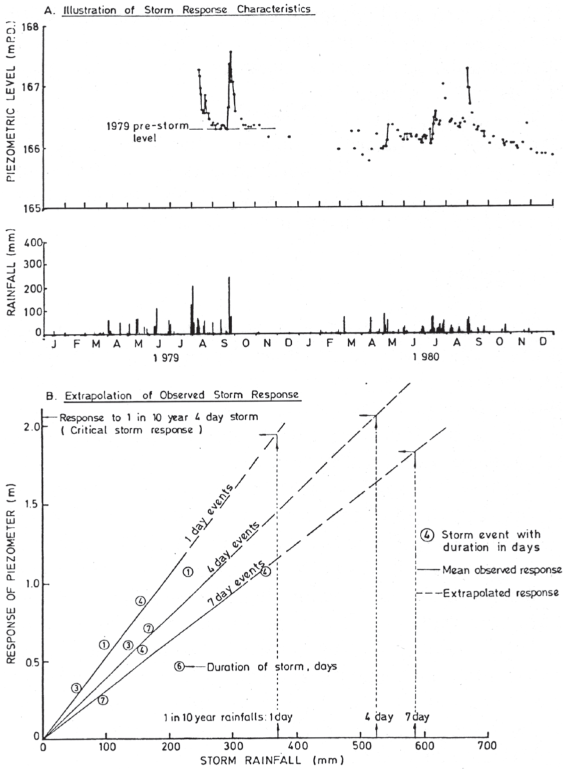

Figure 4. Extrapolation of storm response

The 1979 rainfalls for two weeks and longer were very close to one in ten year events. Hence, for the main water table only minor adjustments to the recorded levels were necessary to give the one in ten year levels for seasonally responding piezometers.

It was observed that different piezometers are sensitive to different lengths of storm event. This is a function of the delay before rises start, the rate of rise and the rate of decay. Piezometers in the steep vegetated undeveloped colluvium slopes reach a peak and start to decline within hours of heavy rainfall, whereas those in the lower slopes do not respond for days or even weeks. The responses to successive storms are additive in the lower slopes but each separate event governs behaviour in the upper slopes. A critical duration was estimated for each piezometer by inspecting the record of responses or, with short records, by comparison with nearby piezometers,

The method for predicting storm response is illustrated for a piezometer in decomposed volcanic rock in Figure 4 where the responses for all storms are plotted against storm rainfall. The duration of each storm is marked and approximate relationships shown for each duration. Responses to one in ten year storms of various durations can be estimated, and the largest is taken to represent the critical storm duration for that piezometer. In the case illustrated, the four day storm was most critical.

Critical durations of storms varied from greater than 14 days for piezometers in thick decomposed granite in the lower slopes to a few days in the same material at higher positions on the slopes. The critical storm duration for decomposed volcanic rock was normally two to four days. Responses were generally two to 3 m but were less where local drainage measures had already been installed. These storm responses were added to the base seasonal levels to obtain the one in ten year piezometric levels.

Perched water tables were treated similarly. Perching is thought to occur at the interface between colluvium and in situ material, whether this is decomposed volcanics or granites. Higher general perching layers were not detected although small positive pore water pressures were recorded by suction instrumentation. There is some suggestion of perching layers in decomposed volcanics, but this is inconclusive. Thus, in the derivation of one in ten year levels, the surface of in situ material has been taken as the base of all perched water tables. However, perched water tables are difficult to measure because piezometers have to be placed just above the level of perching and responses are rapid, in some cases as short as a few hours, so that automatic reading is essential to define the response curve accurately. Readings at one hour intervals were sometimes not sufficiently frequent to detect the start of the rise.

For the upper steep slopes where piezometric data was very limited a different approach was used, but also based on one to ten year rainfall events. From stream gauging the run off was taken as 34 per cent of rainfall. Two cases were taken:

a) Rainfall recharging the colluvium by vertical infiltration and assuming no flow in the downslope direction. This models rainfall on horizontal ground.
b) Assuming that a given rainfall intensity lasts for sufficient time to set up steady flow conditions at a point so that recharge into the colluvium is equal to the outflow downslope of that point. The head that can develop is proportional to the intensity of rainfall and the length of catchment upslope.

The head cannot exceed that for case (a) for a given storm event so that the relationship between maximum one in ten year perched head and catchment length upslope can be derived. As an example, for a 30° slope with a mean permeability of 10^{-4} m/s, storage coefficient of 0.05 and run-off coefficient of 0.34 the relationship as shown in Figure 5 is obtained. These results agreed reasonably well with those from field measurements and ground water modelling.

(iv) Discussion of results

The three methods of analysis described above have provided different types of result. The statistical correlation does not assume that the return period of rainfall events and piezometric levels correspond. The analysis is based on levels, rather than changes in level. Only 12 statistical estimates of one in ten year levels were acceptable, but the method has been shown to be satisfactory. The 12 values were insufficient to interpolate throughout the area but were used for comparison with the other methods.

The modelling studies have provided qualitative results. Much of the understanding of flows within the slopes resulted from attempting to model them. The models helped in assessing aquifer properties and estimating recharge. It was never expected that the models would provide point or areal estimates of one in ten year water levels but they have assisted in understanding responses to different rainfalls. The main conclusion drawn is that ground water responses do not vary in character as rainfall increases.

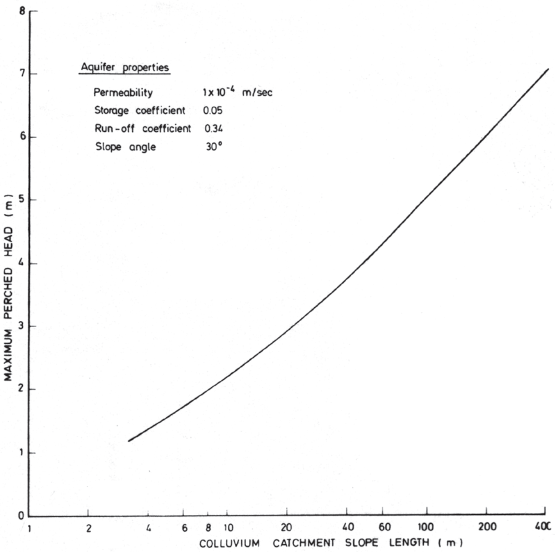

Figure 5. Maximum perched head over base of colluvium (1 in 10 year)

The method of extrapolating observed responses is the least sophisticated but has several advantages. It can be used for almost all piezometers. Any background knowledge of each piezometer can be incorporated. The derived piezometric levels appeared to be sensible and compared well between areas. This regionalisation or pooling of results effectively makes more data available at each location.

Buried beneath the urban area are fresh water mains, salt water mains (for flushing purposes), foul sewers and surface water drains. As fresh water is scarce in Hong Kong the general level of leak detection and repair of the distribution system is good, but from time to time local rapid deterioration or bursts can cause a rise in pore water pressure. Sudden piezometric and seepage changes have been detected, caused either by new leaks or repair of old leaks. During heavy storms surface water drains may become surcharged, due to blockages or intensive flow, which could cause leakage. In the lower slopes beneath the developed area the high level of the main water table during the dry season and early wet season is probably due to the topping up effect of leakage.

SOIL PORE WATER SUCTIONS

Some of the steeper natural and man-made slopes in Hong Kong have marginal stability when analysed using results from conventional effective strength triaxial tests and measured or inferred pore water pressures. Steep cut slopes of ten on six were normal practice until recently and failures of some of these slopes have occurred from time to time during the wet season. The cut surfaces were covered by chunam mainly to prevent surface erosion, clogging of surface drains and possibly large washouts, but also because it was felt that by preventing direct rainfall infiltration

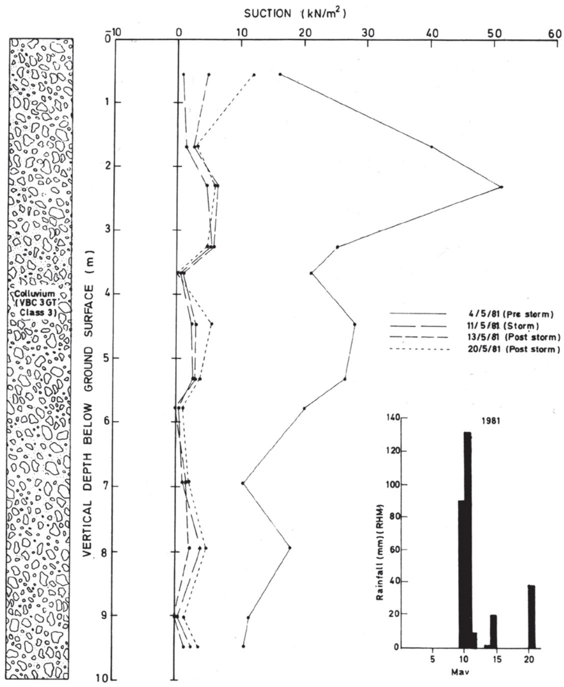

Figure 6. Suction response to storm 10/5/81 to 12/5/81 Chater Ridge - Caisson K3 - Instrument Group E/2/227

any stabilising resistance caused by soil suctions could be retained. A modest shear strength resistance attributed to soil suction is sufficient to give calculated factors of safety for shallow slip surfaces on steep slopes of unity or over.

One of the topics in the study was to investigate the level, persistence and rate of change of pore water suction in order to demonstrate:

a) the possibility of a sustained stabilising effect,
b) to establish whether minimum suction occurs at the same time over significant areas,
c) the water infiltration pattern both vertically and laterally, and
d) change in ground water pressures due to recharge by infiltration.

Approximately 300 tensiometers were installed. The tensiometers were grouped into arrays and clusters at five sites in the upper slopes (Figure 1) and installed beneath vegetated and sealed surfaces in all three types of soil - colluvium, decomposed granite and decomposed volcanic rock. Automatic readout equipment was used at most of the installations to detect any rapidly changing conditions.

The tensiometers were installed both from the ground surface and from the sides of concrete lined pits (caissons). The instrumentation design and installation was controlled by both economical factors and the need to collect sufficient data to have confidence in the consistency or otherwise of the observations.

The hydraulic characteristics of the tensiometers limits the maximum suction level that can be monitored. The onset of cavitation of water in the connecting tubes and water column prevents the measurement of negative water pressure greater than approximately 80 kPa. Therefore, with readout at the surface suctions up to about 75 kPa could be generally monitored in tensiometers installed one to 2 m below the surface and up to about 40 kPa for 4 m deep tensiometers. All tensiometers were de-aired regularly. Also, positive pore water pressures could be monitored.

Five caissons up to 10 m deep were excavated in the vegetated steep upper slopes. Tensiometers were installed 3 m from the sides of these caissons in inclined drillholes and were connected to a monitoring system within the caisson, which made it possible to measure suctions at depth. Most of the tensiometers were made from high air entry value ceramics connected to twin tube polythene coated nylon leads. Automatic monitoring of a group of up to 22 tensiometers was carried out by means of a fluid wafer switch connected to a single pressure transducer. The fluid wafer switch was activated by a stepper motor and controlled by a timing circuit. Tensiometer readings, and zero and reference readings, were traced on to a chart recorder. The tensiometers are similar to those used by Anderson (1979) and the automatic monitoring system is similar to that described by Burt (1978). During storm periods each tensiometer group was scanned continuously in two hour cycles. Manually read tensiometers manufactured by Soilmoisture Corporation were also installed.

In addition to the five deep profiles of suction measurements in the caissons, the areal migration of infiltration vertically and in plan on ridges and valleys was monitored by arrays of instruments set at shallow depth, with leads taken to the caissons for monitoring. The configuration of the arrays was restricted by the need to keep the level of the tips and highest points of the leads close to or above the level of the measurement transducer.

The manually read tensiometers were installed as part of the array system or in isolated groups of four or five at points of particular interest. Frequent observation was difficult especially during typhoon and storm conditions, but generally the rate of change of pressure measured by the instrument was not that rapid and the pattern of response to storms suggests that minimum values were detected.

The instrumentation was installed before the 1980 wet season. During dry periods significant suctions of up to 75 kPa were measured at depths of up to 10 m.

A minor storm with a return period of less than two years occurred in early September 1980 after a relatively quiet period during which steady conditions had been established. The measured suctions beneath vegetated slopes dropped to zero during this event. Within a few days after the storm measured suctions were rising rapidly, particularly beneath slopes where evapotranspiration was active. The behaviour was generally similar in all types of soil, but some minor differences were detected. These may be due to local conditions at each site rather than to basic differences in behaviour of the material types.

In colluvium with a vegetated surface, zero or slightly positive pressures were recorded to the maximum instrumented depth of 10 m. The suction profiles before, during and after a storm at a site in a colluvium ridge are shown in Figure 6.

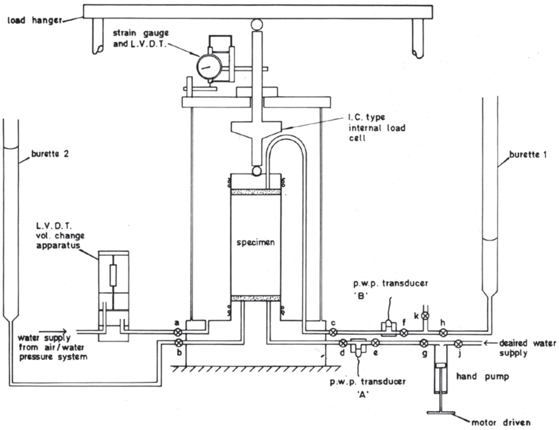

Figure 7. Dead load test apparatus

In a steep natural vegetated slope in decomposed granite, suctions dropped to zero to depths of 5 m but beneath this depth remained above 10 kPa during the storm. Measurements were also made at another site where there was a cut slope in decomposed granite, part of which had an open surface and part was sealed by chunam. The suction beneath the open surface dropped to less than 5 kPa whereas beneath the chunam minimum values of 10 kPa to 20 kPa were measured.

These responses were for short sharp storms which may not have caused the most critical conditions at each site so that the observations should not be taken as reflecting the worst conditions at the measurement sites and, of course, should not be used elsewhere without verification by local instrumentation.

LABORATORY SHEAR STRENGTH TESTS TO SIMULATE SITE CONDITIONS

As part of the Mid-levels study, laboratory tests were performed to assess the shearing resistance of the ground and particular attention was given to simulation of the in situ conditions. This necessitated the development and use of techniques not normally available in commercial testing laboratories.

(i) Dead load tests

The closest simulation of field conditions was achieved using the dead load test. This test allowed the following conditions to be imposed upon specimens:

a) Percolation to reach field saturation.
b) Failure caused by inflow of water raising the pore water pressure.
c) Stress controlled failure rather than strain controlled failure.
d) Constant total anisotropic stresses as in a natural slope.
e) Low effective stresses at failure.

The equipment used to perform these tests is shown schematically in Figure 7. It consists of a triaxial cell fitted with a hanger load system and an internal load cell. Water was introduced by a hand pump and the response of the specimen monitored by transducers. The burettes shown were used to percolate water through the specimen.

The test method consisted of stepwise consolidation to reach a state of anisotropic total stress estimated from consideration of:

a) stresses acting upon an infinite slope failure plane at the depth of the specimen, and
b) that a pore water pressure of 30 kN/m^2 was desirable at failure for accurate measurement purposes.

Following consolidation, water was percolated under a small hydraulic gradient to reach a degree of saturation comparable with the field values. Finally, failure was brought about by introducing water

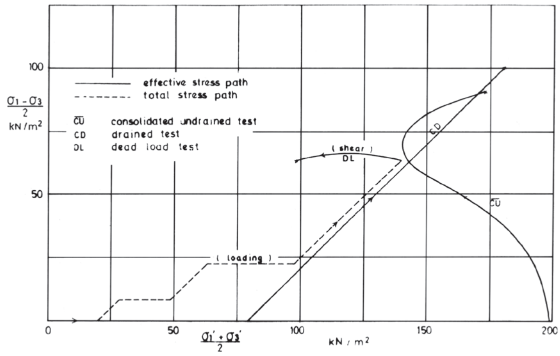

Figure 8. Typical colluvium saturated triaxial test stress paths

using the motor driven hand pump, thus causing the pore water pressure to rise. The readings from the transducers were recorded at intervals of 6s or longer as necessary. The results were analysed using a desk top computer and plotter.

(ii) Drained triaxial tests

Drained triaxial tests were performed generally in accordance with normal procedure. In order that they reflected field conditions they were performed at

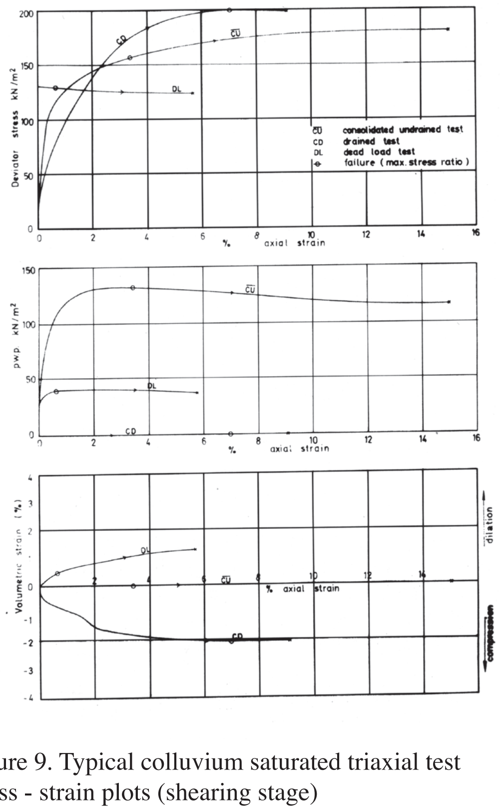

Figure 9. Typical colluvium saturated triaxial test stress - strain plots (shearing stage)

low cell pressures and, prior to shearing, water was percolated through the specimens, This was to simulate the wetting up the soil would experience as a result of a rainfall event.

(iii) Consolidated undrained triaxial tests with pore water pressure measurement

Effective stresses during the back pressure saturation and consolidation stages were maintained close to those in the field.

(iv) Constant water content tests

In order to evaluate the increase in shear strength due to suction, constant water content (unsaturated) triaxial tests were performed.

Samples taken from above the water table were placed on a deaired high entry ceramic and the initial suction measured. Following this, the triaxial cell was assembled and an all round pressure was applied of approximately the in situ overburden pressure. When specimen volume change measurements showed that consolidation was complete the specimens were sheared.

During this test the pore air pressure was vented to atmospheric pressure (via a dry porous stone and the top cap drainage connection) and the pore water pressure was measured through the bottom drainage connection.

By assuming that the suction, $(u_a - u_w)$, where u_a = pore air pressure and u_w = pore water pressure, was equivalent to an effective normal stress the results were compared with those from saturated triaxial tests to evaluate the contribution of the suction component.

(v) Suction conditioned tests

The results from constant water content tests were not considered conclusive and hence tests on suction conditioned samples were performed to extend the information on unsaturated shear strength. Two forms of conditioning were used: axis translation technique and negative pore water pressure technique.

(A) Axis translation technique

This method has been used by other workers carrying out research into unsaturated shear strength (U.S.B.R. (1966) and Fredlund (1981)) and enables specimens to be conditioned to a given suction $(u_a - u_w)$ by elevating the pore air pressure above the pore water pressure which is maintained at a pressure that can be measured with conventional equipment.

Axis translation tests are complex to perform since they require modifications to triaxial equipment. They are also difficult to relate to in situ conditions since the elevated pore air pressure does not exist in the field: consequently, a simpler form of suction conditioning was also investigated in which a small negative pore water pressure was established.

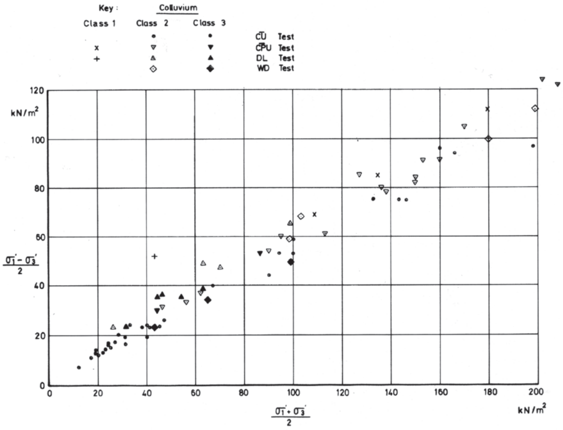

Figure 10. Summary of triaxial test failure stresses - colluvium

(B) Negative pore water pressure technique

This test was a modified version of the drained triaxial test. The technique was to percolate water through the specimen whilst maintaining small (up to -10 kN/m^2) negative pore water pressures. The negative pressure was created by setting the water levels in burettes below the level of the specimen and the hydraulic gradient produced by a difference in elevation of water levels in the burettes.

After percolation the specimens were sheared fully drained and the results compared with conventional saturated tests to determine the effect of the negative pore water pressure.

Conditions in this form of test model those corresponding to in situ suctions existing after rainfall.

(vi) Shear strength test results

(A) Saturated test results

All three of the saturated effective stress tests - the drained triaxial, undrained triaxial and dead load tests - produced consistent shear strength results for colluvium and decomposed volcanic rock samples. Results from decomposed granite rock samples showed less agreement due to material variability. Overall the drained tests showed most scatter, whilst the consolidated undrained tests were more consistent. They both produced generally dilative behaviour. The dead load tests generally formed the upper bound to the shear strength envelope.

In the dead load tests virtually no axial strain occurred until failure was imminent, and at failure the axial strain was typically 1.5 per cent. Prior to failure swelling took place with radial displacement only.

Figures 8 and 9 illustrate the stress paths and stress strain relationships respectively for a typical set of saturated triaxial tests performed on three samples of volcanic colluvium from the same location.

The effective stress test results are plotted on

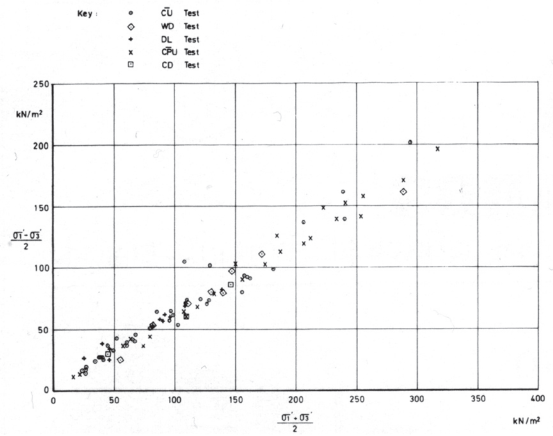

Figure 11. Summary of triaxial test failure stresses - decomposed volcanics

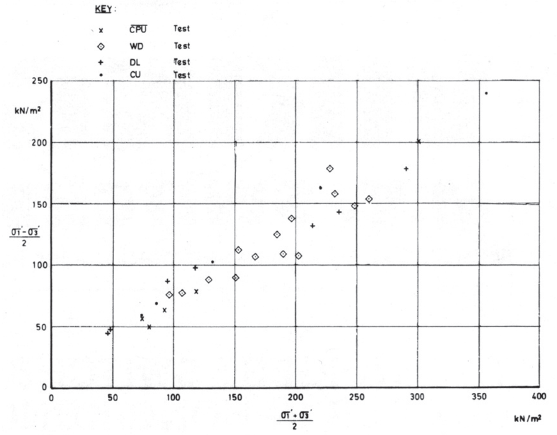

Figure 12. Summary of triaxial test failure stresses - decomposed granite

Figures 10, 11 and 12 for colluvium, decomposed volcanics and decomposed granites, respectively. The heterogeneity of the materials is reflected in the scatter of points, but the type of test may also have a significant effect. The least square fit equations are:

colluvium : τ (kN/m^2) $= 4.4 + \sigma_n^{1} \tan 34.3°$
volcanic : $\tau = 7.4 + \sigma_n^{1} \tan 35.7°$
granite : $\tau = 5.3 + \sigma_n^{1} \tan 41.3°$

(B) Unsaturated test results

The unsaturated tests that were performed were conducted using small suctions and low stresses as anticipated in situ. The results from the constant water content tests and the axis translation type tests indicated an increase in shear strength due to suction, but attempts to establish a unique relationship were restricted because of the sensitivity of the analysis to the saturated shear strength parameters used and the magnitude of the suction involved.

The tests on samples conditioned by the negative pore water pressure technique suggested that small suctions that exist after wetting (i.e. at high degrees of saturation) produce an increase in shear strength, equivalent to an increase in effective normal stress.

THE ASSESSMENT OF STABILITY

The stability of the slopes has been assessed using Janbu's rigorous method of analysis with variably inclined interslice forces.

The measured piezometric data has been used to estimate the pore water pressure distribution in the slopes for the one in ten year storm and these pore water pressures have been used in the stability calculations.

Past failures have also been analysed and while there is general agreement between the assumed shear strength parameters and those implied by the failure, the presence of boulders have been shown to add significantly to the in situ strength of the colluvial soils.

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