STRENGTH PARAMETERS OF RESIDUAL SOILS AND APPLICATION TO STABILITY ANALYSIS OF ANCHORED SLOPES

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ABSTRACT

Residual soils in Malaysia have indicated that the range in thickness of weathering profiles and their engineering properties vary widely. Some authors had successfully identified ranges in SPT 'N' values with weathering profiles but none had attempted to correlate SPT 'N' values with strength parameters. The geotechnical properties of residual soils obtained for a section of an expressway near Kampar, Malaysia are processed and analyzed. Correlations between strength parameters and SPT 'N' values are shown to exhibit good linear relationships. The stability of anchored cut slopes in the same expressway is also reviewed. The computer program, PCSTABL5M, utilizing Janbu's method was used for the slope analysis. Seven cases representing variation in cohesion and ground water profile were analyzed for each slope section.

INTRODUCTION

The consistent growth in Malaysia's economy for the last 8 years or so has inevitably led to a boom in the construction industry. New townships, high standard expressway and very ambitious recreational theme parks are just some of the projects that have been recently implemented. Residual soils form the largest group in the peninsula and is not well understood yet as they are extremely heterogeneous materials and difficult to sample and test.

In view of some of the constraining factors like space and environmental issues, present construction activities especially on hillside slopes in Malaysia have required the use of slope stabilization measures. These steep slopes are very often in residual soils. The present trend in the construction activities in this area is however in much need of some useful information in regard to performance under local conditions.

LOCATION OF EXPRESSWAY

The section of the expressway studied is located approximately 50 km south of Ipoh City in the state of Perak and is oriented in the north-south direction. The length of the section

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of the expressway is approximately 5 km. Particular locations along the expressway have been identified by chainages (CH) in meters. The expressway section begins at CH 27+000 at the north and ends at CH 32+000 at the south.

GEOLOGY

The major portion of the expressway alignment traverses weathered rock of metamorphic origin. The metamorphic rocks are the oldest unit in the area and are thought to be of the permo carboniferous age. It comprises alternating units of quartz mica schist, graphitic schist and quartzite. The geology at CH 27+000, the north section of the expressway, is graphitic schist. Further south, the geology gradually changes to quartz schist and quartzite. The thickness of weathering grade VI zone varies between 1.5 m and 1.4 m, that of weathering grade V zone between 1 m and 35 m, and weathering grade IV zone between 2 m and 32 m. In the steeper topography rock outcrops were evident and this led to the wide range in thickness of the respective weathering grades. The overburden in these steep areas had been eroded exposing the bedrock. Within the same spur, the weathering grade zones V and VI generally indicated greater thickness with higher elevation and closer proximity to the spur ends. This observation concurs with that expected in humid tropics (Geological Working Party Report, 1990), and has been attributed to enhanced subsurface drainage which allows deeper penetration of weathering.

TOPOGRAPHY

The natural topography along the expressway alignment can be described as undulating hills. The expressway is seated between an elevation of 50 m and 200 m above mean sea level. The hills are located on the east side of the expressway. The natural slope gradient across the expressway alignment was gentle in the north (CH 27+000 to CH 29+500) and tended to be steeper at the south. The slope gradients varied between 23° and 34° in the weathered schist.

ENGINEERING PROPERTIES

The data collected and processed to characterize the engineering properties were obtained from a soil investigation report on 51 deep boreholes in the weathered schist. Field tests involved Standard Penetration Test at 1.5 m intervals and field permeability tests. The description of retrieved samples in the report followed outlines of BSI (1981), that is both soil and geological description. The six fold weathering classification scheme described in BSI (1981) is very similar to that in ISRM (1978) and IAEG (1981) and assigns lower weathering grade numbers to the least weathered material. The type and number of laboratory tests contained in the report on retrieved samples were bulk density (197), moisture content (512), particle size distribution (344), specific gravity (331), Atterberg Limit (275), consolidated undrained triaxial (23) and unconsolidated undrained triaxial (5).

The base of weathering grade IV zone was chosen to represent the lower limit of soil/

rock profile following the definition of tropical residual soils by the Geological Society Working Party (1990). A wide range in the measured engineering properties was observed for the respective weathering grades. The range in values are summarized in Table 1. It was observed from plots of index properties against depth for the various weathering grades that no distinct range was apparent to distinguish the different weathering grades. However, an increasing or decreasing trend was possible to be identified between the index properties with depth from linear regression through the calculated mean, taken at particular depth intervals. These observed trends does allow a preliminary assessment of the relative engineering behavior between the different weathering grades. The relative trend observed indicated that the bulk density increased and plasticity decreased for reduced weathering (Fig. 1 to Fig. 3).

Table 1 Geotechnical Parameters for Residual Soils of Schist Origin

Source	Location	Bulk Density	Moisture Content	Liquid Limit	Plasticity Index	Clay	Silt	Sand	Gravel	Specific Gravity	Void Ratio	Consolidated Undrained Triaxial Test			
												c	ϕ	c'	ϕ'
		(kN/m^3)	(%)	(%)	(%)	(%)	(%)	(%)	(%)			(kN/m^2)	(deg)	(kN/m^2)	(deg)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Komoo (1989)	Jalan Gurney Kuala Lumpur		10-48	25-90	18-38		75	10-20	< 5		0.5-0.9				
Ting, et al. (1990)	Senawang Negeri Sembilan	16-20	8-43	37-95	11-41	3-31	27-68	21-57	< 16			56-150	0-37	10-50	21-37
Singh and Ho (1990)	Gopeng Perak	16-17	43-52											30-136	14-28
This Study (1995)	Kampar Perak														
	W.Grade VI	14-20	7-49	32-61	7-29	5-58	9-40	7-49	0-74	2.6-2.66	0.7-1.3	13-40	10-38	8-30	18-45
	W.Grade V	14-22	1-57	27-61	0-30	0-35	0-69	0-94	0-55	2.59-2.66	0.4-1.6	30-95	8-30	9-71	13-42
	W.Grade VI	17-21	3-36	30-52	4-25	0-26	0-55	0-83	2-60	2.61-2.66	0.5-1.0	15-110	11-28	7-9	16-41

STRENGTH PARAMETERS

The strength tests were performed on the undisturbed samples obtained by mazier barrel (a type of triple tube core barrel with a retractable shoe). The flushing medium used to recover samples were either water or foam. The dimension of the mazier barrel was, length (1 m), external diameter (100 m) and internal diameter (72 mm). The type of triaxial tests comprised consolidation isotropic undrained (CIU) tests with pore pressure measurement and unconsolidated undrained (UU) tests. The samples for both tests were saturated prior to shear, implying that the shear strength parameters obtained were the minimum possible.

The range reported for both effective and total stress parameters were very wide as summarized below:

	c (kN/m²)	ϕ (degrees)	c' (kN/m²)	ϕ' (degrees)
Schist	30 -90	13-25	9-70	17-40

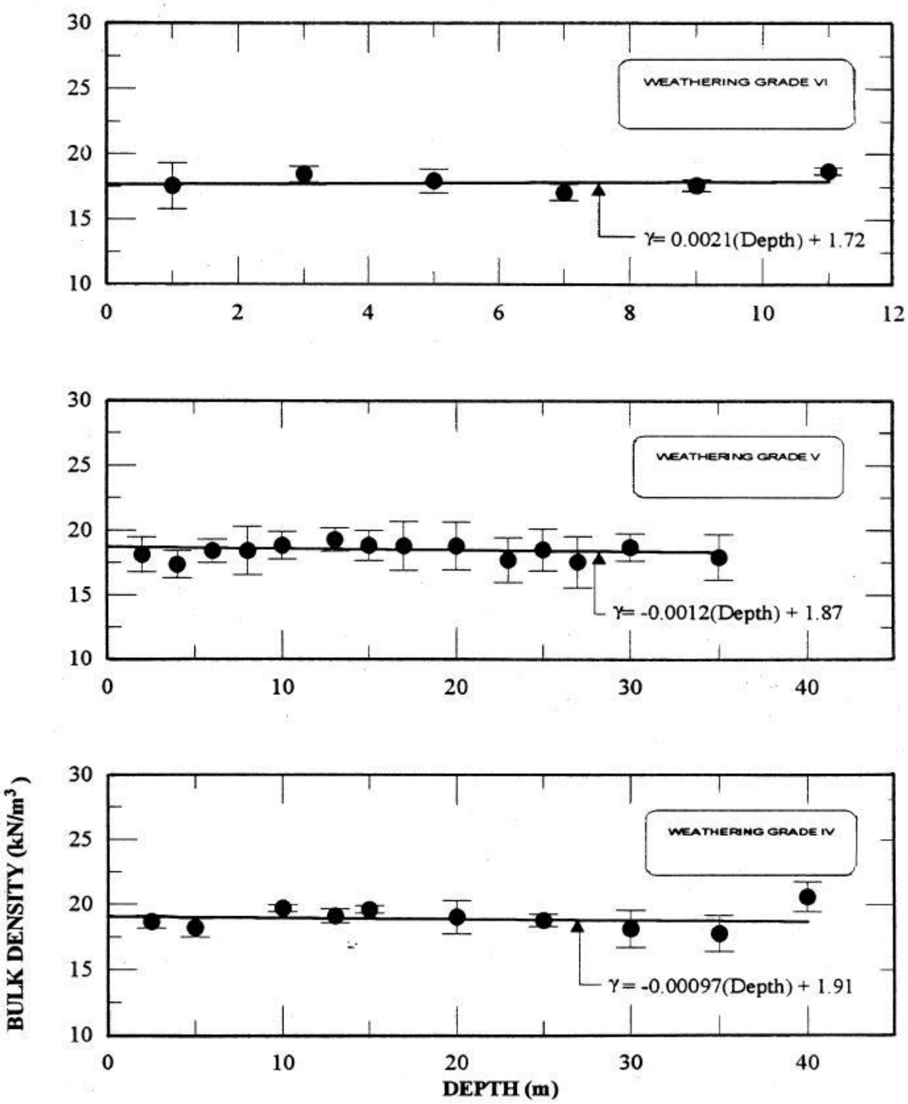


Fig. 1 Variation of Bulk Density with Depth for Weathered Schist

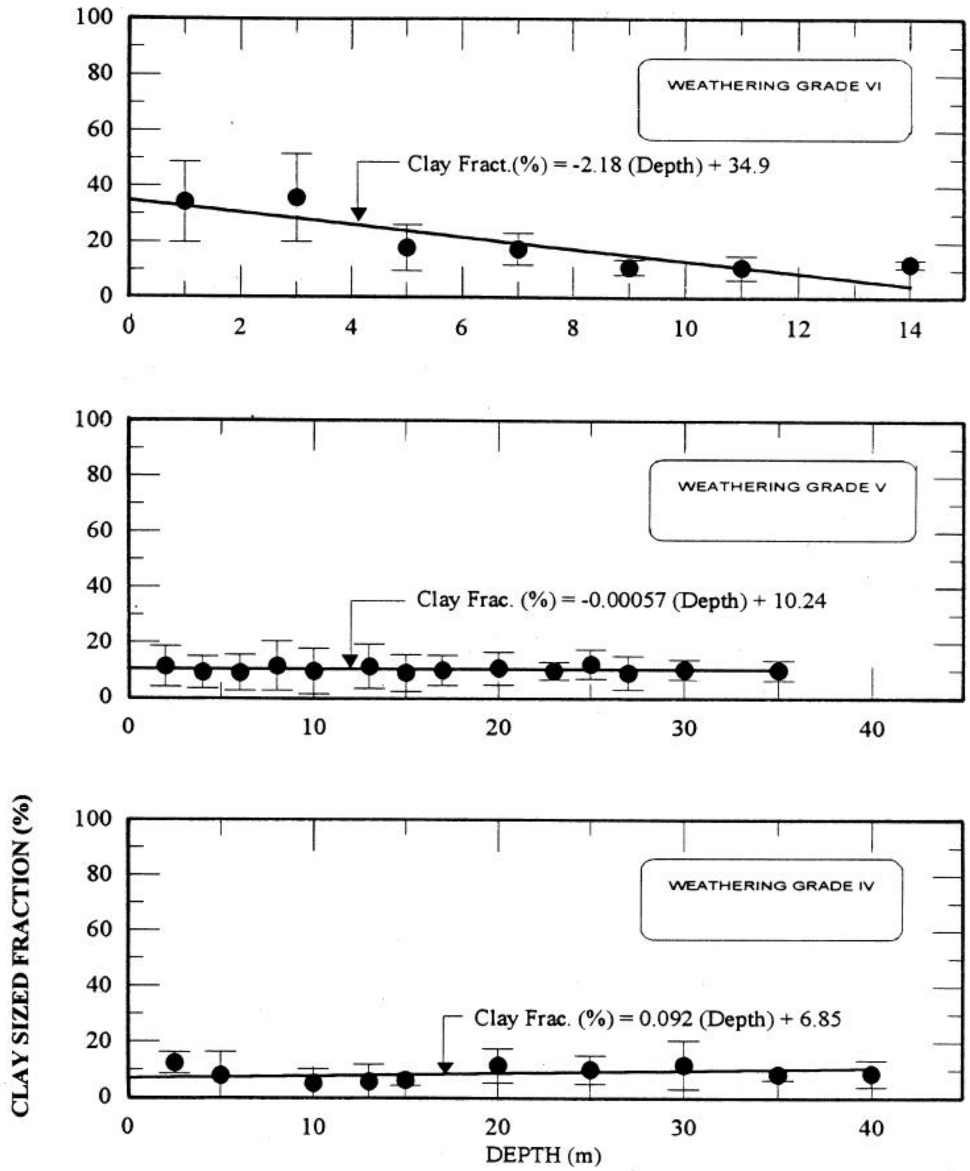


Fig. 2 Variation of Clay Sized Fraction with Depth for Weathered Schist

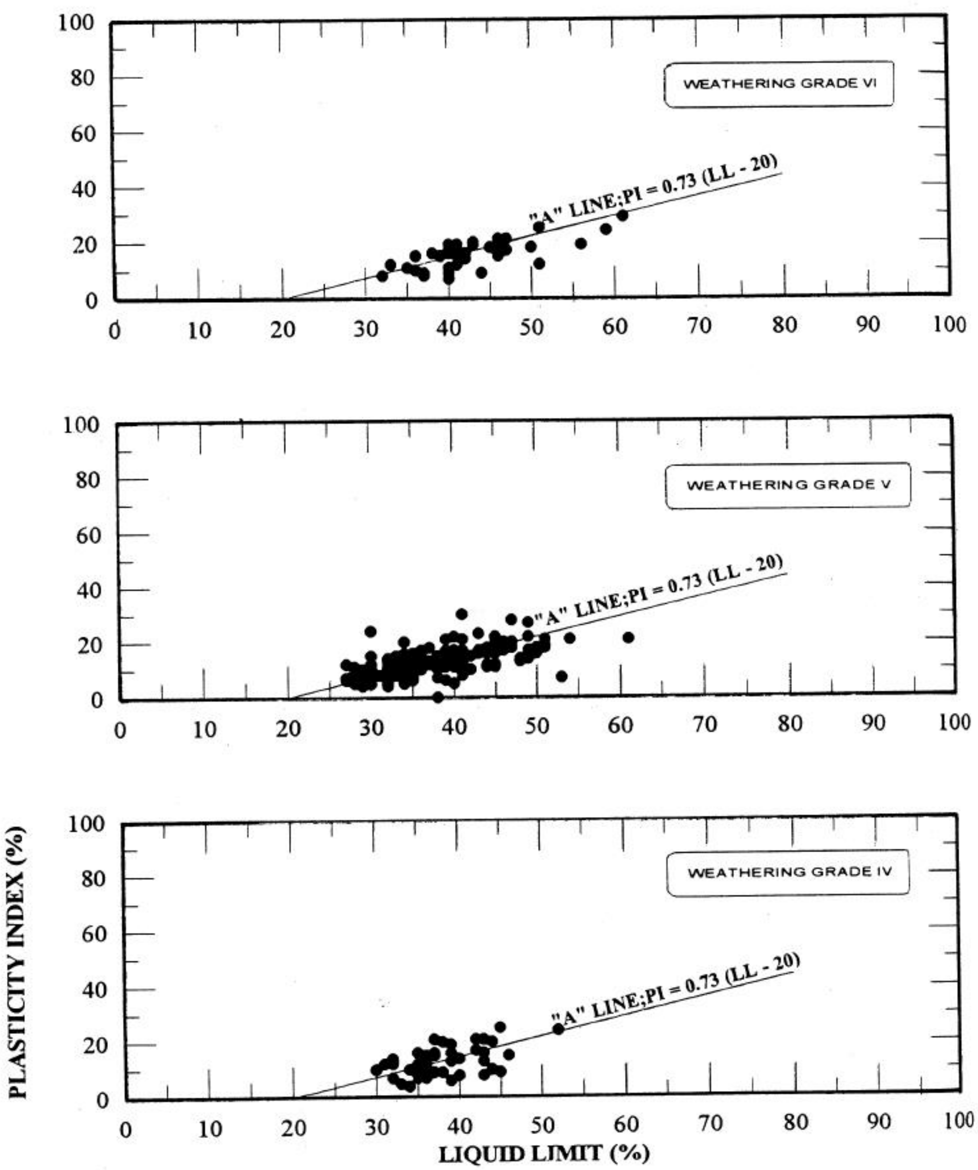


Fig. 3 Variation of Plasticity Index with Liquid Limit for Weathered Schist

Though the samples were saturated prior to shear, the higher values in the range reported in total stress parameter, ϕ, and effective stress parameter, c', were surprisingly high. For the high ϕ value, the high coarse fraction content noted on the samples is thought to have influenced the drainage condition during shear, thus approximating failure in a drained condition. However, for the higher range in value for c', it can be inferred that factors other than pore pressure effects have had an influence on this value. The Geological Society Working Party (1990) has attributed structure and bonding inherited from the patent rock to be the contributing factors.

It was observed that the Terzaghi and Peck (1976) linear relationship between undrained strength and SPT 'N' for cohesive soils generally represented the plotted points (calculated from total stress parameters) quite well for all three weathering grades. These plots (Fig. 4 to Fig. 6) affirm the use of this relationship for preliminary assessment of undrained strength at this location for this residual soil. It also shows that the material behaves in a cohesive manner under the test conditions.

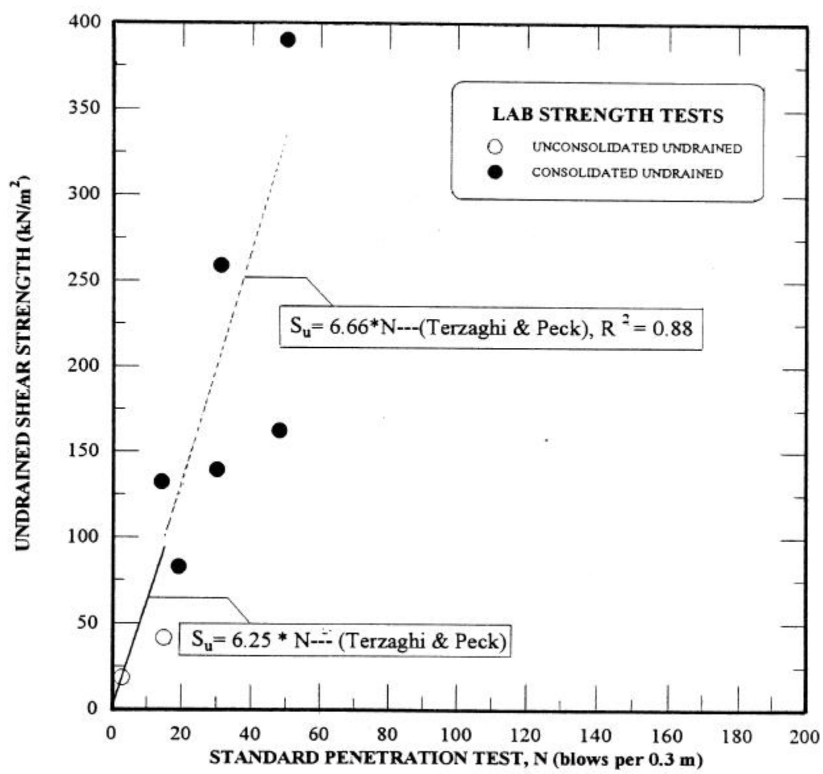


Fig. 4 Variation of Undrained Shear Strength with SPT N for Weathered Schist in Grade IV Zone

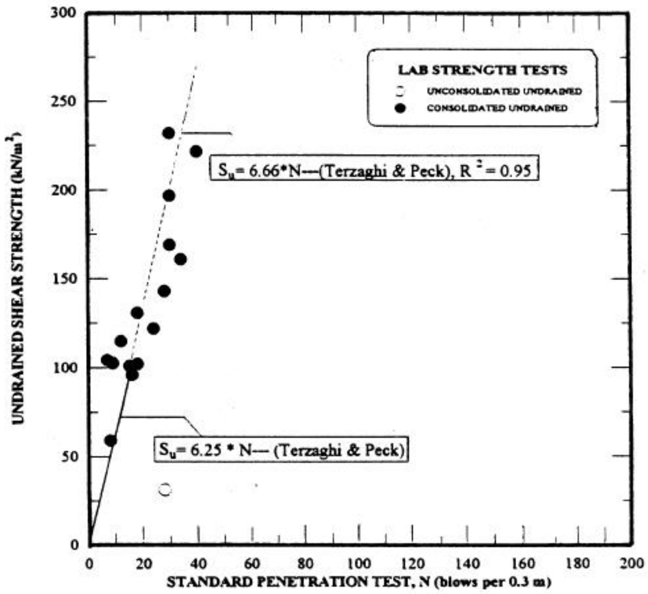


Fig. 5 Variation of Undrained Shear Strength with SPT 'N' for Weathered Schist in Grade V Zone

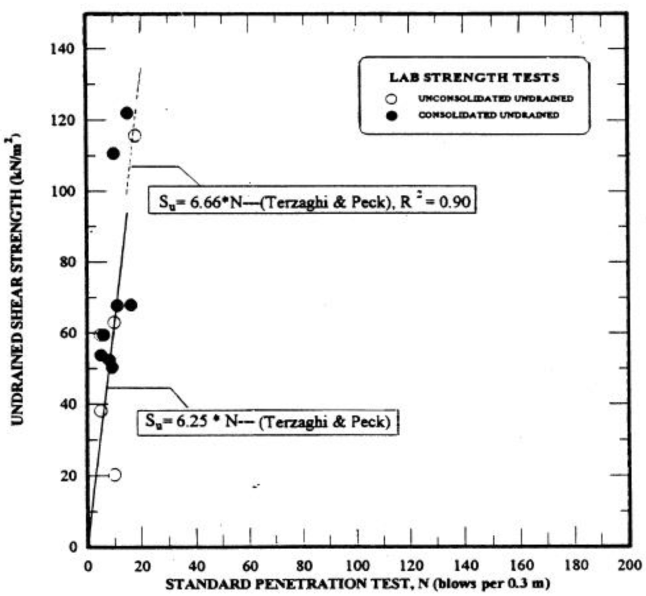


Fig. 6 Variation of Undrained Shear Strength with SPT 'N' for Weathered Schist in Grade VI Zone

For effective parameters, good linear correlation was obtained between ϕ' against SPT 'N' and c' against ϕ' (Fig. 7 and Fig. 8). The clay content was observed to have no influence on the location of the points in Fig. 7 but however in Fig. 8 it was noted that the higher c' values were generally associated with lower clay contents. It was noted that the lower clay percentages (<10%) generally plotted towards the higher c' range (>30 kPa) and the higher clay percentages (>10%) generally plotted towards the lower c' range. Some possible reasons thought to attribute to the high cohesion values are inter-particle bonding and cementation present in less weathered rock (Geological Society Working Party, 1990). The clay content in this site, did show lower values with the less weathered rock and thus the preceding reason may be a valid explanation for the higher cohesion values. The good linear relationship of c' against ϕ' plot with the ϕ' against SPT 'N' value plot do seem to suggest themselves as useful plots to predict the effective stress parameters c' and ϕ' from SPT 'N' values for the residual soils of schist in this location.

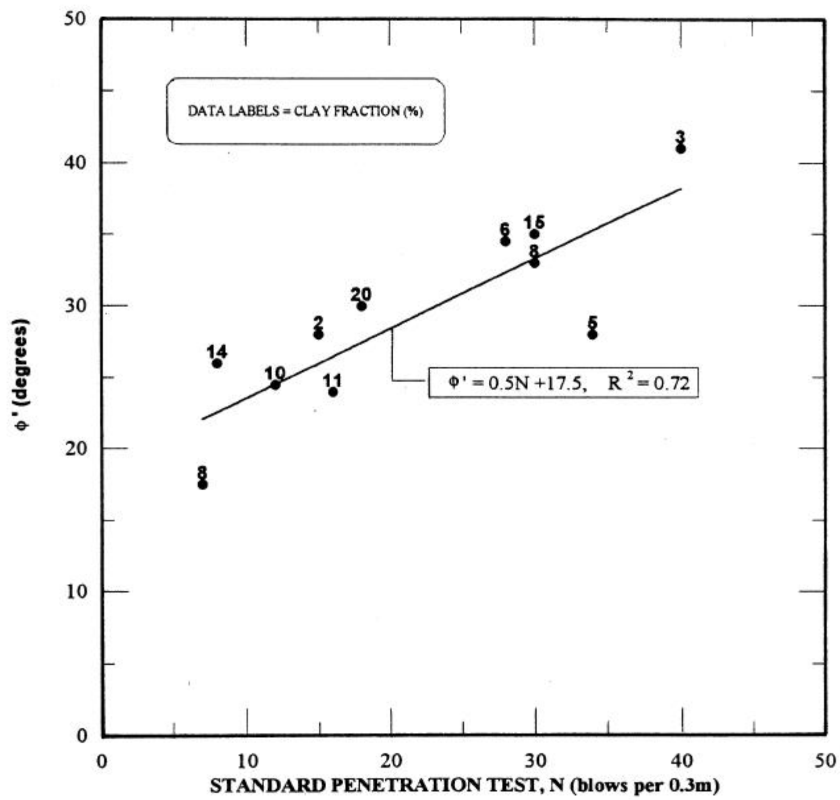


Fig. 7 Variation of ϕ' with SPT 'N' for Weathered Schist

STABILITY ANALYSIS OF ANCHORED SLOPES

Three separate cut slope sections along this expressway were stabilized with ground anchors. The ground anchored stabilized cut slope sections have been referenced by hill numbers. The hills were numbered from the start of the expressway according to the naturally occurring spurs that were intercepted by expressway. Thus Hill 8 was the cut slope section encountered on the eighth spur along the expressway. Similarly the other two sections were identified as Hill 10 and Hill 11.

Topography

Hill 8 is located on the east side of the expressway between CH 29+750 and CH 29+950 and is approximately 200 m in length. The axis of the hill is perpendicular to the expressway at CH 29+810 and the ridge is convex in shape. The natural gradients of the hill taken perpendicular to the expressway alignment is between 22° and 34° with the gentler gradients occurring at the spur ends. The deepest cut in this hill is 44 m.

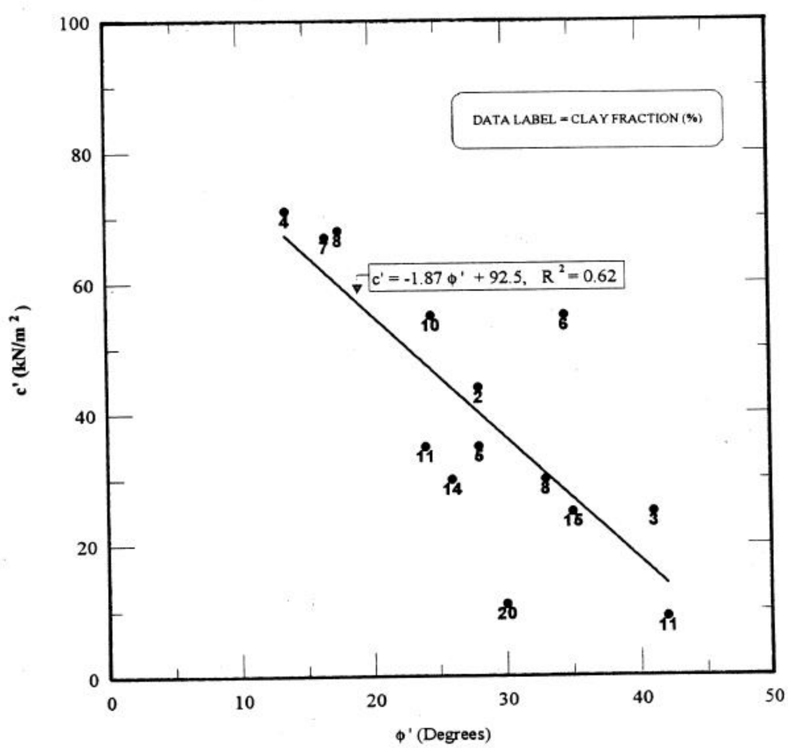


Fig. 8 Variation of c' with ϕ' for Weathered Schist

Hill 10 is located on the east side of the expressway between CH 30+060 and CH 30+280 and is approximately 220 m in length. The axis of the hill is convex in shape at the ridge and is oriented at the acute angle of 70° in plan view to the expressway alignment pointing southwards. The natural gradients of the hill taken perpendicular to the expressway alignment varies between 20° and 35° with the gentler gradients occurring at the spur ends. The deepest cut in this hill is 53 m.

Hill 11 is located on the east side of the expressway between CH 30+340 and CH 30+500 and is approximately 160 m in length. The axis of the hill is convex in shape at the ridge and is oriented at an angle of 60° in plan view to the expressway alignment pointing southwards. The natural gradients of the hill taken perpendicular to the expressway alignment is relatively constant at 28°. The deepest cut in this hill is 49 m.

Geology

The geology in Hill 8, as reported from deep boreholes is quartz mica schist. The thickness of weathering grade zones V and VI varied from 5 m to 17 m. It was observed that this thickness increased with increasing elevation as well as closer proximity to the spur ends on the hill. It was also noted that the rock quality designation (RQD) in the weathering grade zones III and lower, recorded higher percentages at shallower depths, with decreasing elevation on the hill.

In Hill 10 the geology is quartz mica schist with an igneous intrusion of Aplite dike. The depth to the Aplite dike from the ground surface reduced with decreasing elevation on the hill from 20 m at elevation 216.6 m to 5 m at elevation 188.4 m. The weathering grade of the aplite dyke varied between weathering grade zones III and V. The thickness of the Aplite dyke reduced with decreasing elevation from 40 m at elevation 216.6 m to 5 m at elavation 173.3 m. The quartz mica schist above the Aplite dyke was in weathering grade VI zone whilst that below was in weathering grade zones IV and lower. The deep boreholes indicated that the expressway is seated on the quartz mica schist well below the Aplite dike. The total thickness of the weathering grade IV zone on this hill was between 18 m and 30 m and was observed to show a tendency of increasing thickness in the south direction. The RQD was generally above 50% in the weathering grade III zone.

Similar to Hill 8 the geology in Hill 11 is quartz mica schist. The deep boreholes indicated that the exposed cut slope were in weathering grade zones V and VI. The thickness of weathering grade zone VI varied between 12 m and 40 m. The thickness was observed to increase with increasing elevation. The lowest weathering grade zone reported throughout the depth of the deep boreholes (between 30 m and 50 m) on this hill did not decrease beyond zone V.

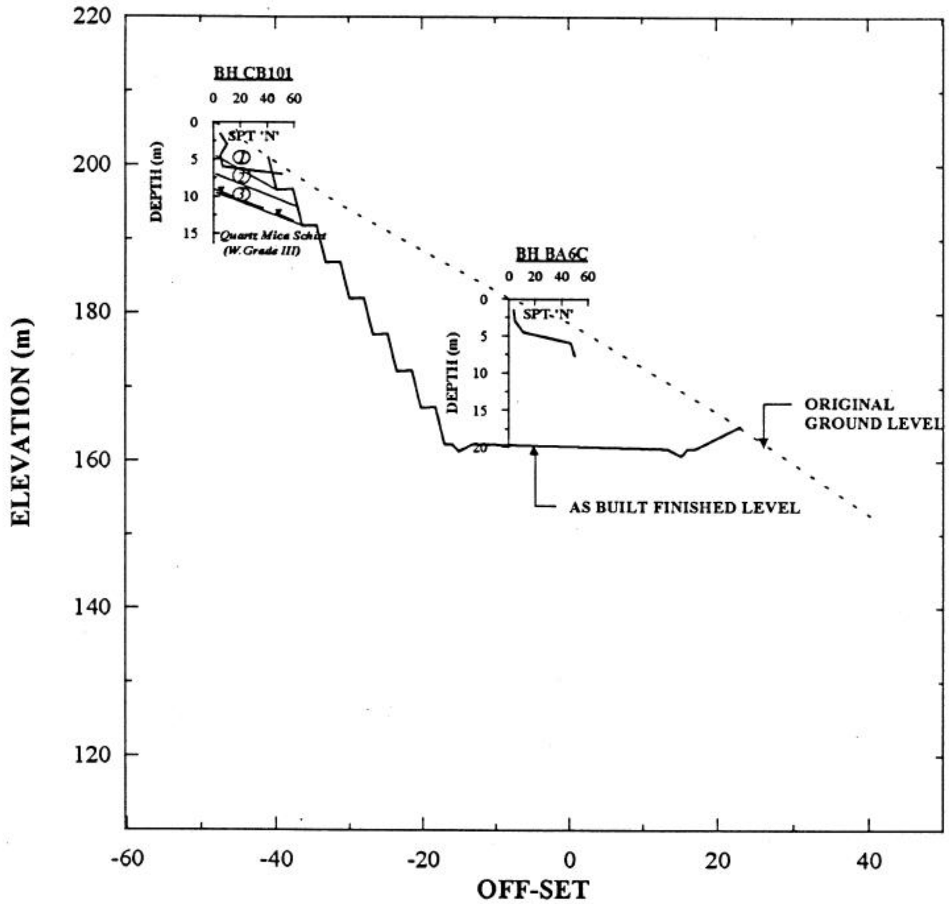
Geotechnical Properties

In Hill 8, the top 5 m was generally silty clay with clay content between 17% and 40%.

Below 5 m was sandy silt with a clay content less than 6%. The sand/gravel content generally exceeded 50% in these 2 layers. The moisture content generally stayed between 25% and 30%. The liquid limit generally varied between 40% and 45% with plasticity index approximately constant at 15%. The SPT 'N' blow count varied between 4 and 8 in the silty clay layer but exceeded rapidly above 10 reaching values greater than 50 in the sandy silt layer. The drained strength parameters for the respective layers were calculated from the linear relationship represented between ϕ' against SPT 'N' value and c' agains ϕ' which were obtained from samples tested in the schist geology. The drained strength parameters between CH 29+750 and CH29+820 for layer 1 were $c' = 15$ kPa and $\phi' = 20°$ with bulk density being 17.9 kN/m^3 and for layer 2 were $c' = 30$ kPa and $\phi' = 22.5°$ with bulk density being 18 kN/m^3. The drained strength parameters between CH 29+820 and CH29+950 for layer 1 were $c' = 15$ kPa and $\phi' = 21.5°$ with bulk density being 18 kN/m^3 and for layer 2 were $c' = 30$ kPa and $\phi' = 35°$ with bulk density being 18.3 kN/m^3. A typical cross-sectional profile is illustrated in Fig. 9 for CH 29+800.

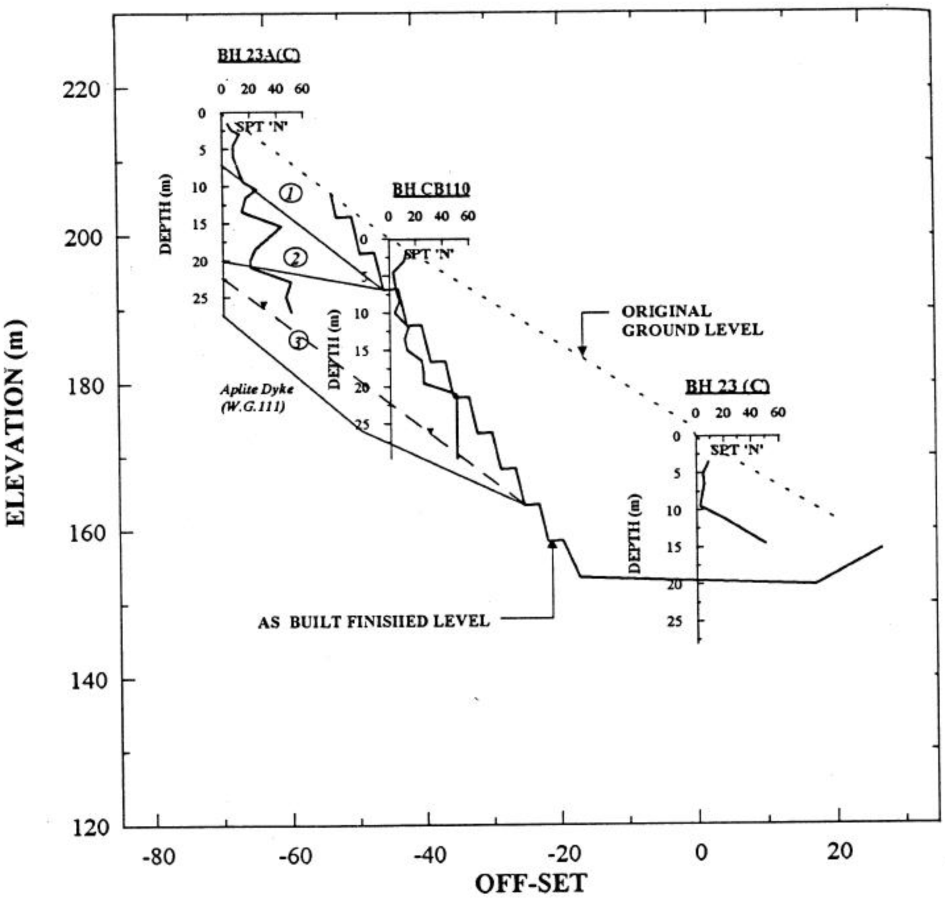
In Hill 10, the top 6 m was silty clay with clay content between 11% and 58%. Below 6 m was sandy silt with clay content less than 8%. The sand/gravel content generally exceeded 60% in these 2 layers. The moisture content varied between 15% and 25%. The liquid limit varied between 35% and 50% with the plasticity index varying between 10% and 15%. The average SPT 'N' blowcount in the silty clay layer increased from 4 to 10 in the south direction. The SPT 'N' blowcount in the sandy silt layer varied between 5 and 15 between the depth of 6 m and 18 m but exceeded rapidly reaching values greater than 50 beyond these depths. The drained strength parameters for the respective layers were calculated from the linear relationship represented between ϕ' against SPT 'N' and c' against ϕ' which were obtained from samples tested in schist geology. The drained strength parameters in the Aplite dike were obtained from shear box test results recorded on a block sample. The drained strength parameters for the silty clay layer were $c' = 25$ kPa and $\phi' = 21.5°$ with bulk density being 17.9 kN/m^3 and for the sandy silt layer were $c' = 30$ kPa and $\phi' = 27.5°$ with bulk density being 18.9 kN/m^3. The drained strength parameters for the Aplite dike were $c' = 18$ kPa and $\phi' = 35°$ with bulk density being 17.8 kN/m^3. A typical cross-sectional profile is illustrated in Fig. 10 for CH 30+180.

In Hill 11, the top 5 m was clayey silt with clay content between 8% and 26%. Below 5 m was sandy silt with clay content less than 8%. The sand/gravel content generally exceeded 70% in these 2 layers. The moisture content varied between 4% and 33%. The liquid limit varied between 28% and 41% with the plasticity index varying between 6% and 12%. The average SPT 'N' blowcount in the clayey silt layer was 10. In the sandy silt layer the SPT 'N' values varied between 12 and 30 in the weathering grade VI zone but always exceeded 50 in the weathering grade VI zone. The drained strength parameters for the respective layers were calculated from the linear relationship represented between ϕ' against SPT 'N' and c' against ϕ' which were obtained from samples tested in the schist geology. The drained strength parameters for the clayey silt layer were $c' = 15$ kPa and $\phi' = 23.5°$ with bulk density being 17.9 kN/m^3 and, for the sandy silt layer in weathering grade VI zone were $c' = 30$ kPa and $\phi' = 27.5°$ with bulk density being 18.1 kN/m^3. For the sandy silt layer in



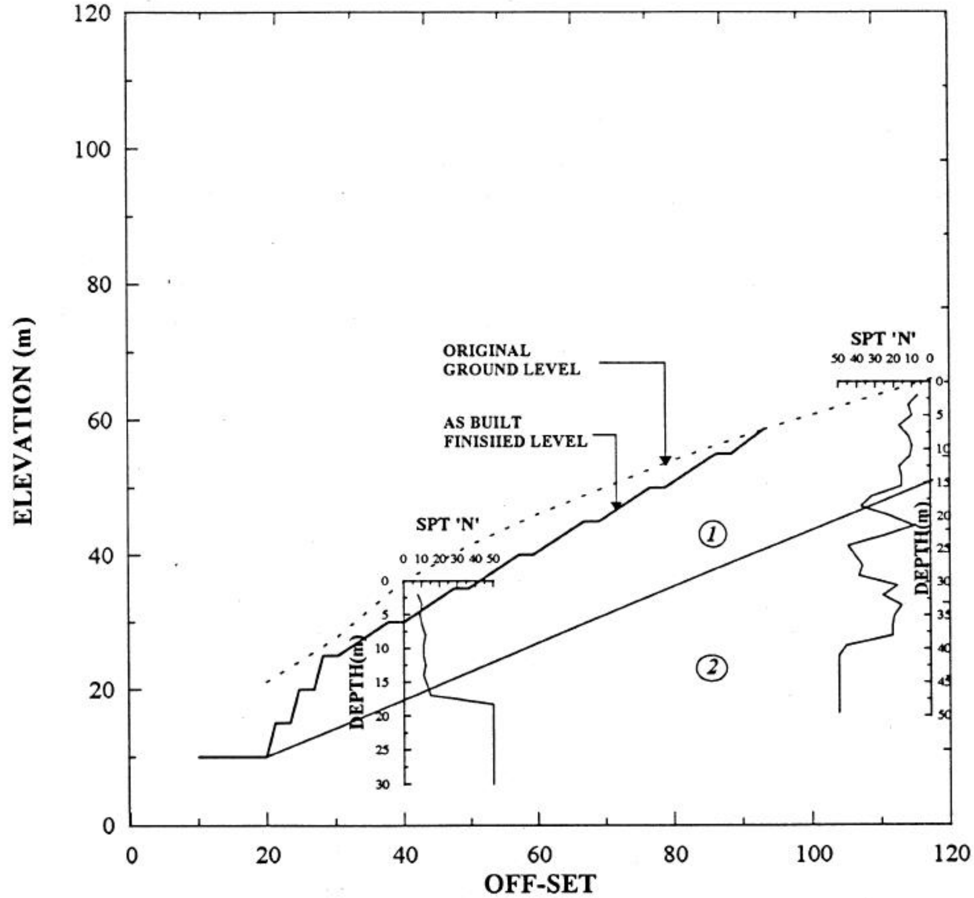
Layer No.	Descrpt.	Geology	W.Grade	Bulk Density (kN/m³)	c' (kN/m²)	phi'	w (%)	L.L (%)	P.I. (%)	Clay (%)
1	Silty Clay	Quartz Mica Schist	VI	17.9	15	20	20-33	42-45	16-20	18-34
2	Sandy Silt	Quartz Mica Schist	V	18	30	22.5	20-25	37-46	10-15	6-10
3		Quartz Mica Schist	IV	22	0	45	6-19	-	-	-

Fig. 9 Typical Soil Strength Profile at CH 29+800 (Hill 8)



Layer No.	Descrpt.	Geology	W.Grade	Bulk Density (kN/m³)	c' (kN/m²)	phi'	w (%)	L.L (%)	P.I. (%)	Clay (%)
1	Silty Clay	Quartz Mica Schist	VI	17.9	25	21.5	15-29	46-51	12-16	8-58
2	Sandy Silt	Quartz Mica Schist	V	18.9	30	27.5	7-25	38-41	5-15	2-8
3	Sandy Silt	Aplite Dyke	IV	17.8	18	35	13-23	34-36	8-15	6-10

Fig. 10 Typical Soil Strength Profile at CH 30+180 (Hill 10)



Layer No	Descrpt.	Geology	W.Grade	Bulk Density (kN/m³)	c' (kN/m²)	phi'	w (%)	L.L (%)	P.L (%)	Clay (%)
1	Clayey Silt	Quartzite	VI	17.9	15	23.5	14-33	28-37	8-12	8-26
2	Sandy Silt	Quartzite	V	18.5	30	35	4-25	33-41	6-10	3-7

Fig. 11 Typical Soil Strength Profile at CH 30+420 (Hill 11)

weathering grade V zone the drained strength parameters were $c' = 30$ kPa and $\phi' = 35°$ with bulk density being 18.5 kN/m^3. A typical cross-sectional profile is illustrated in Fig. 11 for CH 30+420.

Ground Anchors

The type of anchors, installation method and general geometry of the anchored slope were the same in all the hills. The type of anchors used was Type 'A' (BSI, 1989), that is tremie (gravity displacement) grouted straight shaft drill hole. The drill hole diameters were 150 mm. The drilling method adopted was rotary drilling with compressed air used as the flushing media. The prestress force for individual anchors varied between 120 kN and 500 kN transmitted through 15.24 mm diameter low relaxation prestressing strands. The lower capacity anchors were placed in the higher elevation of the cut slope and closer proximity to the spur ends, the reason being that the soil resistance was lower in these locations. The total length of anchors beneath the cut slope face was 21 m where soil was encountered throughout its length. The fixed length being 6 m. At locations where rock was encountered a minimum total length of 10 m was maintained with a minimum fixed length of 4 m beneath the rock.
The anchors were installed at an inclination of 15° downwards from the horizonatl with center to center spacing of 4 m. The anchor forces were transmitted back to the surrounding ground through anchor pads. The dimension of the anchor pads were 1.5 m x 1.5 m with a thickness of 300 mm. At locations where capacity was insufficient at the anchor pad location, soil nails were installed to transmit the bearing pressures to deeper and higher bearing strata. All anchors were prefabricated with double protection layers against corrosion. High density polyethylene (HDPE) was used as protection layers. The reasons for the stringent protective measures were that the pH detected from tested samples indicated acidic conditions ($pH < 7$) and that these were permanent anchors.

During the course of construction, all the anchors were subjected to on-site acceptance tests following the procedures described in BSI (1989). This test required loads of up to 1.5 times the working load to be applied in stages to the anchor together with displacement time measurements to monitor prestress losses. The number of anchors in the respective hill's were Hill 8 (267), Hill 10 (330) and Hill 11 (54).

Stability Analysis Results

The slope stability analysis was performed using a computer program PCSTABL5M developed by Purdue University, West Lafayette. The contribution of ground anchor forces in the slopes were analyzed in the program utilizing Flamant's formulae as proposed by Morlier and Tenier (1982) and the Simplified Janbu's method of analysis for non circular failure surface. For each slope cross-section, seven cases were analyzed. The seven cases were:

Case 1 - Slope with anchors and groundwater table as measured

Case 2 - Slope with anchors and groundwater table 1 m above that measured
Case 3 - Slope with anchors and groundwater table 2 m above that measured
Case 4 - Slope with anchors and dry condition
Case 5 - Slope without anchors and groundwater table as measured
Case 6 - Slope with anchors and groundwater table as measured and cohesion reduced by one third
Case 7 - Slope with anchors and groundwater table as measured and cohesion reduced by one half

Cases 2 and 3 represented cases due to rise in groundwater table caused by surface infiltration. The rise in water table was calculated using expression derived by Lumb (1975) that describes the formation of a wetting band (zone of 100% saturation) and is dependent on the porosity, permeability, initial and final degrees of saturation of the soil forming the slope and the anticipated ten year return period rainfall that will infiltrate into the ground directly above or behind the slope. The approach assumes that the wetting band descends vertically under the influence of gravity even after cessation of rain until it reachges the main water table. The surface of the main water table will rise with a consequent rise in pore pressure.

The expression proposed by Lumb (1975) is:

$$h = kt / n (S_f - S_o) \tag{1}$$

where:

h	=	depth of wetting front
k	=	coefficient of permeability
n	=	porosity
S_o	=	initial degree of saturation
S_f	=	final degree of saturation
t	=	duration of rainfall

The mean measured field permeability reported was 2.1 x 10^{-6} m/s, the maximum rainfall recorded during 24 hours was 194 mm for a return period of 10 years at the closest rainfall station to the site (Ingham and Bradford, 1960), the mean porosity calculated was 0.5 and the mean difference in degree of saturation was 20%. The wetting band thickness calculated from these values was 1.8 m. Thus, cases 2 and 3 were chosen to represent a range above and below the calculated value. Of particular interest to note was that the calculated value was close to the value usually reported for Hong Kong residual soils of 2 m (Brand, 1985).

Case 4 was run to establish the improvement in factor of safety if drainage facilities were introduced into the slope, that is lowering the ground water table below the most critical failure surface. Case 5 was run to establish the need of stabilization measures for the slopes analyzed and also to indicate the effect of slope stabilization measures on the improvement in factor of safety. Cases 6 and 7 represented cases of true cohesion or actual intercept of the

Mohr-Coulomb envelope. Sowers (1963) observed from triaxial test results on residual soils derived from igneous and metamorphic origin, that the true cohesion or actual intercept of the envelope on the shear axis is substantially less than the apparent cohesion. It was also reported that in many cases the reduction was half or one third of the apparent cohesion.

A total of 17 cross-sections were analyzed for slope stability between the three hills. All of these sections were stabilized with ground anchors. The geometry of the anchored portion of the slope was 4V:1H. The prestress anchor forces varied between 12 000 kg and 50 000 kg with a spacing of 4 m apart and the anchor lengths varied between 10 m and 21 m. The stability analysis was limited to the slope material in weathering grade IV and above as the boreholes reported the weathering grade III and lower material were competent rock. The rock line was estimated with the use of anchor drilling records and borelogs. The ground water profile was estimated from the piezometric levels recorded in the borelogs as well as observed seepage at weepholes after construction. The results of the slope analysis are summarized in Table 2, Table 3, and Table 4. Generally in all the cross-sections the computed factor of safety decreased with rise in groundwater table and fall in value of strength parameter c'. It was further observed that for the case with drop in c' values, the percentage drop in factor of safety (as compared to Case 1) was very much smaller with increasing thickness of weathering grade IV and higher material beneath the top of the anchored slope. It can be reasoned that a reduced c' value on lower slope heights has more pronounced effect on the reduction in factor of safety.

Typical output of the stability analysis indicating the location and shape of the crirical failure surface are shown in Figs. 12 to 14.

CONCLUSIONS

1. The relative trend observed for weathered schist indicated that the bulk density increased and plasticity decreased for decreasing weathering grades.

2. The limitation of the trends described in (1) was that for a particular weathering grade the range of values for an index property can be fairly wide. However, the observed trends does not allow a preliminary assessment of the relative engineering behavior between the different weathering grades.

3. The calculated undrained strength from total stress parameters concurred well with the Terzaghi and Peck (1967) empirical correlation with SPT 'N' values and affirms the use of this relationship for preliminary assessment of undrained strength at this location for this residual soil.

4. Fairly consistent linear relationships were exhibited by the schist residual soils from the plots between the effective stress parameter, ϕ' against SPT 'N' values and ϕ' against c'. These plots seem to suggest themselves as useful plots to predict the effective stress parameters form SPT 'N' values for the residual soils of schist in this location.

Table 2 Summary of Stability Analysis Results for Slopes at Hill 8

Chainage	29+780 (1)	29+800 (2)	29+820 (3)	29+840 (4)	29+860 (5)	29+900 (6)	29+920 (7)
Height of Cut Slope (metres)	27	41	44	42	40	33	25
Height of Anchored Slope (metres)	27	35	44	40	35	33	25
Geometry of Anchored Slope	4V:1H with 2.2m wide berms at 5m height intervals						
Gradient of Backslope (degrees)	31	34	29	28	34	23	22
Estimated Thickness of Weathering Grade IV and Higher Material at Top of Cut Slope (metres)	5	7	10	8	10	10	8
Number of Anchored Slopes in Weathering Grade IV and Higher Material	2	2	3	2.5	4	4	3
Total Anchor Force Per Metre Run Through Weathering Grade IV and Higher Material (kN/m)	137	33	203	170	412	550	412
Factor of Safety (Slope Stability)							
Case 1	2.115	1.452	1.876	2.499	1.399	2.53	2.329
Case 2	2.078	1.441	1.826	2.396	1.372	2.359	2.255
Case 3	2.024	1.355	1.741	2.284	1.295	2.216	2.163
Case 4	2.115	1.452	1.877	2.515	1.417	2.928	2.344
Case 5	1.48	1.314	0.95	1.503	0.497	1.304	1.418
Case 6	1.688	1.238	1.783	2.331	1.261	2.21	1.986
Case 7	1.478	1.108	1.727	2.247	1.211	2.05	1.84

Case 1 — Slope with anchors and ground water table as measured
Case 2 — Slope with anchors and ground water table 1m above that measured
Case 3 — Slope with anchors and ground water table 2m above that measured
Case 4 — Slope with anchors without ground water table (i.e. dry)
Case 5 — Slope without anchors and ground water table as measured
Case 6 — Slope with anchors, ground water table as measured and cohesion reduced by 1/3
Case 7 — Slope with anchors, ground water table as measured and cohesion reduced by 1/2

Table 3 Summary of Stability Analysis Results for Slopes at Hill 10

Chainage	30+120 (1)	30+140 (2)	30+160 (3)	30+180 (4)	30+200 (5)	30+220 (6)
Height Of Cut Slope (metres)	36.5	42	44	53	48	34
Height Of Anchored Slope (metres)	35	40	44	50	48	34
Geometry Of Anchored Slope	4V:1H with 2.2m wide berms at 5m height intervals					
Gradient Of Backslope (degrees)	26	33	33	35	23	20
Estimated Thickness Of Weathering Grade IV And Higher Material At Top Of Anchored Slope (metres)	20	23	28	28	28	21
Number Of Anchored Slopes In Weathering Grade IV And Higher Material	6.5	7	7	8.5	8.5	5
Total Anchor Force Per Metre Run Through Weathering Grade IV And Higher Material (kN/m)	652	652	616	754	891	514
Factor Of Safety (Slope Stability)						
Case 1	1.554	1.317	1.212	1.239	1.261	1.438
Case 2	1.513	1.303	1.209	1.199	1.226	1.416
Case 3	1.468	1.264	1.183	1.177	1.199	1.377
Case 4	1.556	1.372	1.26	1.299	1.293	1.44
Case 5	0.971	0.967	0.95	0.907	0.933	1.051
Case 6	1.477	1.208	1.11	1.13	1.14	1.339
Case 7	1.44	1.149	1.059	1.072	1.079	1.288

Case 1 --- Slope with anchors and ground water table as measured
Case 2 --- Slope with anchors and ground water table 1m above that measured
Case 3 --- Slope with anchors and ground water table 2m above that measured
Case 4 --- Slope with anchors without ground water table (i.e. dry)
Case 5 --- Slope without anchors and ground water table as measured
Case 6 --- Slope with anchors, ground water table as measured and cohesion reduced by 1/3
Case 7 --- Slope with anchors, ground water table as measured and cohesion reduced by 1/2

Table 4 Summary of Stability Analysis Results for Slopes at Hill 11

Chainage	30+400 (1)	30+410 (2)	30+420 (3)	30+440 (4)	30+460 (5)
Height Of Cut Slope (metres)	45	47	49	41	28
Height Of Anchored Slope (metres)	15	15	15	10	5
Geometry Of Anchored Slope	4V:1H with 2.2m wide berms at 5m height intervals				
Gradient Of Backslope (degrees)	28	28	28	28	28
Estimated Thickness Of Weathering Grade IV And Higher Material At Top Of Anchored Slope (metres)	>15	>15	>15	>10	>5
Estimated Thickness Of Weathering Grade VI Material At Top Of Anchored Slope (metres)	7.5	8	11	8	4
Number Of Anchored Slopes In Weathering Grade VI Material	2	2	3	2	1
Total Anchor Force Per Metre Run Through Weathering Grade VI Material (kN/m)	275	275	412.5	275	137.5
Factor Of Safety (Slope Stability)					
Case 1	1.675	1.671	1.318	1.729	2.543
Case 2	1.675	1.671	1.318	1.729	2.543
Case 3	1.675	1.671	1.318	1.729	2.543
Case 4	1.675	1.671	1.318	1.729	2.543
Case 5	1.123	1.117	0.92	1.092	1.679
Case 6	1.509	1.506	1.19	1.534	2.135
Case 7	1.426	1.422	1.126	1.348	1.932

Case 1 — Slope with anchors and ground water table as measured
Case 2 — Slope with anchors and ground water table 1m above that measured
Case 3 — Slope with anchors and ground water table 2m above that measured
Case 4 — Slope with anchors without ground water table (i.e. dry)
Case 5 — Slope without anchors and ground water table as measured
Case 6 — Slope with anchors, ground water table as measured, cohesion reduced by 1/3
Case 7 — Slope with anchors, ground water table as measured, cohesion reduced by 1/2

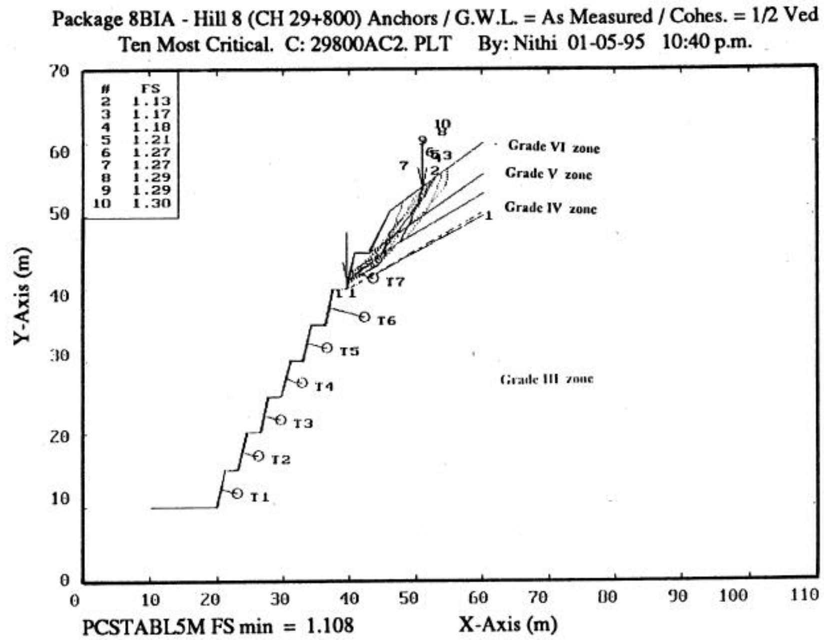


Fig. 12 Stability Analysis Result for CH 29+800 (Hill 8)

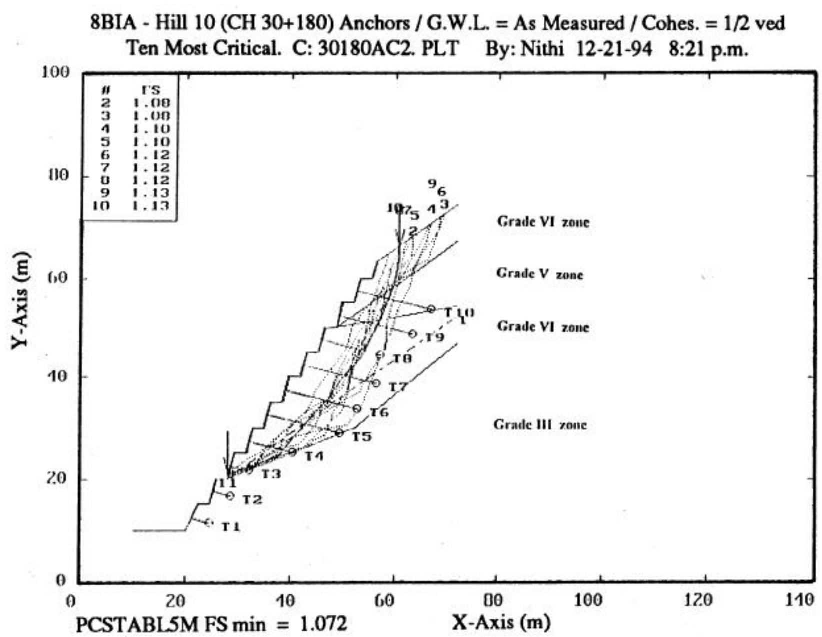


Fig. 13 Stability Analysis Result for CH 30+180 (Hill 10)

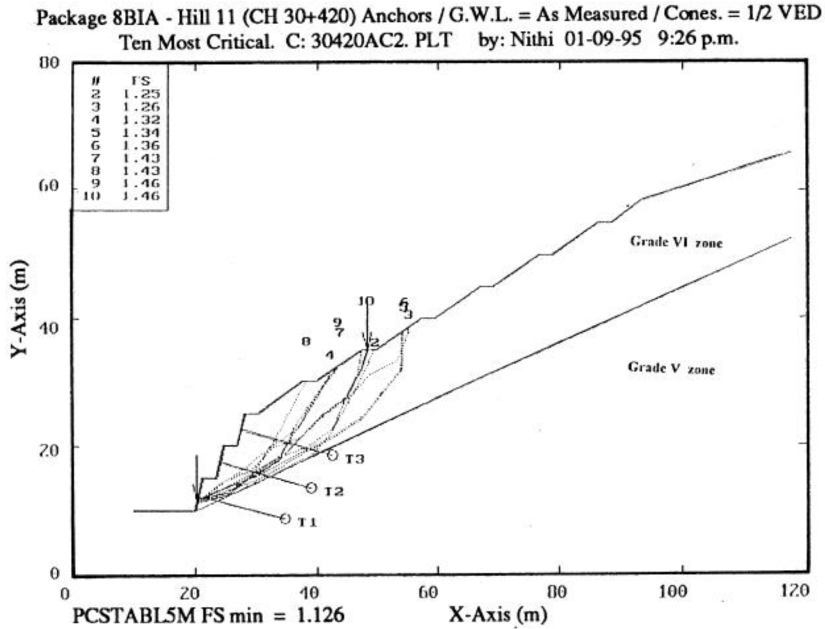


Fig. 14 Stability Analysis Result for CH 30+420 (Hill 11)

5. In the analysis of slopes it was noted that the results indicated that the percentage drop in the factor of safety was more critical for the case with reduced c' value compared to the case with the rise in water table. For the case with lower c' value, the percentage reduction in factor of safety was much larger with lower slope heights.

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