SHEAR STRENGTH PROPERTIES OF SOME MALAYSIAN RESIDUAL SOILS

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ABSTRACT

High cuts in granitic residual soil deposits for construction of new highways are common in Peninsula Malaysia where about a third of the land area is underlain by granitic and other non-volcanic igneous rock formations. Frequent failures of these slopes during raining seasons had become an increasing problem. A good understanding of the behavior of these granitic soils is important to the geotechnical engineers in carrying out analyses and design of the slopes. A detailed geotechnical study was carried out in 47 failed cut slopes along the existing Kuala Lumpur-Karak Highway. Four different types of shear strength tests were performed on undisturbed samples taken from these slopes. They include conventional single-stage and multi-stage $\overline{CIU}$ tests, direct shear tests, and a non-conventional triaxial test in which an anisotropically consolidated specimen was brought to failure by increasing the pore water pressure. Shear strength parameters obtained from these four different types of tests were compared and evaluated. Their applicability in the slope design was studied through back analyses of failed slopes.

It was found that, the pore water pressure controlled triaxial tests gave higher value of the angles of shearing resistance than those obtained from both the multi-stage and single-stage $\overline{CIU}$ tests. The multi-stage $\overline{CIU}$ tests gave higher effective cohesion intercept than those obtained from the other tests. Back analyses of the failed slope using the strength parameters obtained from the different shear tests showed that the safety factors estimated for the observed sliding surfaces were less than unity in most cases. The strength parameters from pore water pressure controlled tests appeared to give a safety factor nearest to unity.

INTRODUCTION

Granite and other non-volcanic igneous rocks take up approximately one-third of the total land area of about 130,000 square kilometers of West Malaysia, commonly referred as Peninsula Malaysia (see Fig. 1). These form most of the main mountain ranges and extend into lowlands to a limited extent. Several highways through these mountainous areas required very high soil and rock cuts, in some places exceeding 100m. The Kuala Lumpur - Karak Toll Highway, a part of the overall Federal Route II, the major east-west link in the central part of Peninsula Malaysia, is one such highway (Fig. 1). One stretch of this highway, approximately 50 km long, crosses mountainous areas and involves a large number of high cut slopes in residual granitic soils. About 7 years after the opening of the highway, many of the high cut slopes began to have stability and erosional problems which became progressively serious with time.

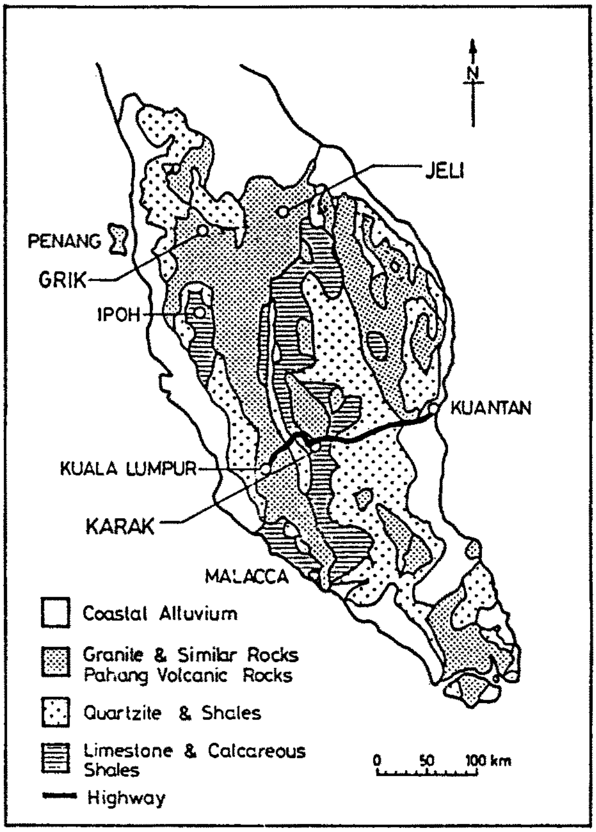

FIG. I GEOLOGY OF WEST MALAYSIA

A detailed geotechnical investigation was carried out in 66 failed cut slopes and embankment slopes along the highway for the design of slope remedial works. Of the 66 problem areas investigated, 47 cut slopes were in residual granitic soils. In these granite cut slopes, 93 boreholes were drilled, about 800 undisturbed and disturbed samples were collected for laboratory testing, 121 piezometers were installed for groundwater monitoring and 40 constant head field permeability tests were carried out in boreholes. Laboratory testing on this granitic soils included 122 sets of triaxial shear strength tests, 16 sets of direct shear tests and several hundred sets of physical properties tests.

Four different types of shear strength tests were carried out on the soil samples. The results obtained from these different types of tests are compared. Their applicability for the assessment of slope stability is evaluated based on back analyses of the failed slopes.

LABORATORY TESTING FOR SHEAR STRENGTH

Shear strength parameters of a soil is generally obtained in the laboratory by shearing specimens either in a triaxial cell or in a direct shear box. Triaxial test permits control of the principal stresses applied to the specimen, the drainage conditions, and measurements or control of the pore water pressure. Direct shear test is much simpler as compared to triaxial tests, but it permits only partial control over the drainage conditions.

Majority of the shear strength tests carried out in this study were multi-stage $\overline{\text{CIU}}$ ($\overline{\text{CIU}}$-M) tests in which a single specimen was isotropically consolidated and sheared under undrained conditions for three stages. Several series of single-stage $\overline{\text{CIU}}$ ($\overline{\text{CIU}}$-S) tests were carried out for comparison of results. For the $\overline{\text{CIU}}$-S tests, three specimens were consolidated isotropically to different consolidation pressures and then sheared under undrained conditions. In both series of tests, the soil samples were sheared by increasing the axial load at a constant strain rate and maintaining the cell pressure constant. The pore water pressures in the specimens were measured during shear.

In order to simulate the field conditions more closely, several series of single-stage $\overline{\text{CAU}}$ tests with changes in pore water pressure ($\overline{\text{CAU}}$-P) were carried out. In these tests, the specimens were anisotropically consolidated under different field stress conditions and then sheared by increasing the pore water pressure in the specimen.

TESTING PROCEDURES

Specimens used in all the triaxial test series were 70mm diameter and 140mm high. Specimens were saturated with a back pressure of 200 kN/m^2 until the pore water pressure response in the specimen was at least 97% of changes in the cell pressure.

In the multi-stage $\overline{\text{CIU}}$ tests, the specimen was isotropically consolidated under the first stage consolidation pressure. The specimen was then loaded under undrained condition by increasing the axial load on the specimen at a constant rate of strain of 0.07 per cent per minute until the effective principal stress ratio reached a maximum. At this point (i.e. the first failure point) the axial load was slowly withdrawn and the next stage of consolidation was carried out which was followed by the second stage of loading. Each specimen was tested for three stages which gave three points on the failure envelope.

In the single-stage $\overline{\text{CIU}}$ testing, three individual specimens were used. Each specimen was consolidated to one of three consolidation pressures and then sheared under undrained condition by increasing the axial load at the same constant strain rate of 0.07 per cent per minute. In $\overline{\text{CAU}}$ pore water pressure controlled testing, three specimens were used. The saturated specimens were first consolidated to different consolidation pressures. Anisotropic consolidation pressures were then applied by increasing the axial load and maintaining the cell pressure constant. A dead weight was used to apply the axial load. Stage increments of axial load were needed to reach the final anisotropic stress condition for each specimen to avoid failure of the specimen during the consolidation process. After the consolidation process was completed, the specimens were ready for shearing. In these tests, the specimens were brought to failure by increasing the pore water pressure while maintaining the axial load and the cell pressure constant. Pore water pressure in the specimen was increased in stages, with large increments at the beginning and smaller increments when the specimen was close to failure. Each increment of the pore water pressure was applied at the top of the specimen and allowed to equalize through the specimen before the next increment was applied. At failure, the specimen deformed continuously without any change in the applied pore water pressure.

Direct shear tests were carried out on 60mm diameter and 20mm thick specimens. The specimen was set up in the shear box with porous stones at top and bottom and then filled with water and allowed for saturation. Normal load was applied on the specimen using a dead weight. When the vertical displacement stopped, the specimen was sheared at a constant displacement rate of 0.238mm/min. Three to four specimens with different normal loads were sheared for each series.

GENERAL SOIL PROFILE AND PHYSICAL PROPERTIES

Typically, the subsurface profile in these granitic hills consists of residual granitic soil cover with embedded corestones underlain by the granite bedrock. Thickness of the soil cover at investigated slopes varied from 6m to 45m, the smaller values at slopes located in high altitudes and large values at slopes located in lower altitudes.

Field and laboratory test results show that the properties of the soil cover vary with the depth. Variation of the soil properties found at a typical slope is shown in Fig. 2. In general, the soil cover can be divided into three layers but in some places one or two layers are absent in the soil profile. The average thickness of the layer 1 (topsoil layer) is 12m and it was found at almost every slope investigated. The layer 2 (middle layer) which has an average thickness of 6.5m is located 3-20m below the ground surface and the layer 3 (bottom layer) overlying the granite bedrock is about 7.0m thick. Of the total thickness of the soil cover, 50-70% is the layer 1 and the rest is the other two layers in approximately equal proportions.

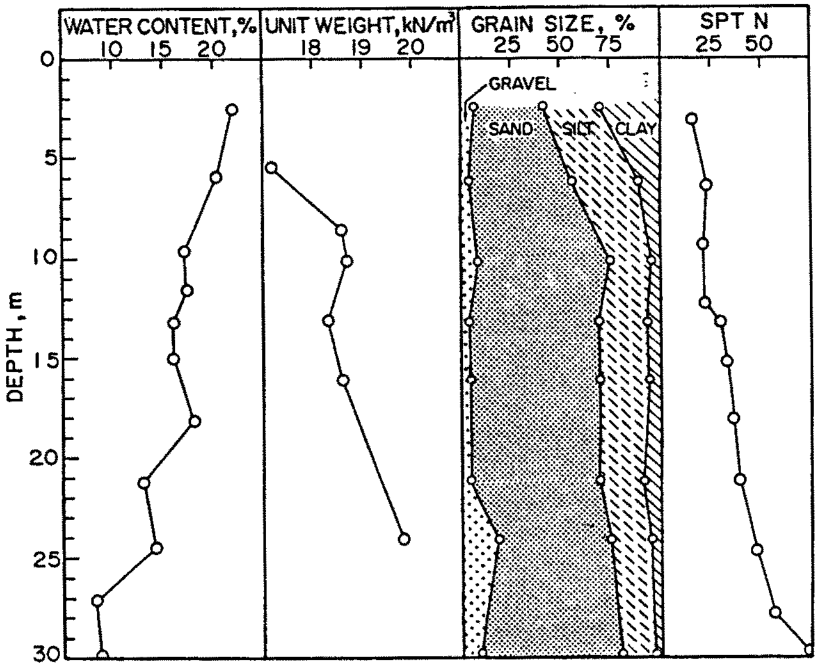

FIG. 2 TYPICAL VARIATION OF PHYSICAL PROPERTIES WITH DEPTH

Physical properties of the three soil layers are shown in Fig. 3 together with SPT N values. The amount of the fine fraction of the soil decreases with increasing in depth below the ground surface. The total unit weight of the soil increases and the natural moisture content decreases with depth. Variation in the SPT N value is very small within one layer, but between layers the difference of N values are very large.

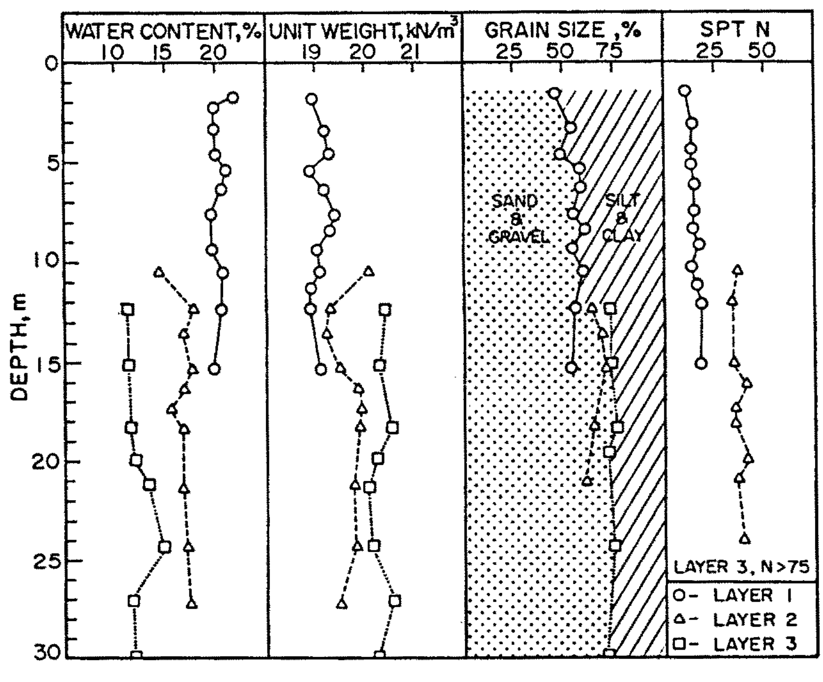

FIG. 3 PHYSICAL PROPERTIES OF THE SOIL LAYERS

Various quantities of corestones or boulders were found embedded in the soil overburden at many places. They were generally found near the bedrock surface, but in some places they were found very near the ground surface. These boulders are usually round in shape and their sizes vary from about 0.5m to more than 6m in diameter.

SHEAR STRENGTH PARAMETERS

Multi-Stage $\overline{CIU}$ tests were carried out in general to establish the shear strength parameters of the soil layers. The other tests, namely single-stage $\overline{CIU}$ tests, pore water pressure controlled $\overline{CAU}$ tests and direct shear tests were carried out only for the purpose of comparison of strength parameters obtained form different tests.

Values of the effective angle of shearing resistance (ϕ') and the effective cohesion intercept (c') obtained for the three soil layers at different slope locations are plotted against sample depth in Fig. 4. It is seen that the individual strength parameters of the three soil layers at different locations vary considerably. The effective angle of shearing resistance of layer 1 soil varies between 25° and 42°, but majority of the values fall between 31° and 36°. The effective angle of shearing resistance of the middle layer varies between 30° and 40° and the average is 35°, similar to that of the topmost layer. The effective cohesion intercept however, varies significantly between 0 and 50 kN/m². Only a few series of test results are available for the layer 3 soil. Results show that the effective angel of shearing resistance varies between 25° and 40°, but most of the values are below 35°, which is the average value of the other two layers. On the other hand, high effective cohesion intercept values were obtained from the tests for the soil layer, generally more than 30kN/m².

COMPARISON OF RESULTS FROM DIFFERENT SHEAR TESTS

Specimens for comparison tests were prepared from the same sample tube to minimize the effect of nonuniformity of soils on test results. Most of the samples taken were from the layer 1 where the soil is more uniform.

In single-stage $\overline{CIU}$ tests and pore water pressure controlled CAU tests, loading was continued for large strains since a separate specimen was used for each consolidation stage. In multi-stage $\overline{CIU}$ tests, the specimen was unloaded soon after passing the failure point for each stage except for the last stage where the specimen was loaded well beyond the failure point.

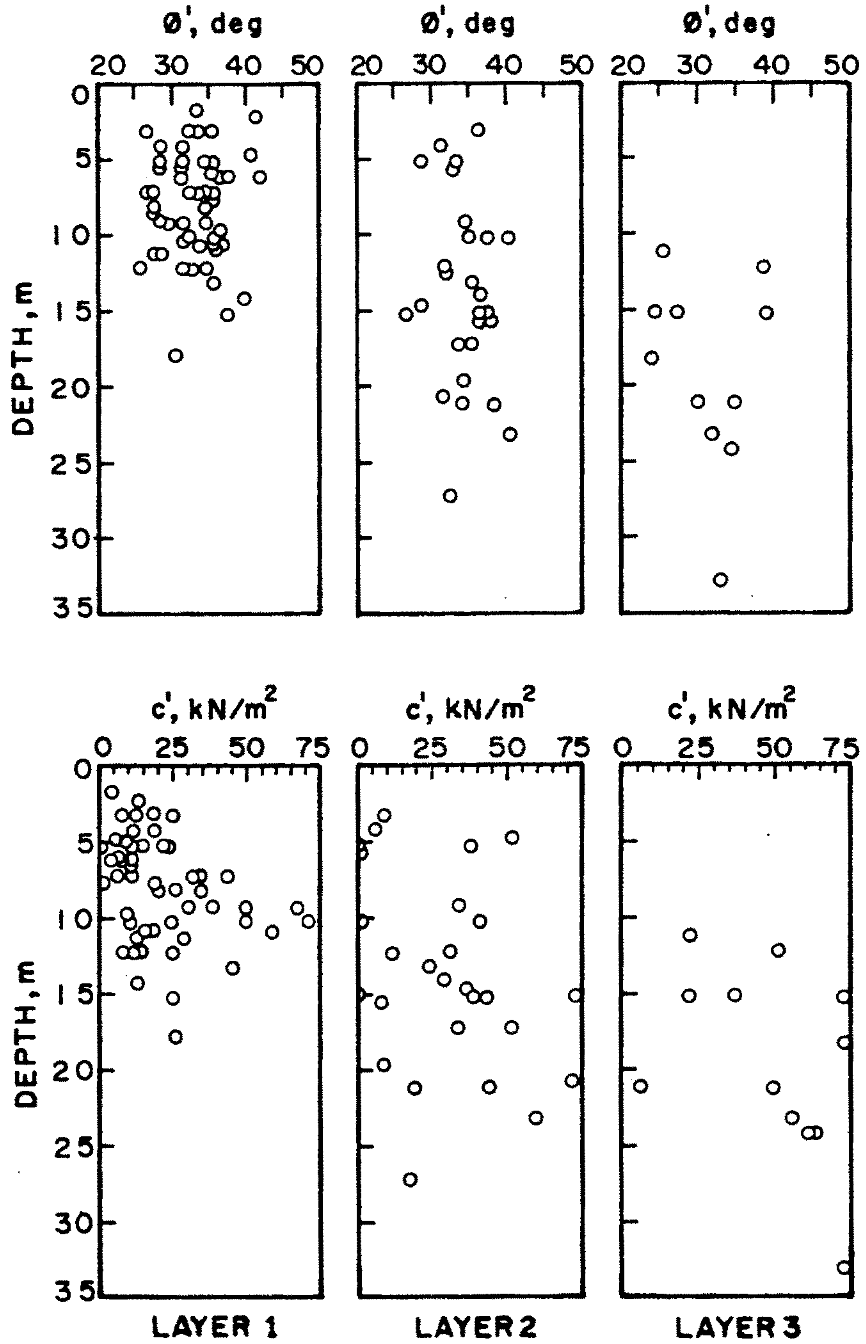

FIG. 4 EFFECTIVE STRENGTH PARAMETERS OF THE SOIL LAYERS

The results of the effective shear strength parameters obtained from six series of single-stage $\overline{CIU}$ tests are compared with the corresponding multi-stage $\overline{CIU}$ test results in Fig. 5. The results show that except for one pair of tests, the effective angle of shearing resistance estimated from single-stage $\overline{CIU}$ tests are always larger than those estimated from multi-stage $\overline{CIU}$ tests. On the other hand, all six series of the single-stage $\overline{CIU}$ tests gave smaller effective cohesion intercept values than those from corresponding multi-stage $\overline{CIU}$ tests.

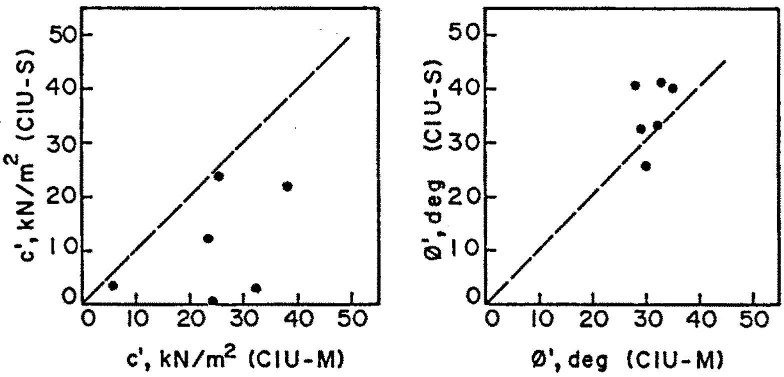

FIG. 5 COMPARISON OF STRENGTH PARAMETERS – SINGLE-STAGE $\overline{\text{CIU}}$ VS MULTI-STAGE $\overline{\text{CIU}}$

In Fig. 6, the results obtained from pore water pressure controlled $\overline{\text{CAU}}$ test are compared with that of the multi-stage $\overline{\text{CIU}}$ test. The effective angle of shearing resistance obtained from pore water pressure controlled $\overline{\text{CAU}}$ tests are larger than those from multi-stage $\overline{\text{CIU}}$ except for one pair of tests. The effective cohesion intercept from pore water pressure controlled $\overline{\text{CAU}}$: tests is larger than that from multi-stage $\overline{\text{CIU}}$ test in five pairs of tests, but smaller in 3 pairs.

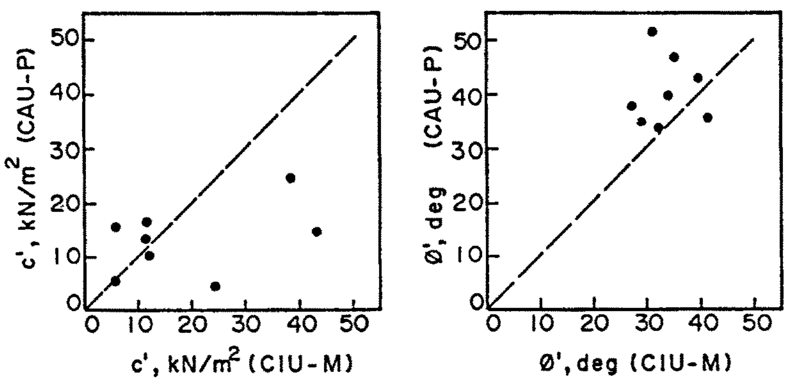

FIG. 6 COMPARISON OF STRENGTH PARAMETERS – PORE WATER PRESSURE CONTROLLED CAU VS MULTI - STAGE $\overline{\text{CIU}}$

The pore water pressure controlled $\overline{\text{CAU}}$ test results are compared with single-stage $\overline{\text{CIU}}$ test results in Fig. 7. Four pairs of test results are compared. In all four cases, the pore water pressure controlled $\overline{\text{CAU}}$ tests gave larger value of effective angle of shearing resistance and effective cohesion intercept than those obtained from the single-stage $\overline{\text{CIU}}$ tests.

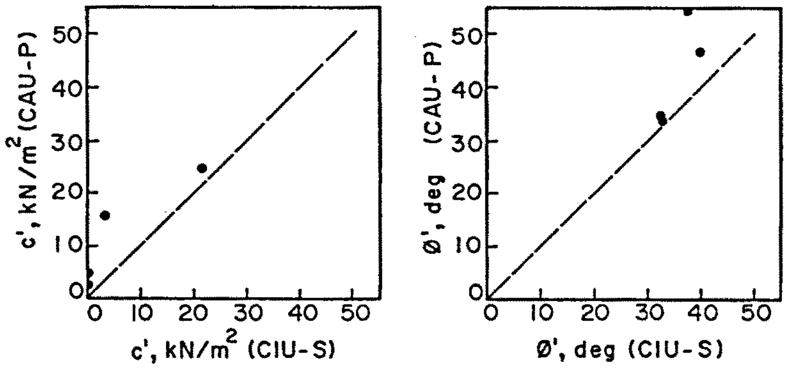

FIG. 7 COMPARISON OF STRENGTH PARAMETERS – PORE WATER PRESSURE CONTROLLED $\overline{\text{CAU}}$ VS SINGLE - STAGE $\overline{\text{CIU}}$

Four series of direct shear test results are compared with corresponding multi-stage $\overline{\text{CIU}}$ test results in Fig. 8. The direct shear tests gave higher values of angle of shearing resistance in 3 pairs of tests. Effective cohesion intercept estimated from direct shear tests is larger in two pairs and smaller in other two pairs than those estimated from multi-stage $\overline{\text{CIU}}$ tests.

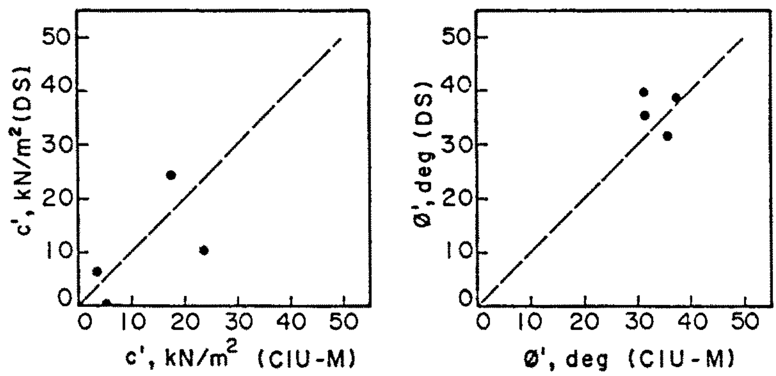

FIG. 8 COMPARISON OF STRENGTH PARAMETERS – DIRECT SHEAR VS MULTI - STAGE $\overline{\text{CIU}}$

In summary,

a) Single-stage $\overline{\text{CIU}}$ tests give smaller effective cohesion intercepts and larger angles of shearing resistance than those obtained from multi-stage $\overline{\text{CIU}}$ tests.

b) Pore water pressure controlled $\overline{\text{CAU}}$ tests give higher values of effective angles of shearing resistance than those obtained from both the single-stage $\overline{\text{CIU}}$ and multi-stage $\overline{\text{CIU}}$ tests. The values of effective cohesion intercept obtained from the pore water pressure controlled $\overline{\text{CAU}}$ tests are also higher than those obtained from single-stage $\overline{\text{CIU}}$ tests.

c) Direct shear tests give higher effective angle of shearing resistance than those estimated from multi-stage $\overline{\text{CIU}}$ tests.

USE OF STRENGTH PARAMETERS FOR STABILITY ANALYSES

Shear strength of the subsoil is one of the major variables that controls the stability of a slope. The other variables are slope geometry, unit weight of the subsoil and the groundwater condition. As described in the previous section, the values of the shear strength parameters of the residual granitic soils are dependent upon the testing condition, or more correctly, the applied stress condition. It is important to know which of these tests could more closely estimate the representative shear strength parameters of the soil. Several failed and unfailed slopes were analyzed using the strength parameters obtained from different types of tests and two typical results are presented below. These two slopes had minimum soil variation within the depth of possible sliding surfaces and groundwater was not found within this depth. Therefore, the effect of soil variation and groundwater on stability calculations could be treated as negligible. The slope geometries are based on survey data, and the subsoil profiles and properties are based on the field investigation data. Morgenstern and Price's method of slices for non-circular slip surfaces was used to estimate the safety factors.

Case 1

This cut slope is about half a kilometer long and 45m high at the maximum point. the slope had been cut to an average slope gradients of about 55°, with 1-2m wide benches at every 6m rise. A general slope failure had taken place at one section of the slope during the raining season before the investigation. The failed area was about 30m wide and extended from about the first bench upto the top of the cut or a little beyond. A total of five boreholes were drilled at three slope sections including the failed section and 7 piezometers were installed for groundwater monitoring.

The soil profile of the slope at the failed section is very uniform to about 25m below the ground surface except that there is a slight increase in the SPT N value below about 15m (Fig. 9). Four series of multi-stage $\overline{CIU}$ tests were carried out on

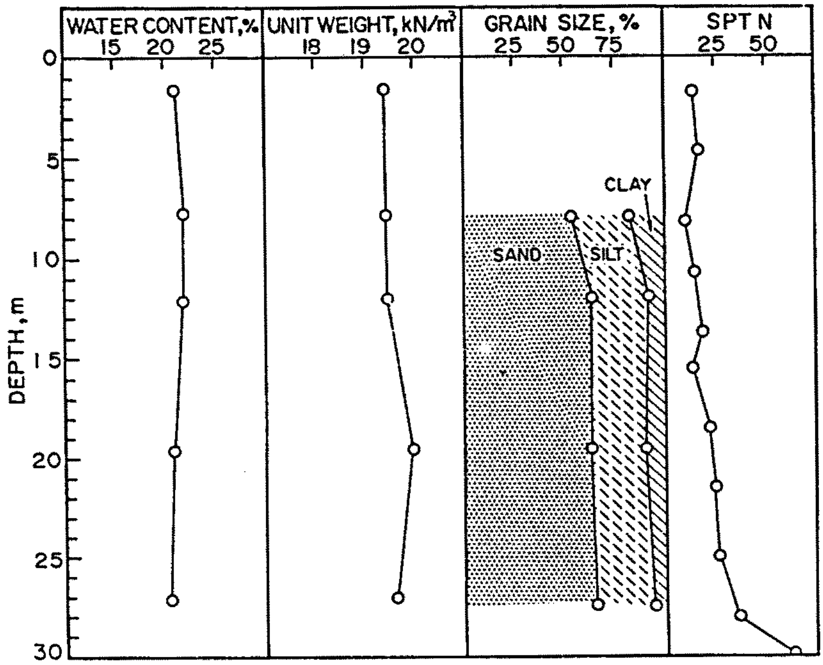

FIG. 9 SOIL PROFILE AT FAILED SLOPE - CASE I

samples taken from 7.8m, 12.2m, 19.7m and 27.2m depths. Stress paths obtained from these four test series and the strength parameters estimated from each series are shown in Fig. 10. The effective angle of shearing resistance estimated from different test series varied within a narrow range (32.9° - 35.7°) and the effective cohesion intercept varied from 2.1 - 8.4 kN/m² except for the test on the deepest soil sample which gave 17.1 kN/m². Figure 10 shows the combined plot of stage 1 stress paths from all four test series. The failure points in each of these four stress paths fall on a straight line very closely and the effective strength parameters estimated from this failure envelope are 35.6° and 2.6 kN/m². These values are equivalent to that obtained from a single-stage $\overline{CIU}$ test series.

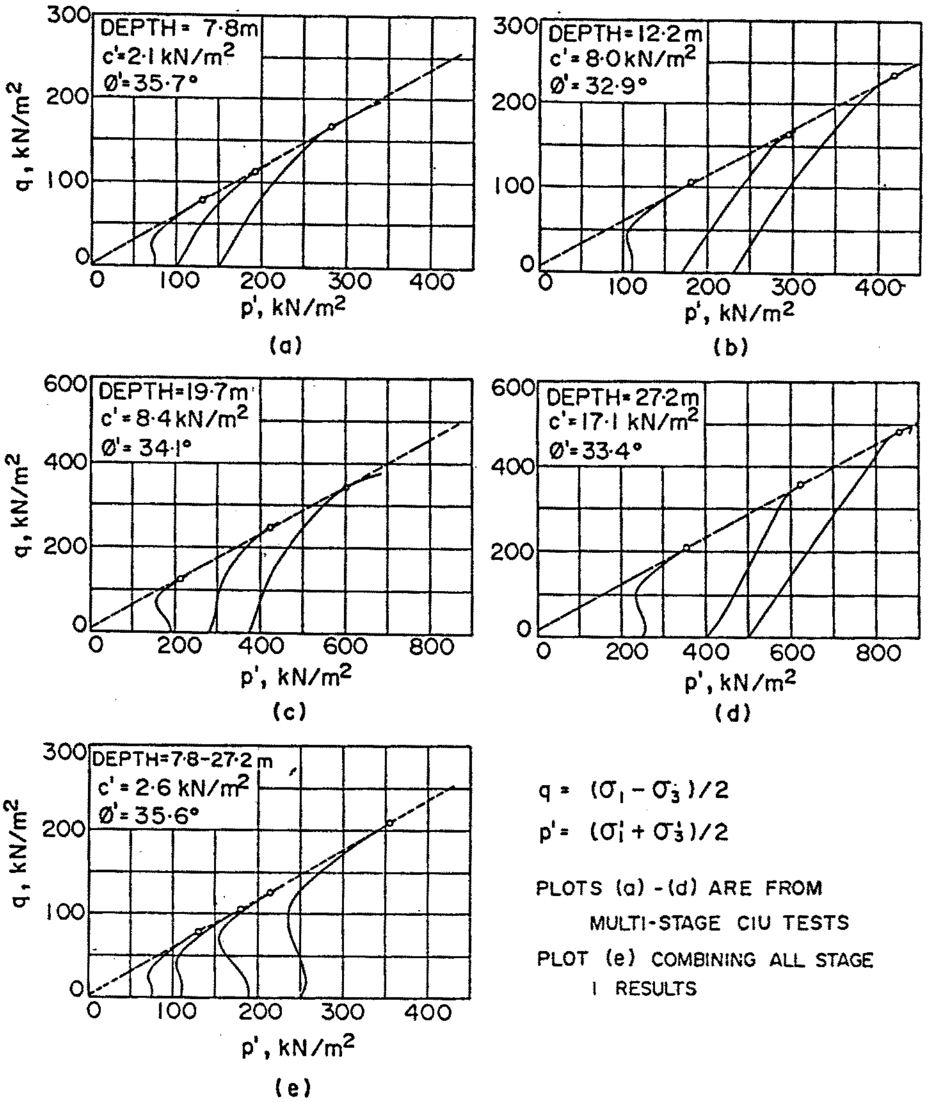

FIG. 10 STRESS PATHS FROM TRIAXIAL TESTS - CASE I

Stability of the original cut slope at the failed section was analyzed using strength parameters estimated from multi-stage $\overline{\text{CIU}}$ test results (average values from all four series) as well as from the equivalent single-stage $\overline{\text{CIU}}$ test results. All seven piezometers were dry during the monitoring period and therefore groundwater was not taken into account in the stability analyses. Groundwater seepage was not observed during slope inspections. Several sliding surfaces including the actual observed failure surface were considered. Four representative surfaces are shown in Fig. 11. Results of the stability analyses are summarized in Table 1. It can be seen that the estimated factor of safety values (0.68 corresponding to single-stage $\overline{\text{CIU}}$ data and 0.79 corresponding to multi-stage $\overline{\text{CIU}}$ data) are very low. Furthermore, the position of the critical sliding surface has also shifted from the actual failure surface when strength parameters from the single-stage $\overline{\text{CIU}}$ test were used.

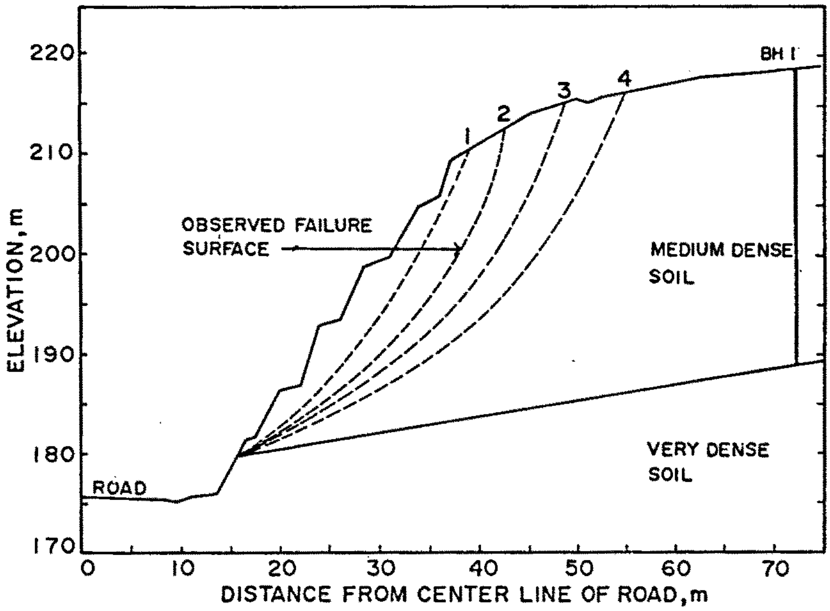

FIG. 11 SLOPE GEOMETRY AND SLIDING SURFACES – CASE 1

Case 2

This cut slope is completely in soil, about 220m long and 30m high. Nearly the whole slope had failed at the time of investigation. Three boreholes were drilled along two slope sections and four piezometers were installed for groundwater monitoring. Neither the bedrock nor boulders were found in any of the boreholes (maximum depth is 27.5m). The soil cover is uniform compared to the other slopes investigated (Fig. 12). None of the piezometers recorded any groundwater. It is possible that the percolating water seeps through this uniform layer and accumulates on a dense layer or the bedrock at a deeper depth. Although this slope failed during the raining season in 1985, the failure may not be triggered by rising water table, but due to other effects such as increase in soil weight, reduction in suction pressure or soil shear strength.

TABLE I STABILITY ANALYSES RESULTS - CASE 1

SHEAR STRENGTH PARAMETERS			FACTOR OF SAFETY			
TYPE OF TEST	c' kN/m^2	ϕ' deg.	SURFACE			
			1	2	3	4
CIU-M	9.0	34.0	0.82	0.79	0.89	0.94
CIU-S	2.6	35.6	0.68	0.73	0.82	0.93

TABLE II STABILITY ANALYSES RESULTS - CASE 2

SHEAR STRENGTH PARAMETERS			FACTOR OF SAFETY			
TYPE OF TEST	c' kN/m^2	ϕ' deg.	SURFACE			
			1	2	3	4
CIU-M	11.5	31.6	-	0.85	0.83	0.89
CIU-S	3.2	33.0	0.70	0.70	0.73	0.83
CAU-D	15.3	33.8	-	1.00	0.90	1.01

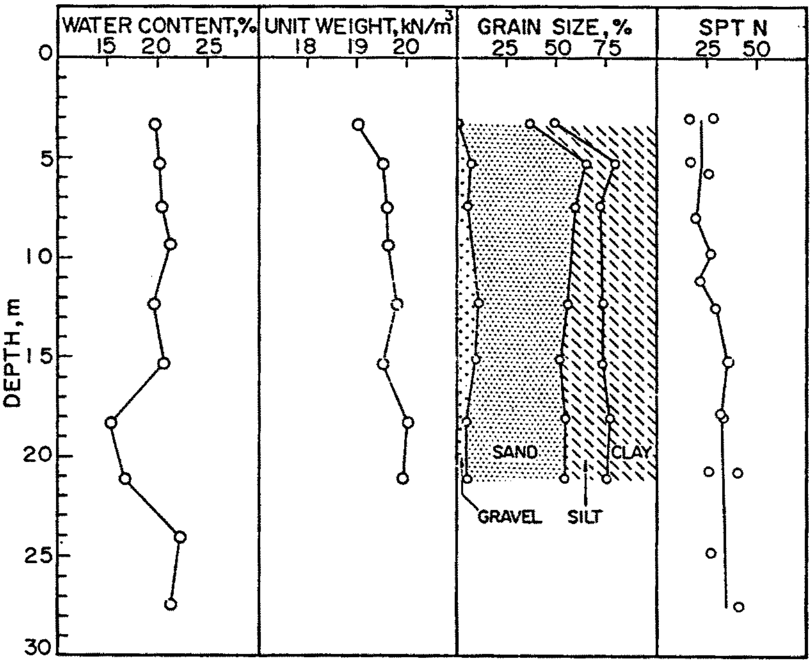

FIG. 12 SOIL PROFILE AT FAILED SLOPE - CASE 2

Three types of shear strength tests were carried out on the soils from this uniform layer. They were multi-stage CIU, single-stage $\overline{CIU}$ and pore water pressure controlled $\overline{CAU}$ tests. All the tests were carried out on samples taken form 7.2 - 8.2m depth The effective angle of shearing resistance obtained from the three types of tests varied form 32.9° to 33.8° and the effective cohesion intercept from 3.2 kN/m² to 15.3 kN/m².

Several sliding surfaces including the observed failure surface was considered in the stability analyses. Four representative surfaces are shown in Fig. 13. Results of the stability analyses are summarized in Table 2. It can be seen that shear strength parameters obtained from all these tests gave rather low factor of safety. Moreover, the position of the critical surface was different from the actual failure surface. The effective strength parameters obtained from pore water pressure controlled $\overline{CAU}$ test gave the highest factor of safety which is close to unity.

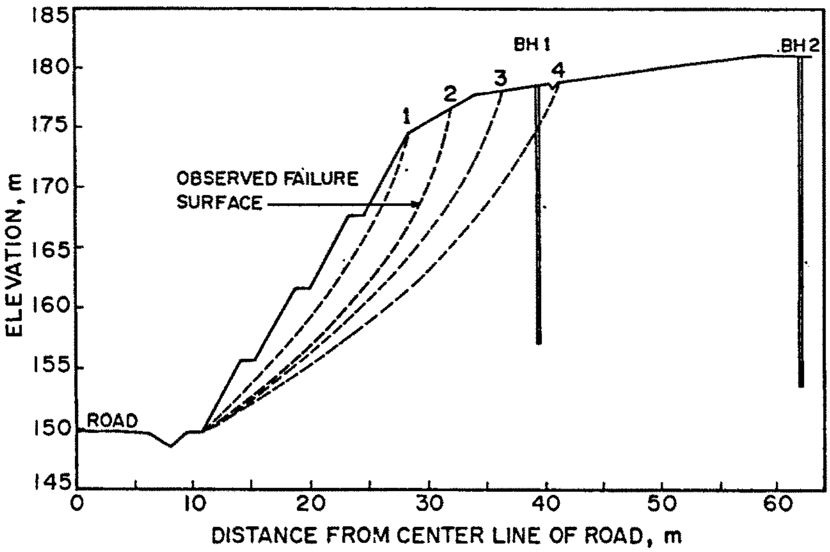

FIG. 13 SLOPE GEOMETRY AND SLIDING SURFACES – CASE 2

CONCLUSIONS

The residual soil cover found in the granitic hills is generally 20-30m thick, but in some places it is more than 45m and in other places it is as thin as 6m. The soil cover is thinner at slopes located in higher altitudes than at those slopes located in lower altitudes. In general, the residual soil cover can be divided into three zones though in some places one or two of these zones are absent. Of the total thickness of the soil cover, 50-70% is Zone 1 at the top of the soil profile and the rest is Zone 2 and 3 approximately in equal proportions. The consistency of the soils in the three zones are clearly different, the SPT N values recorded for Zones 1, 2 and 3 are 10-20, 30-40 and more than 100 respectively. In general, the coarse grain fraction in the soil increases with depth, about 55% in Zone 1 and 70% in Zones 2 and 3. The clay portion in the fine fraction of Zone 1 varies within the 0-8m depth range. At shallower depths the percentage of clay is higher. This may be due to the transformation of feldspar sand into clay during weathering of the soil.

Majority of the shear strength tests carried out in this project are multi-stage $\overline{CIU}$ tests and the results show that the effective angles of shearing resistance obtained for the soils in Zones 1, 2 and 3 are 31°-36°, 30°-40° and 25°-40° respectively. Variation of the angle of shearing resistance obtained for soils from Zones 2 and 3 are larger than that of Zone 1 and this could be partly due to the disturbance during sampling.

Results of the comparison of shear strength parameters obtained from different types of tests can be summarized as follows.

a) Single-stage $\overline{CIU}$ tests give smaller cohesion intercepts and higher angle of shearing resistance than those obtained from multi-stage $\overline{CIU}$ tests.

b) Anistropically consolidated pore water pressure controlled triaxial tests give higher value of angle of shearing resistance than those obtained from both the single-stage $\overline{CIU}$ and multi-stage $\overline{CIU}$ tests. The cohesion intercepts obtained from the pore water pressure controlled tests are also higher than those obtained from single-stage $\overline{CIU}$ tests.

c) Comparison of the results of few direct shear tests with that of the multi-stage $\overline{CIU}$ tests show that direct shear test tends to give higher value of the angle of shearing resistance.

Back analyses of the failed slopes using the strength parameters obtained from the different shear tests showed that safety factors estimated for the observed sliding surfaces were less than one in most cases. This suggests that the laboratory tests may be under estimating the actual shear strength parameters of soils found in the present project area. The strength parameters, from pore water pressure controlled tests appeared to give the factors of safety nearest to one.

ACKNOWLEDGEMENTS

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