# *Strength Behaviour of Phyllites Under Triaxial Stress Condition*

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**ABSTRACT**

Laboratory investigations on phyllites, obtained from Tehri area in lesser Himalaya have been reported herein. In the present study, an attempt has been made to determine the strength indices, physical properties, mineralogical composition and then to classify the phyllites on the basis of Deere and Miller (1966) chart. The effect of saturation on strength indices in unconfined and confined conditions has also been found. Thin section studies revealed that phyllites mainly contain biotite, quartz and felspar. Based on the uniaxial compressive strength and modulus ratio values, the phyllites fall in the category DL, on average both in dry and saturated conditions. Anisotropy curve has been found to be of U-type. Due to saturation, uniaxial strength is reduced by 20per cent. Triaxial tests at different confining pressures up to 50 MPa revealed that ultimate strength ($\sigma_1$) varied non-linearly with confining pressure ($\sigma_3$). This non-linearity decreases with saturation. Transition from brittle to ductile failure has been observed at about 22.5 MPa confining pressure in saturated condition and about 45 MPa in dry condition.

It has been observed that Hoek-Brown criterion is very much suited to phyllites in dry conditions. Whereas, Ramamurthy criterion compares well with the observed results in saturated condition. The strength parameters of the anisotropic rock mass have also been determined.

*Key Words:* Phyllites, triaxial tests, anisotropy, shear strength, rocks, stress-strain

## 1. INTRODUCTION

With phenomenal increase in civil engineering activities, selection of suitable sites is becoming increasingly limited, posing unforeseen challenges to the geotechnical engineers. This is particularly so in India where the developmental work for harnessing the natural resources need gigantic dams, deep tunnels, large underground cavities in the young Himalayas, and other places. Besides, geological discontinuities, the region of Himalayas is within the earthquake prone area.

For realistic and safer design, understanding of the strength and deformation behaviour of rocks and rock masses is of paramount importance.

Rock mass behaviour can best be understood by insitu tests. However, such insitu tests are difficult because of heavy loads involved and are expensive due to their massive scale of operation. As compared to this, laboratory tests on specimens of rock can be easily conducted under simulated field conditions to get a comprehensive understanding of rock behaviour. Accordingly, a systematic study on phyllites of Tehri area has been made. The investigation involves:

(i) Review of nature of phyllites, its mineralogical composition and engineering behaviour of rocks in general.
(ii) Determination of specific gravity of rock material, water absorption, porosity, density and sonic wave velocity.
(iii) Determination of strength indices by point load tests and Brazilian tests.
(iv) Uniaxial and triaxial compression testing of phyllites in dry and saturated conditions.

Having conducted the various tests and the interpretation of test results, following objectives have been achieved:

(i) Classification of phyllites based upon uniaxial compressive strength and modulus of elasticity.
(ii) Assessment of shear strength and deformation behaviour of phyllites under triaxial stress conditions in both dry and saturated states.
(iii) Selection of a suitable failure criterion.

## 2. THE ROCK STUDIED

Phyllites are broadly categorized as metamorphic rock. Phyllites mark a stage of more advanced metamorphism as a result of which its grain size is coarser than slates. In phyllites, saricite and chlorite with or without biotite exceeds 50 percent. Albite may amount to as much as 20 percent. Quartz is the major constituent. When the amount of quartz exceeds that of the phyllosilicates, the rock is termed as quartz-phyllites. In this study, rock specimens tested are from the Tehri area of lesser Himalaya. The rocks exposed in and around the Tehri area, are the phyllites of Chandpur series. These Chandpur rocks are in contact with Shimla slates. These phyllites are, at places, in contact with the younger dolomites and quartzites of Garhwal group.

The phyllites occurring in the area are banded in appearance and are classified into three categories: phyllites grade I, II and III on the basis of their lithological composition, physical competence and degree of tectonism. Grade I rocks constitute about 45 percent of the area. These are predominantly arenaceous, massive in character and distinctly jointed with compressive strength varying from 100 to 160 MPa. Grade II phyllites constitute about 25 percent of total rock exposed in the gorge. This intermediate variety of phyllites is composed of alternate bands of argillaceous and arenaceous materials and are considerably impregnated with quartz veins, both along and across foliations. Grade II phyllites are highly jointed and have five sets of joints. The compressive strength varies from 60 to 90 MPa. Grade III phyllites are generally weathered and schistose in composition with numerous joints, cleavages and minor pucker rings.

## 3. EXPERIMENTAL INVESTIGATIONS

An extensive laboratory experimental programme (Table 1) has been planned and executed carefully on phyllite cores of size 54 mm collected from Tehri area at IIT Roorkee, India (Prasad, 1998). The rocks tested are mainly Grade-III phyllites. Cores were cut and lapped to obtain specimens that meet the tolerance limits laid down by IS:9179-1979.

Table 1 - Experimental investigation programme

| Property | Test | Test Procedure | Test Condition | No. of Specimens Tested |
|---|---|---|---|---|
| Mineralogical composition | Thin section studies | ISRM (1981) | Dry | 2 |
| Physical properties | Water absorption<br>Density<br>Specific gravity<br>Porosity<br>Sonic wave velocity | IS:13030-1991<br><br><br><br>ISRM (1981) | | 3<br>10<br>3<br>3<br>2 |
| Strength index | Point load strength test<br><br>Brazilian test | IS:8764-1978<br><br>IS:10082-1981 | Dry<br><br>Dry | 28<br><br>11 |
| Strength properties | Uniaxial compression test<br><br>Triaxial compression test | IS:9143-1979<br><br>IS:13047-1991 | Dry<br>Saturated<br>Dry<br>Saturated | 11<br>07<br>18<br>25 |

## 4. RESULTS AND INTERPRETATION

### 4.1 Mineralogical Composition

Quantitative and qualitative analysis of phyllites have been attempted using the results of thin section studies. In these samples, alternate bands of biotite and felsic minerals were found. The rock contains about 50 per cent of biotite, 25 per cent of quartz, 15 per cent of felspar and remaining 10 percent accessory minerals which were not clearly visible.

### 4.2 Physical Properties

Physical properties, namely, water absorption, density, specific gravity, porosity and sonic wave velocity in dry state were determined and the values are presented in Table 2.

Table 2 – Physical properties of phyllites

| Sl.No. | Property | Mean Value |
|---|---|---|
| 1. | Water absorption, per cent | 0.65 |
| 2. | Density, kg/m$^3$ | 2.77 (dry)<br>2.81 (saturated) |
| 3. | Sonic wave velocity, km/sec. | 5.09 |
| 4. | Specific gravity | 2.85 |
| 5. | Porosity, per cent | 1.82 |

### 4.3 Point Load Strength Index

Point load strength tests were conducted on 28 samples (10 axial and 18 diametral). The point load strength values are given in Table 3. In case of diametrical tests, the loading direction was kept perpendicular and parallel to the schistosity. Whereas, in the case of axial tests the angle of loading direction with schistosity varied from 50$^o$ to 90$^o$.

Table 3 – Point load strength ($I_p$) test results

| Sl.No. | Axial Test, $I_p$, (MPa) | Diametral Test $I_p$, (MPa) | |
|---|---|---|---|
| | | Perpendicular to Schistosity | Parallel to Schistosity |
| 1. | 1.95 | 3.01 | 0.18 |
| 2. | 1.89 | 0.22 | 0.04 |
| 3. | 2.20 | 1.79 | 0.30 |
| 4. | 1.72 | 0.52 | 1.31 |
| 5. | 1.06 | 0.87 | 0.92 |
| 6. | 0.79 | 2.44 | 1.53 |
| 7. | 0.71 | 5.68 | 0.18 |
| 8. | 0.83 | 6.77 | 0.22 |
| 9. | 0.81 | 2.62 | 0.26 |
| 10. | 2.66 | - | - |
| Mean | 1.55 | 2.66 | 0.55 |
| Std. Dev. | 0.35 | 2.12 | 0.52 |

Results show that variation in the case of axial tests is the least, though in some cases alignment of axial load was not perfectly perpendicular to schistosity. In the case of diametral tests there are large variations which may give misleading interpretation. However, the mean values of diametral tests in both alignments of loading have been used to define the anisotropy of rock mass which is taken as the ratio of point load strength index parallel to that of perpendicular to the plane of schistosity (Manual on Rock Mechanics, 1988).

Anisotropy Index, $I_a$ = 0.21

### 4.4 Brazilian Tensile Strength

The tensile strength of the phyllites was determined using Brazilian tests. The mean tensile strength is found to be 8.72 MPa with a standard deviation of 1.27.

### 4.5 Uniaxial Compressive Strength

Uniaxial compression test was conducted on dry (11 Nos.) and saturated (7 Nos.) specimens. The results are shown in Table 4. The stress strain curves indicate that phyllites behave more like a brittle material. The failure of UCS specimens occurred mainly by vertical splitting.

Table 4 – Uniaxial compressive strength test results

| Sl.No. | L/D | Dry/Saturated | β (degree) | UCS (MPa) | Young's Modulus (GPa) | |
|---|---|---|---|---|---|---|
| | | | | | $E_t$ | $E_s$ |
| 1. | 2.07 | Dry | 15 | 35.25 | 9.03 | 5.69 |
| 2. | 2.05 | -do- | 5 | 49.46 | 11.23 | 10.31 |
| 3. | 2.10 | -do- | 10 | 43.97 | 8.77 | 5.64 |
| 4. | 2.03 | -do- | 25 | 21.98 | 10.40 | 13.75 |
| 5. | 2.08 | -do- | 10 | 35.18 | 13.79 | 12.50 |
| 6. | 2.09 | -do- | 10 | 41.77 | 6.74 | 9.02 |
| 7. | 2.08 | -do- | 6 | 49.47 | - | - |
| 8. | 1.90 | -do- | 80 | 52.76 | 4.96 | 3.05 |
| 9. | 2.12 | -do- | 70 | 61.56 | 8.18 | 5.08 |
| 10. | 2.04 | -do- | 65 | 54.96 | 6.50 | 4.10 |
| 11. | 1.88 | -do- | 55 | 59.14 | 6.56 | 4.34 |
| Mean | - | - | - | 45.73 | 7.35 | 7.35 |
| Standard Deviation | - | - | - | 11.50 | 2.51 | 3.58 |
| 12. | 2.08 | Saturated | 20 | 28.38 | 5.95 | 4.91 |
| 13. | 2.13 | -do- | 25 | 25.28 | 5.06 | 3.33 |
| 14. | 2.13 | -do- | 25 | 24.70 | 5.29 | 3.07 |
| 15. | 2.10 | -do- | 10 | 34.93 | - | - |
| 16. | 2.02 | -do- | 15 | 37.38 | - | - |
| 17. | 1.76 | -do- | 60 | 46.17 | 4.97 | 4.36 |
| 18. | 1.92 | -do- | 90 | 59.36 | 6.48 | 5.51 |
| Mean | - | - | - | 36.60 | 5.55 | 4.24 |
| Standard Deviation | - | - | - | 11.65 | 0.58 | 0.93 |

The reduction in uniaxial compression strength is about 20per cent due to saturation. Similarly there is about 35per cent decrease in tangent modulus, $E_t$, determined at 50per cent of ultimate load. The values of secant modus, $E_s$, are also given in Table 4. Using uniaxial compressive strength and tangent modulus, Deere and Miller (1966) classification has been attempted to classify the rocks. These values for dry and saturated conditions are plotted in the Deere and Miller's chart (Fig. 1).

According to this classification system, most of the dry phyllites were found to be in the category of DM and CL. A few dry specimen fall in the category of EM and DL also. Saturated samples were found to be in the category of DM, DL and CL. On an average both dry and saturated rocks can be categorized as DL, i.e. low strength low modulus rocks .

### *4.5.1 Influence of schistosity (anisotropy) on uniaxial compressive strength*

Schistose rocks such as phyllites and schists, present a different behaviour from that of intact rock under uniaxial compression. Failure in such rocks can occur either along clearly defined structural features or through the intact rock pieces themselves. Where the rock pieces are small compared to the size of the structure being analysed, it is reasonable to assume that there will always be a sufficient number of critically oriented pieces in the rock mass and that failure of these pieces will occur along the schistosity (Hoek and Brown, 1997).

Further, the anisotropy of the rocks may be considered for developing an approach for predicting the squeezing potential under high overburden.

The studies conducted by various investigators indicate that compressive strength at failure is maximum at $\beta = 0^o$ or $90^o$ and is minimum when $\beta$ is around $30^o$. Considering the compressive strength at $\beta = 90^o$ as the representative strength, the anisotropy ratio, $R_c$, defined as $\sigma_{c90} / \sigma_{cmin}$ (Ramamurthy, 1993) can be calculated.

Table 5 presents a classification of rocks on the basis of inherent anisotropy. Slates and phyllites have anisotropy ratio varying from 2.0 to 6.0. In some cases it may be more than 6.0 also (Ramamurthy 1993).

Table 5 – Classification of inherent anisotropy (Ramamurthy, 1993)

| Anisotropy Ratio, $R_c$ | Class |
|---|---|
| 1.0 – 1.1 | Isotropic |
| 1.1 – 2.0 | Low anisotropy |
| 2.0 – 4.0 | Medium anisotropy |
| 4.0 – 6.0 | High anisotropy |
| > 6.0 | Very high anisotropy |

The anisotropy ratio, $R_c$, for the saturated and dry Tehri phyllites ranges between 2.4 and 2.7. Hence the phyllites may be said to be possessing medium anisotropy as per Table 5.
According to Ramamurthy (1993), slates, phyllites and similar rocks, depicts U type of anisotropy represented as the variation of compressive strength with orientation angle ($\beta$). The shape of the anisotropic curve throughout the range of orientation angle can be

predicted by Eq. 1 if the compressive strength values at three orientations of $0^o$, $30^o$ and $90^o$ are known.

$$\sigma_c = A - D [\cos 2 (\beta_m - \beta)] \qquad (1)$$

where $\sigma_c$ = uniaxial compressive strength at orientation angle, $\beta$
A,D = constants
$\beta_m$ = schistosity angle for which the strength is minimum

The orientation angle $\beta$ is measured from the direction of loading.

The experimental data points and curve from Eq. 1 are shown in Figs. 2 and 3 for dry and saturated specimens respectively. It is clear that most of the experimental points lie on or around the predicted curves. Hence Eq. 1 may be used in case of Tehri phyllites also.

### 4.5.2 Long-Term Strength

Long-term strength, also called creep limit or creep threshold, can be defined as the maximum stress sustained by the rock below which failure will not occur, no matter how long the force has been applied.

To understand the microscopic fracturing phenomenon and to determine the long-term strength two typical stress-strain curves one for dry samples and other for saturated samples were plotted on log-log graph (Figs. 4 and 5). Two kinks in the curve are seen. In the case of dry sample (Fig. 4), the first kink which may be viewed as representing closure of existing cracks takes place at about 29 percent of failure strain and unstable crack propagation starts at about 71 percent of failure strain. The second kink shows that corresponding long-term strength is about 93 percent of ultimate short- term strength.

In the case of saturated sample (Fig. 5), the two kinks are at about 54 per cent and 77 per cent respectively and the long-term strength is about 70 per cent of the ultimate strength. Here, like ultimate strength, long-term strength is also reduced due to saturation. Delay in the two kinks may be due to hydrodynamic lag and viscous lag of water present in fine cracks.

### 4.6 Triaxial Compressive Strength

Triaxial compression tests were conducted to obtain shear strength behaviour of phyllites in triaxial stress conditions. Based on the test results, applicability of different strength criteria have been verified in order to identify a criterion which facilitate the strength prediction for phyllites.

A summary of triaxial strength values are presented in Tables 6 and 7 for dry and saturated conditions respectively. It may be pointed out here that the tests conducted at the same $\sigma_3$ gave different $\sigma_1$ which is attributed to the inherent anisotropy (schistocity) of the phyllites. It is therefore suggested that average strength parameters be obtained by thorough statistical investigation of the test data in the case of anisotropic cracks.

It is not very surprising to observe a scatter in the test data even in the isotropic rocks due to presence of microcracks within the intact rock specimens.

Table 6 – Triaxial compressive strength test data-dry condition

| Sl. No. | $\sigma_3$ (MPa) | $\sigma_1$ (MPa) |
|---|---|---|
| 1. | 15.0 | 76.13 |
| 2. | 15.0 | 128.52 |
| 3. | 20.0 | 137.89 |
| 4. | 20.0 | 142.26 |
| 5. | 20.0 | 146.62 |
| 6. | 20.0 | 97.50 |
| 7. | 22.5 | 103.28 |
| 8. | 25.0 | 169.09 |
| 9. | 25.0 | 158.32 |
| 10. | 30.0 | 167.54 |
| 11. | 30.0 | 200.29 |
| 12. | 35.0 | 159.44 |
| 13. | 35.0 | 178.10 |
| 14. | 35.0 | 167.72 |
| 15. | 40.0 | 198.20 |
| 16. | 45.0 | 181.02 |
| 17. | 45.0 | 188.10 |
| 18. | 50.0 | 221.38 |

#### *4.6.1 Influence of confining pressure*

Figures 6 and 7 show the variation of axial stress, $\sigma_1$ with confining pressure, $\sigma_3$. It is obvious that failure strength increases with confining pressure. Mogi's line ($\sigma_1 = 3.4\ \sigma_3$; Mogi, 1965) is also drawn for both dry and saturated cases. It is revealed that in the dry case failure changes from brittle to ductile at about 45 MPa confining pressure. This transition is obtained at about 22.5 MPa of confining pressure in the saturated case.

It is obvious that due to non-linearity of $\sigma_1$ versus $\sigma_3$ curve, the unconfined compressive strength is apparently increased. This strength enhancement is thrice and twice of UCS in dry and saturated conditions respectively.

It has been found that in both the cases of dry and saturated conditions, axial strain at failure increases with confining pressure.

### *4.6.2 Shear strength parameters*

Plot of $p = [(\sigma_1 + \sigma_3)/2]$ versus $q = [(\sigma_1 - \sigma_3)/2]$ for dry and saturated conditions are shown in Figs. 8 and 9. The average values of cohesion, c and angle of shearing resistance, $\phi$ obtained for the best fit straight lines are found to be 12.2 MPa and $35^{o}$ respectively for dry case and 7.7 MPa and $28^{o}$ respectively for saturated case.

Table 7 – Triaxial compressive strength test data-saturated condition

| Sl. No. | $\sigma_3$ (MPa) | $\sigma_1$ (MPa) |
|---|---|---|
| 1. | 10.0 | 55.85 |
| 2. | 11.0 | 56.85 |
| 3. | 12.5 | 56.16 |
| 4. | 15.0 | 106.70 |
| 5. | 15.0 | 88.80 |
| 6. | 15.5 | 85.36 |
| 7. | 17.5 | 67.71 |
| 8. | 17.5 | 61.16 |
| 9. | 20.0 | 76.76 |
| 10. | 20.0 | 102.26 |
| 11. | 22.5 | 87.99 |
| 12. | 22.5 | 92.00 |
| 13. | 25.0 | 121.06 |
| 14. | 25.0 | 110.14 |
| 15. | 27.5 | 84.26 |
| 16. | 27.5 | 90.81 |
| 17. | 27.5 | 92.99 |
| 18. | 27.5 | 125.74 |
| 19. | 30.0 | 117.33 |
| 20. | 30.0 | 124.00 |
| 21. | 35.0 | 100.49 |
| 22. | 40.0 | 154.19 |
| 23. | 45.0 | 131.94 |
| 24. | 45.0 | 93.65 |
| 25. | 50.0 | 176.26 |

### *4.6.3 Effect of confining pressure on shear strength parameters*

Figures 10 and 11 show Mohr's envelopes for dry and saturated cases. It is obvious that in both the cases the Mohr's envelopes are nonlinear. Due to the non-linearity, with increase in normal stress cohesion increases whereas friction angle decreases in both the cases. These variations are presented in Figs. 12 and 13.

## 4.7 Failure Criteria for Phyllites

A number of failure criteria are available in the literature. However generalized Hoek and Brown (1997) and Ramamurthy (1985) IIT Delhi criterion have been applied in the present study.

### *4.7.1 Generalization Hoek-Brown criterion for rock masses*

Generalized Hoek-Brown criterion (Hoek et al., 1992) covers the complete range of rock mass and has been found to work well in practice. The generalized Hoek-Brown failure criterion is given by

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left[m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right]^a \tag{2}$$

where, $m_b$ is the value of the Hoek-Brown constant (m) for the rock mass, and $s$ and $a$ are the constants which depend upon the characteristics of rock mass.

In order to use the Hoek-Brown criterion for jointed rock masses, three properties of rock mass have to be estimated. These are

(i) The uniaxial compressive strength, $\sigma_{ci}$ of the intact rock pieces
(ii) The value of the Hoek-Brown constant, $m_i$ for intact rock and
(iii) The value of the Geological Strength Index, GSI for rock mass.

Wherever possible the value of $\sigma_{ci}$ and $m_i$ should be determined by statistical analysis of the results of a set of triaxial tests on carefully prepared core samples. Hoek and Brown (1980) used a range of 0 to 0.5 $\sigma_{ci}$ of minor principal stress and in order to be consistent, it is essential that the same range be used in any laboratory triaxial tests on intact rock specimens.

The GSI, introduced by Hoek (1994) and Hoek, Kaiser and Bawden (1995) provides a system for estimating the reduction in rock mass strength for different geological conditions.

To derive generalized Hoek-Brown criterion for phyllites, triaxial test data with $0 < \sigma_3 < 0.5\ \sigma_c$ were used to determine material constant, $\sigma_{ci}$ and $m_i$ as per the method suggested by Hoek and Brown (1997). The results have been summarized in Table 8.

Table 8 – Constant of generalized Hoek-Brown criterion

| Condition | $\sigma_{ci}$ | $m_I$ | $r^2$ | GSI | $m_b$ | a | s |
|---|---|---|---|---|---|---|---|
| Dry | 48.64 | 10.94 | .993 | 70 | 3.747 | 0.5 | 0.0356 |
| Saturated | 41.58 | 3.45 | 0.60 | 70 | 1.181 | 0.5 | 0.0356 |

For dry condition, the failure criterion for phyllites can be written as,

$$\sigma_1 = \sigma_3 + 48.64\,(0.225\;\sigma_3 + 0.0356)^{0.5} \tag{3}$$

and for saturated condition

$$\sigma_1 = \sigma_3 + 41.58\,(0.083\;\sigma_3 + 0.0356)^{0.5} \tag{4}$$

The test results and proposed criterion are shown in Figs. 14 and 15 for dry and saturated conditions respectively.

In the determination of material constant, $\sigma_{ci}$ and $m_i$, coefficient of determination, $r^2$ was found to be 0.99 for dry condition, whereas it was 0.60 for saturated condition. Hence applicability of generalized Hoek-Brown criterion is more suited to dry phyllites. In the case of saturated condition, it underestimates the strength.

### *4.7.2 Ramamurthy criterion*

Mohr-Coulomb theory was modified by Ramamurthy (1985) to represent the nonlinear strength criterion for intact rock in the form

$$\frac{(\sigma_1 - \sigma_3)}{\sigma_3} = B_i \left(\frac{\sigma_c}{\sigma_3}\right)^{\alpha_i} \tag{5}$$

where, $B_i$ and $\alpha_i$ are material constants.

The value of $\alpha_i$ and $B_i$ can be estimated by conducting a minimum of two triaxial tests at confining pressure greater than 5 per cent of $\sigma_c$ for the rock. Above expression is applicable in the ductile range and in most of the brittle region.

For anisotropic and jointed rocks, the strength criterion may be represented by

$$\frac{(\sigma_1 - \sigma_3)}{\sigma_3} = B_j \left(\frac{\sigma_{cj}}{\sigma_3}\right)^{\alpha_j} \tag{6}$$

where, $\sigma_{cj}$ is uniaxial compressive strength at any orientation, and $\alpha_j$ and $B_j$ are the values of $\alpha$ and B at the orientation under consideration. These parameters, $\alpha_j$ and $B_j$ can be determined from the equations proposed by Ramamurthy (1985).

Since samples used in triaxial testing were having the different $\beta$, hence it was difficult to define $\alpha_j$ and $B_j$. Therefore, strength criterion for intact rock itself was fitted. Figures 16 and 17 show the plot for calculation of $\alpha$ and B. Summary of the values of constants are given in Table 9.

Table 9 – Constants of Ramamurthy (1985) criterion

| Condition | $\alpha$ | B |
|---|---|---|
| Dry | 0.75 | 3.207 |
| Saturated | 0.65 | 2.128 |

Figures 18 and 19 show the test results and a line corresponding to Ramamurthy's criterion. It is obvious that Ramamurthy's criterion overestimates the strength in lower confining pressure range (below 25 MPa) in case of dry condition. In case of saturated condition, on the other hand, the Ramamurthy's criterion line is passing in between the test data (Fig. 19) indicating that it predicts the strength much accurately for almost all the confining pressure range.

## 5. CONCLUSIONS

Based upon the experimental data and subsequent interpretation, the following conclusions may be drawn in case of phyllites.

1. Phyllites are medium to fine-grained and grey-coloured rock. Thin section studies show that rock contains about 50 per cent of biotite, 25 per cent of quartz and 15 per cent of felspar as major constituents.
2. The unconfined compressive strength (UCS) varies from 21.98 to 69.59 MPa and due to saturation, strength reduces by about 20 per cent. Average modulus of elasticity at 50 per cent of ultimate strength under uniaxial compression was found to be 8.6 and 5.5 GPa for dry and saturated rock respectively.
3. Under uniaxial compression, specimens failed by vertical splitting indicating brittle type of failure.
4. According to Deere and Miller's classification, phyllites fall in the category DL and DM where D stands for low strength, L for low modulus ratio and M for medium modulus ratio.
5. In case of dry rock specimens, long-term strength was found to be about 90 per cent of ultimate strength. For the saturated rock, it was found to be about 70 per cent of its ultimate strength.
6. Phyllites possess U-type of anisotropy with minimum strength obtained at schistosity angle of about $30^{o}$. The anisotropy ratio varies between 2.4 and 2.7 indicating medium anisotropic rocks.
7. The generalized Hoek-Brown criterion suits well for the prediction of strength of dry phyllites, whereas in saturated condition it underestimates the strength of rock.
8. Ramamurthy's criterion compares well with experimental results for saturated condition. But in the case of dry condition, it overestimates the strength in lower confining pressure range.

### *References*

Deere, D.U. and Miller, R. (1966). Engineering Classification and Index Properties for Intact Rock, Tech. Report, A.F.W. Lab., New Mexico.

Hoek, E. (1994). Strength of Rock and Rock Masses, ISRM, News Journal, 2 (2), pp. 4-16.

Hoek, E. and Brown, E.T. (1980). Underground Excavations in Rock, London, Instn. Min. Metall., p. 527.

Hoek, E. and Brown, E.T. (1997). Practical Estimates of Rock Mass Strength, Vol. 14, No. 8, Int. J. of Rock Mech. Min. Sc. and Geomech. Abstract, pp. 1165-1186.

Hoek, E., Kaiser, P.K., and Bawden, W.F. (1995). Support of Underground Excavation in Hard Rock, Rotterdam Balkema.

Hoek, E., Wood, D. and Shah, S. (1992). A Modified Hoek-Brown Failure Criterion for Jointed Rock Masses, Intl Soc. Rock Mech. Symp., Eurock (Ed. Hudson), Brit. Geol. Soc., London, pp. 209-214.

IS:8764-1978. Method of Determination of Point Load Strength Index of Rocks, pp. 3-5.

IS:9143-1979. Method of Determination of Unconfined Strength of Rock Material, pp. 3-6.

IS:9179-1979. Method for Preparation of Rock Specimen for Laboratory Testing, pp. 3-6.

IS:10082-1981. Method of Test for Determination of Tensile Strength by Indirect Tests on Rock Materials, pp. 3-12.

IS:13030-1991. Method of Test for Laboratory Determination of Water Content, Porosity, Density and Related Properties of Rock Material, pp. 3-7.

IS:13047-1991. Method for Determination of Strength of Rock Material in Triaxial Compression, pp. 3-6.

I.S.R.M. (1981).Basic Technical Properties of Rock Masses, Int. J. Rock Mech. Min. Sci., Vol. 18, pp. 85-110.

Manual on Rock Mechanics (1988).Central Soil and Material Research Station, CBIP, New Delhi.

Mogi, K. (1965).Deformation and Fracture of Rock Under Confining Pressure, Bulletin of the Earthquake Research Inst., Vol. 43, pp. 349-379.

Prasad, M.M. (1998).Engineering Classification and Properties of Phyllite, M.E. Dissertation, Dept. of Civil Engg., University of Roorkee, Roorkee, India.

Ramamurthy, T. (1985).Stability of Rock Mass, Eight IGS Annual Lecture, University of Roorkee, India, pp. 1-74.

Ramamurthy, T. (1993).Strength and Modulus Responses of Anisotropic Rocks, Comprehensive Rock Engg. (Ed. Hudson), Pergamon Press, Vol. I, Chap. 13, pp. 313-319.

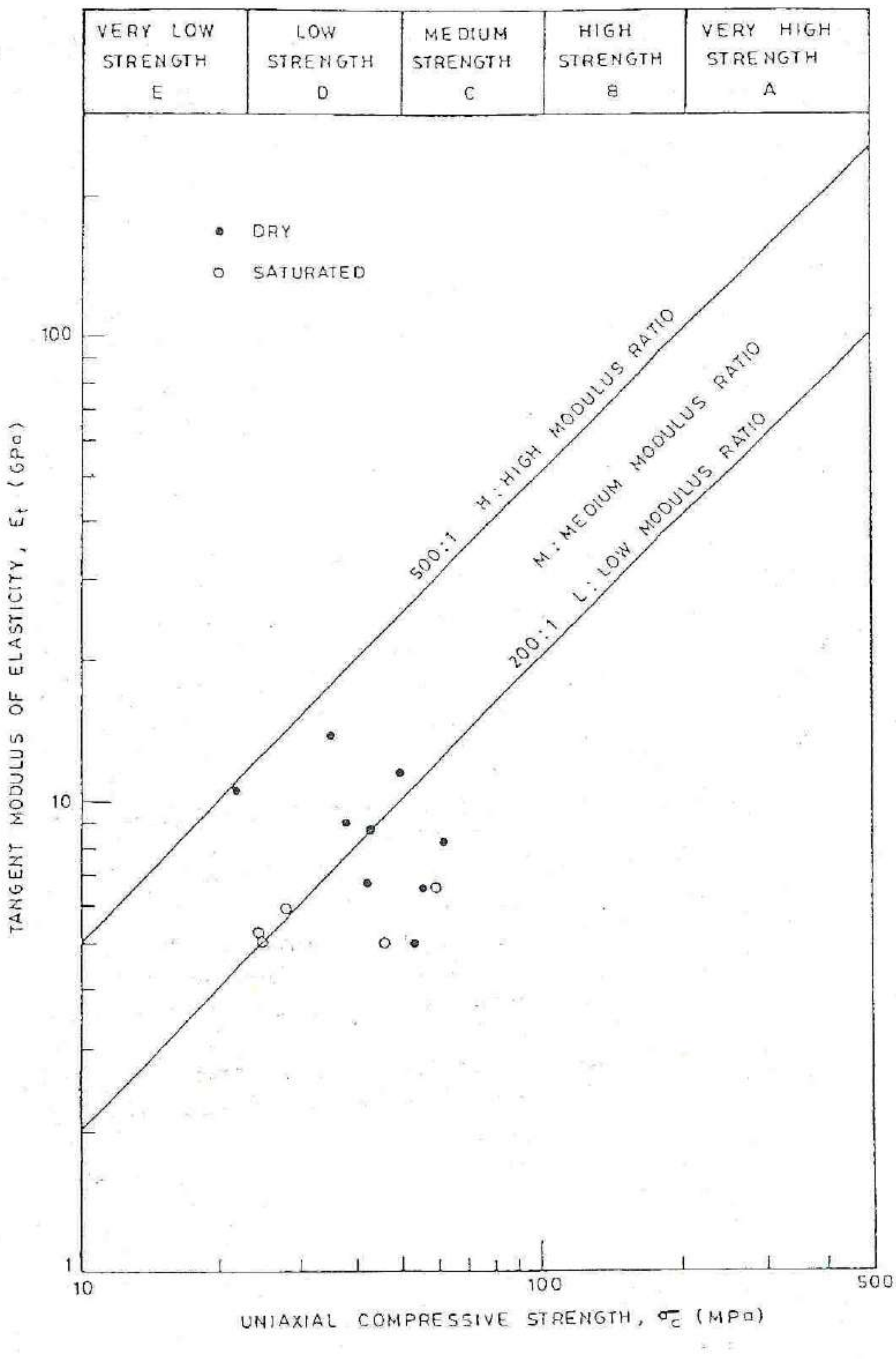


Fig. 1 – Engineering classification of phyllites (Deere and Miller, 1966)

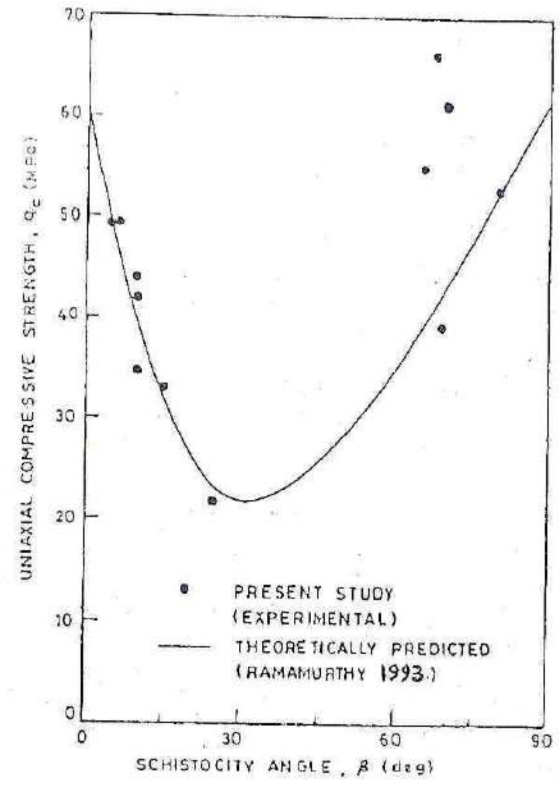


Fig. 2 – Variation in uniaxial compressive strength with schistosity – dry case

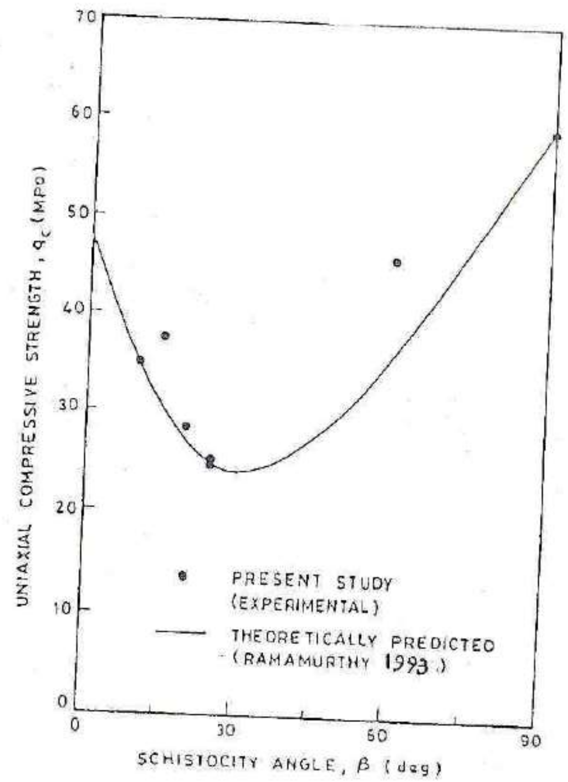


Fig. 3 - Variation in uniaxial compressive strength with schistosity – saturated case

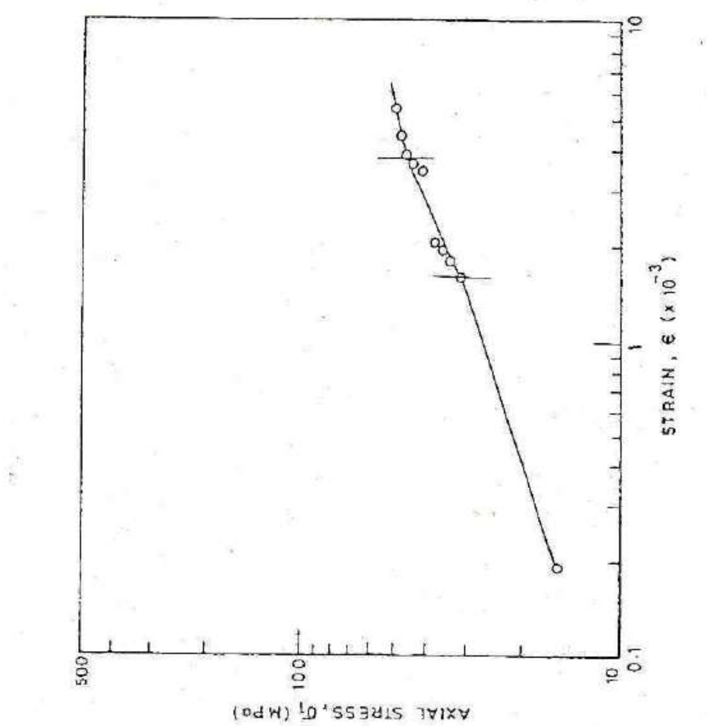


Fig. 4 – Log-log plot of stress-strain curve-dry case

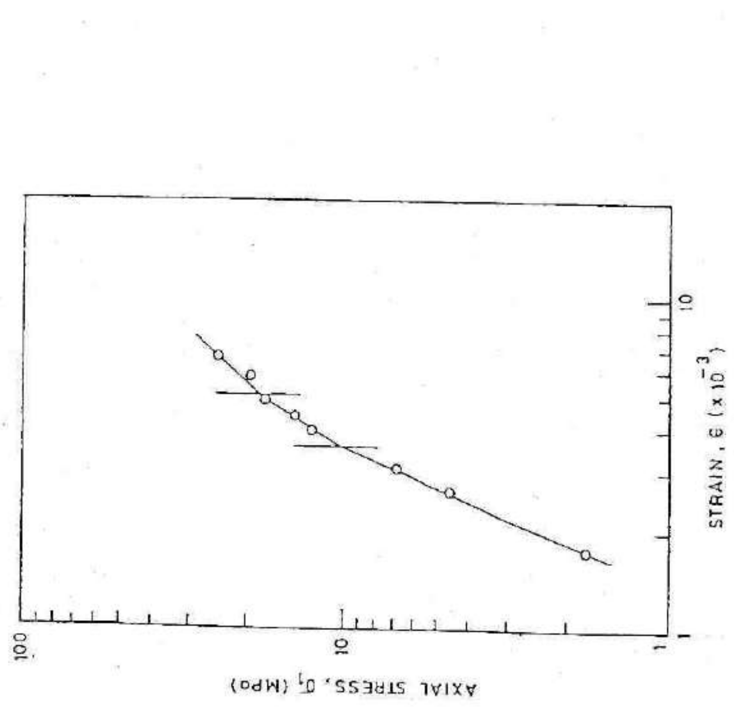


Fig. 5 – Log-log plot of stress-strain curve – saturated case

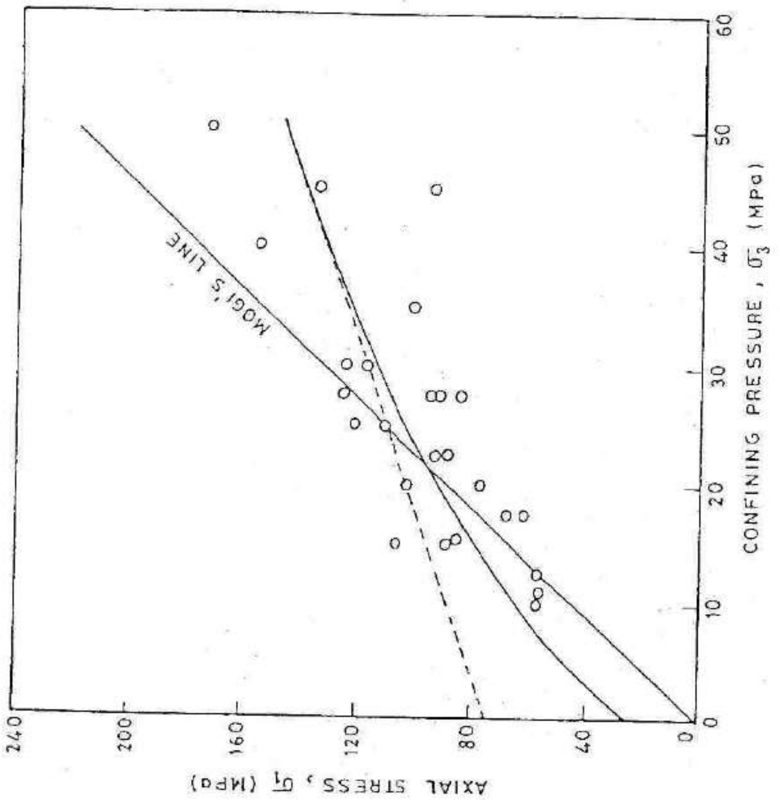


Fig. 7 – Variation of axial stress with confining pressure-saturated case

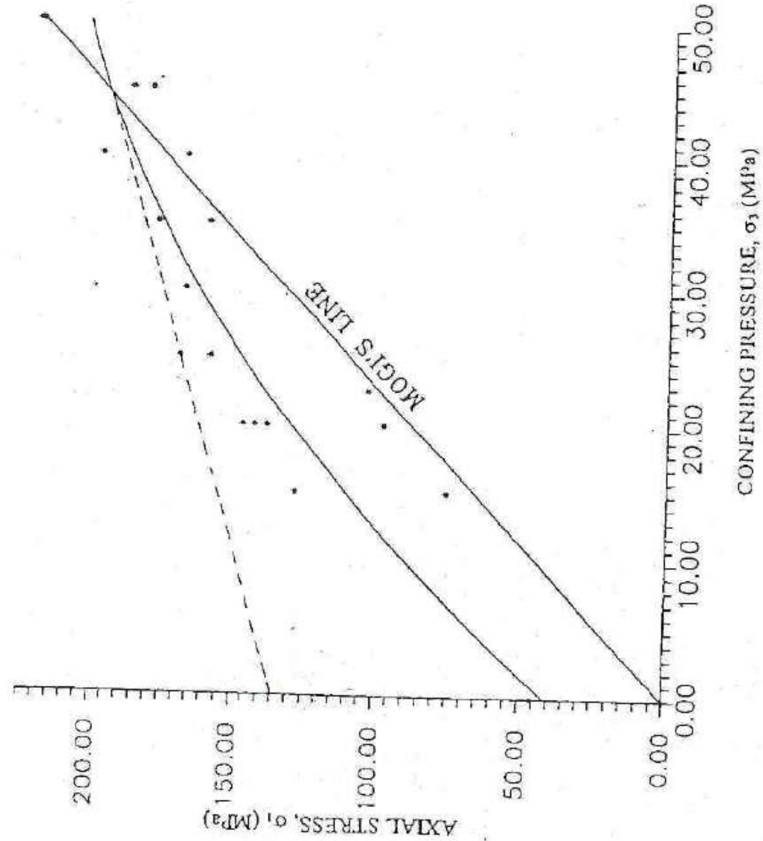


Fig. 6 – Variation of axial stress with confined pressure-dry case

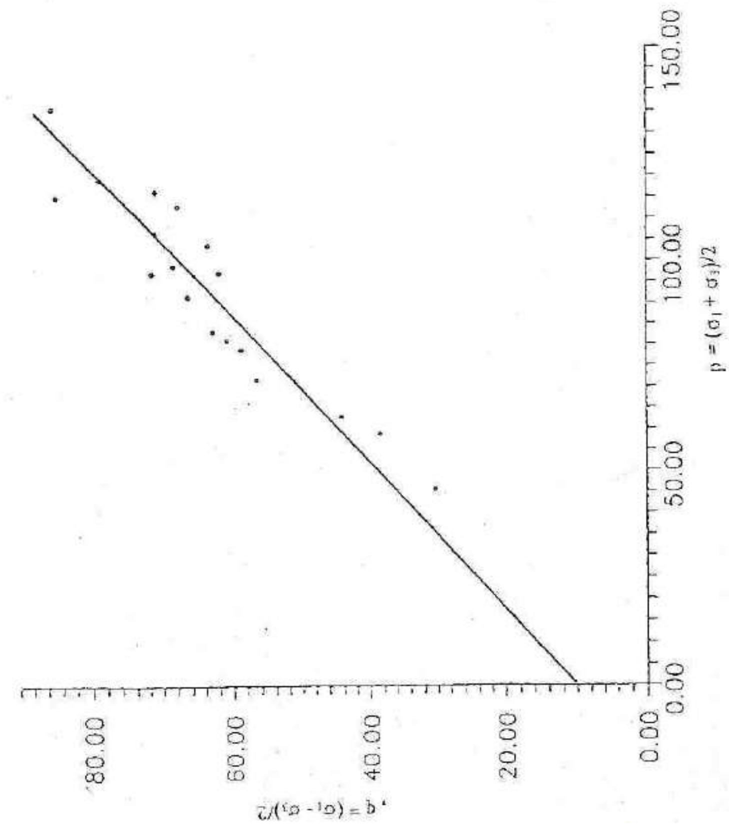


Fig. 8 – p-q plot for phyllites – dry case

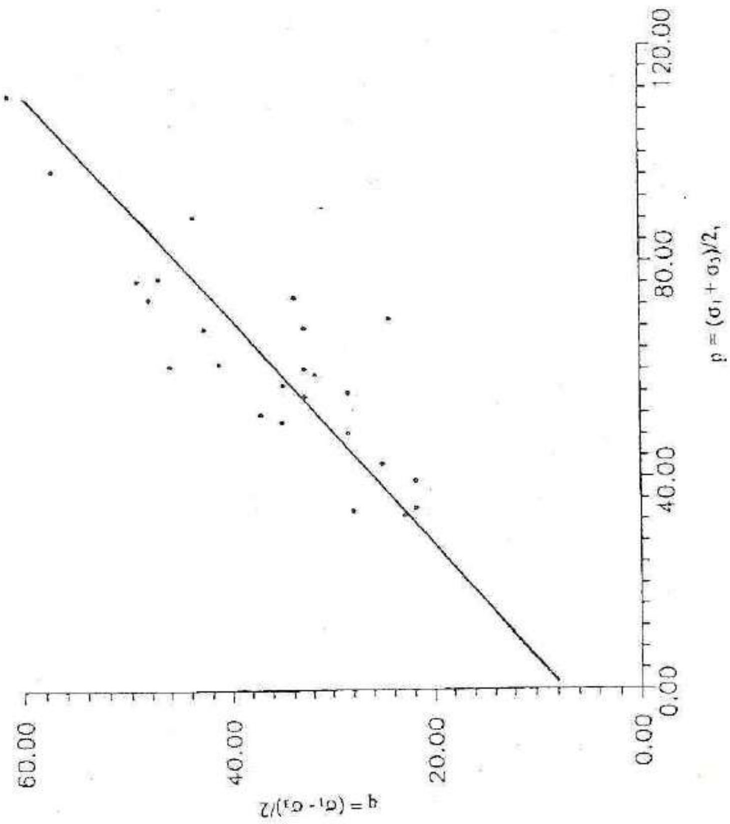


Fig. 9 – p-q plot for phyllites saturated case

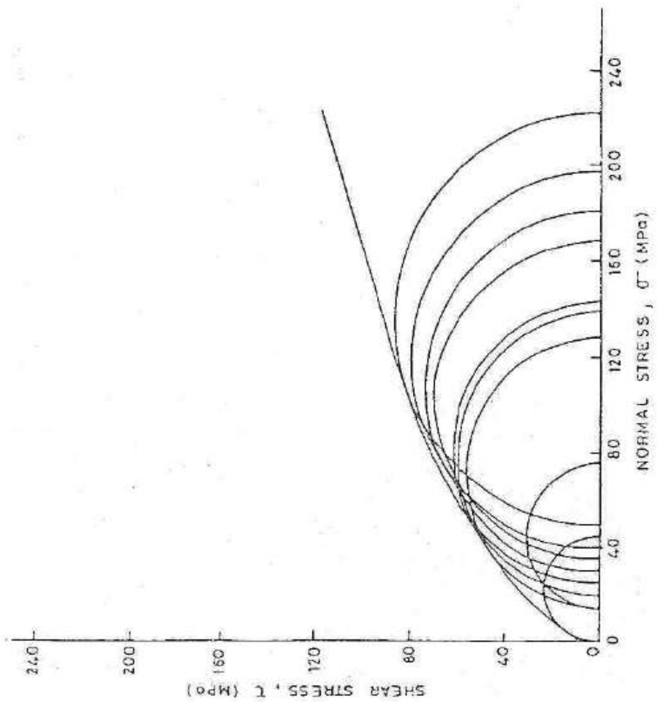


Fig. 10 – Mohr's envelope-dry case

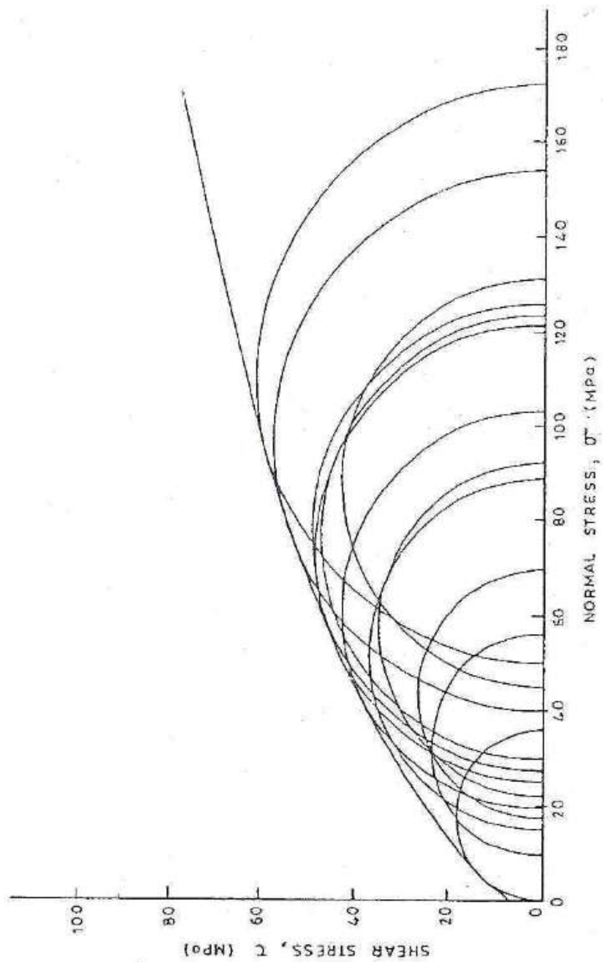


Fig. 11 – Mohr's envelope-saturated case

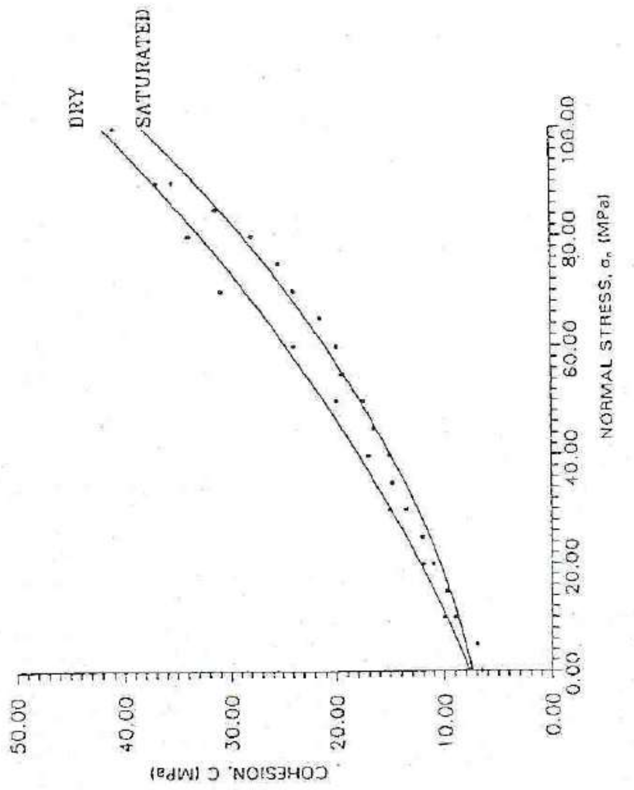


Fig. 12 – Variation in cohesion with normal stress

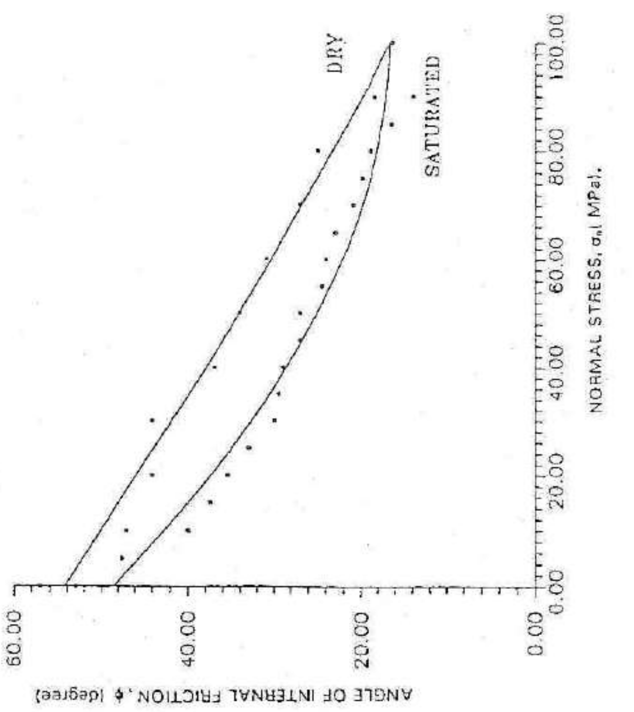


Fig. 13 – Variation in angle of internal friction with normal stress

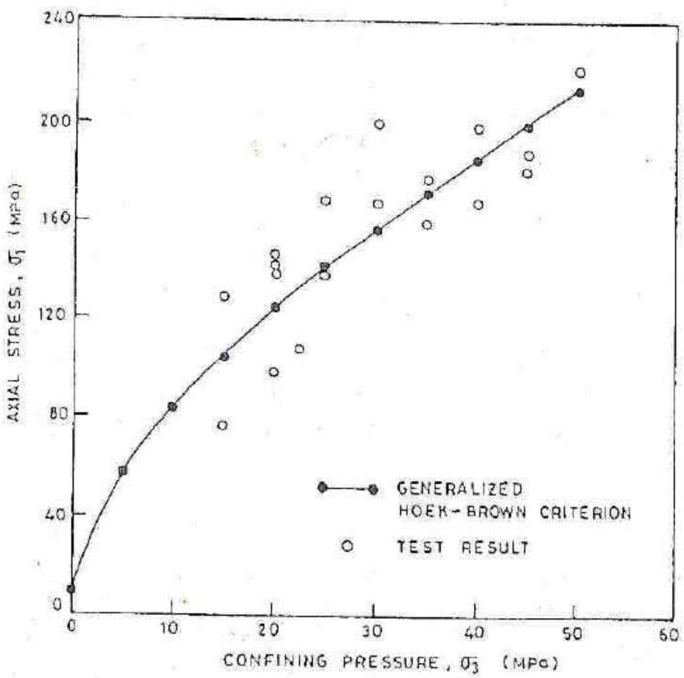


Fig. 14 – Generalized Hoek-Brown criterion and test results – dry case

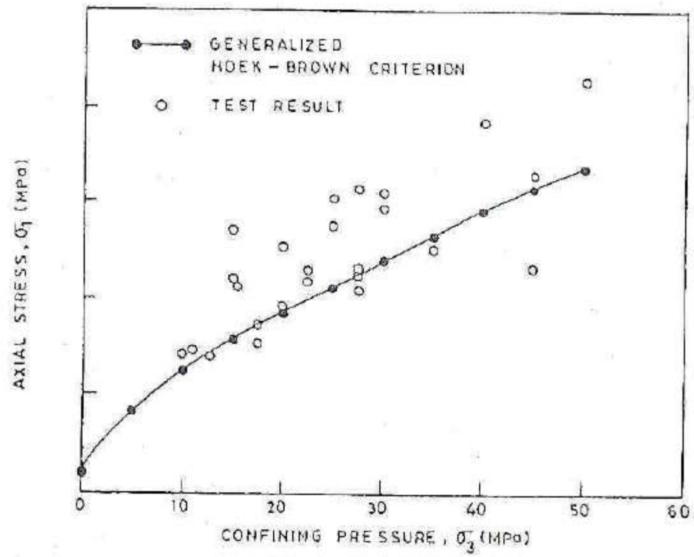


Fig. 15 – Generalized Hoek-Brown criterion and test results – saturated case

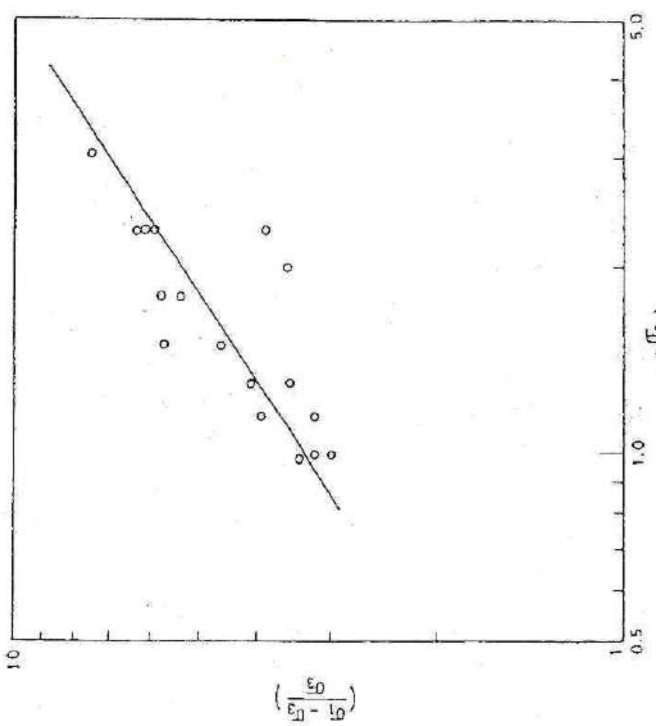


Fig. 16 – Log-log plot between $\left(\frac{\sigma_1-\sigma_3}{\sigma_3}\right)$ and $\left(\frac{\sigma_c}{\sigma_3}\right)$ - dry case

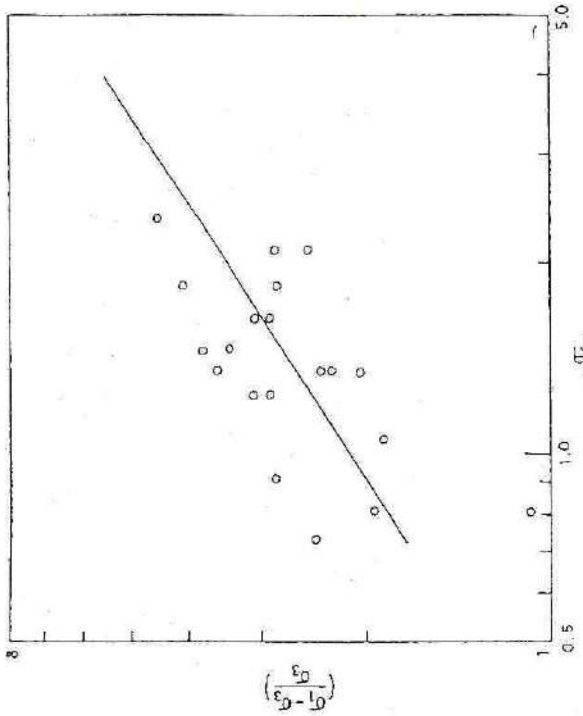


Fig. 17 – Log-log plot between $\left(\frac{\sigma_1-\sigma_3}{\sigma_3}\right)$ and $\left(\frac{\sigma_c}{\sigma_3}\right)$ - saturated case

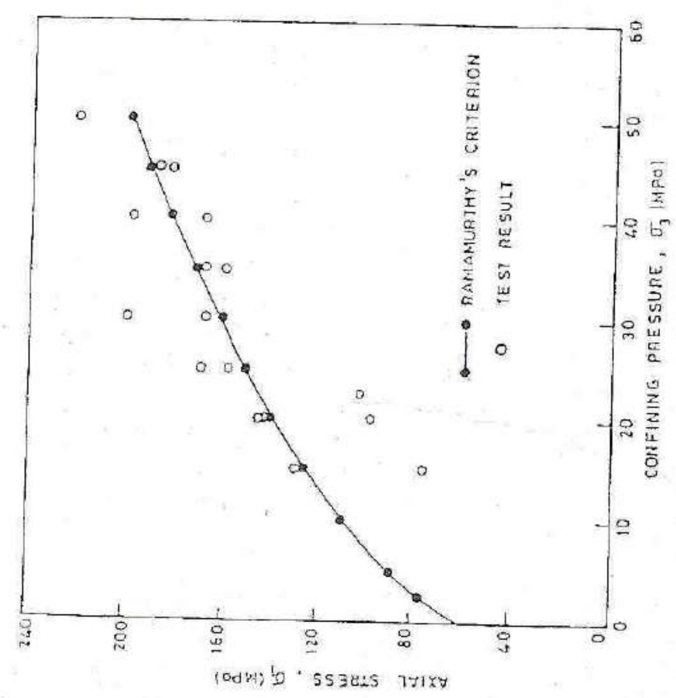


Fig. 18 – Ramamurthy's criterion and test results - dry case

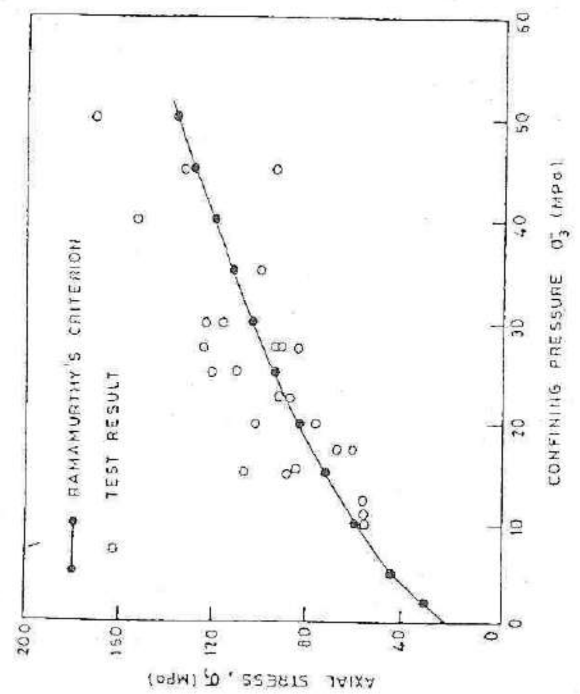


Fig. 19 – Ramamurthy's criterion and test results - saturated case