



Shear Strength Assessment of Rockfill Materials through JRC-JCS Model and Critical State Concept

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ABSTRACT

Rockfills are granular materials whose shear strength behaviour is highly non-linear and strongly dependent on normal stress or confining pressure at which the material is tested. Conducting field tests to capture this non-linear, stress-dependent shear strength behaviour is challenging. This article proposes that the non-linear, stress-dependent friction angle of rockfills can be approximated by using shear strength models originally developed for rock joints and jointed rocks. Two models, namely Barton's JRC-JCS shear strength model, and the Modified Mohr Coulomb (MMC) criterion for jointed rocks, are considered in the present article. Based on extensive laboratory testing of rock-soil mixtures, empirical correlations are proposed to estimate the shear strength response of rockfills through simple laboratory and field measurements. These correlations are expected to be very helpful in analysis and design of embankments and dams.

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Keywords: Friction angle; Non-linear shear strength; JRC-JCS model; MMC criterion.

1. INTRODUCTION

With ever-increasing demand of electricity there is great emphasis on production of clean energy in India. In recent years, Pumped Storage Projects (PSP) have emerged as one of the most promising options for augmenting power generation. Currently ten number of PSPs with installed capacity of 6445.60 MW are in operation in the country, and an additional 82660 MW is planned to be added by the year 2035–36 (CEA, 2025). At many project sites, the rockfill embankments are employed to create water reservoirs. For successful analysis and design of rockfill dams and embankments as water- retaining structures, a sound understanding of the shear strength behaviour of the rockfill materials at the site is crucial. While materials such as sand, silt and clay can be tested in the field or laboratory with high confidence, testing rockfill is extremely difficult, time consuming, costly and in many cases, not feasible. Indirect approaches are, therefore, preferred to evaluate the shear strength of rockfill materials.

Rock Mechanics is a relatively young discipline within geotechnical engineering that focuses on the engineering behavior of intact rocks, rock joints, and jointed rock masses. Over the past few decades, substantial research has been carried out on the strength behavior of rock joints and jointed rock masses. The present article discusses how some of these approaches can be applied to assess the shear strength of rockfill materials.

2. SHEAR STRENGTH OF ROCKFILLS

Rockfills are generally considered cohesionless materials, with their strength primarily governed by particle friction arising from interlocking between particles. The shear strength is commonly expressed in terms of the peak friction angle, while cohesion between particles is assumed to be negligible. Since frictional strength is largely a result of interlocking, the peak friction angle exhibits pronounced nonlinearity with respect to stress levels, particularly under low normal stresses. This strong nonlinear behavior, coupled with the high dependency of the friction angle on confining pressure, necessitates careful consideration of all factors influencing strength behavior when determining the peak friction angle. The peak friction angle of rockfills has been shown to be influenced mainly by the following factors:

- i. Stress level i.e., normal stress level or confining pressure,
- ii. Density/ relative density,
- iii. Particle size, shape, and gradation,
- iv. Proportion of gravel content, and
- v. Particle strength.

The confining pressure or the normal stress level on failure plane is one of the most critical parameters governing the peak friction angle of rockfills. The shear strength behavior has been found to be highly non-linear (Marshal, 1973; Cerato and Lutenecker, 2006; Asadzadeh and Soroush, 2009; Lee et al., 2009). Particle angularity also plays a significant role in determining the peak friction angle. Angular quarried materials tend to exhibit very high internal friction angles at low confining stress levels, owing to their rough surfaces and strong interlocking. At this stage, high dilation is commonly observed. With increasing confining stress, rockfills show pronounced curvature in their strength envelopes due to particle crushing, asperity breakage, and suppression of dilation. As a result of this non-linear shear strength behavior, the peak friction angle decreases with increasing normal stress (Fig. 1).

Relative density and density of rock fill affect the peak friction angle in the sense that higher relative density exhibits higher friction angle. In field, the desired density can be achieved through several measures like proper gradation, placement and compaction techniques.

Particle strongly affects shear strength of rockfills. A number of researches (Holtz and Gibbs, 1956; Holtz and Willard, 1956; Patwardhan et al., 1970; Fedorov and Sergevnina, 1973; Donaghe and Torrey, 1979; Iannacchione, 1997; Iannacchione and Vallejo, 2000; Vallejo and Mawby, 2000; Yagiz, 2001; Kokusho and Hiraoka, 2004; Liu et al., 2006; Simoni and Houlsby, 2006; Yazdanjou et al., 2008; Hamidi et al., 2009; Coli et al., 2011; Xu et al., 2011; Li, 2013; Li et al., 2013; Vallejo

et al., 2014) have studied influence of proportion of large sized particles on shear strength of rock soil mixtures. In general, an increasing trend of peak friction angle with increasing gravel content has been observed. A few studies reported the reverse trend too.

Angularity in shape of particles increases interlocking between particles thereby increasing frictional resistance. On the other hand, rounded particles easily rotate without offering much frictional resistance and hence exhibit lower value of peak friction angle.

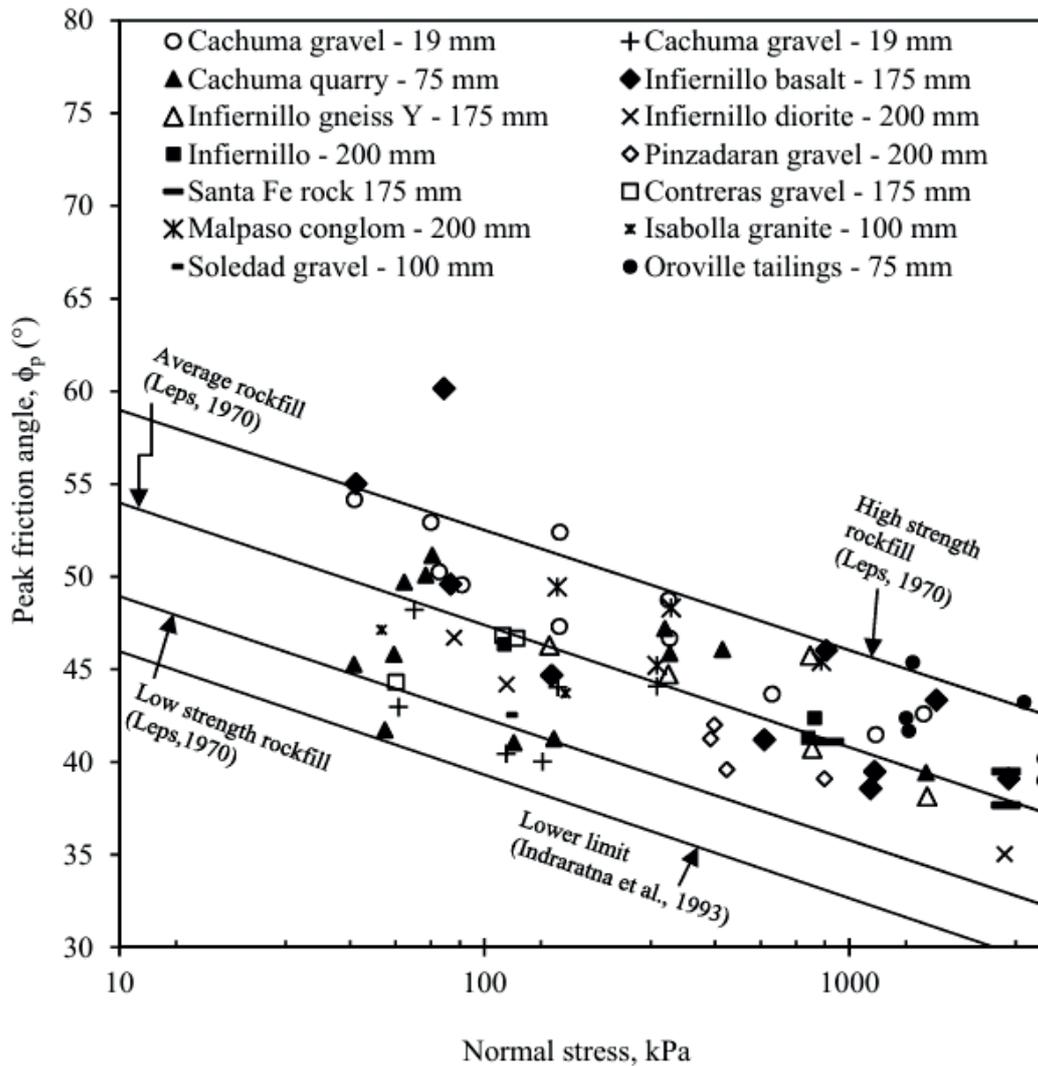


Figure 1 - Variation of peak friction angle of rockfills with normal stress (Indraratna, 1994)

Gradation of particles directly affects the density and hence influences the shear strength of granular materials. However, a mixed observation is made on the effect of gradation on shear strength of rockfills. In general, it is expected that well graded materials compacted to their maximum density are likely to give higher shear strength than poorly graded materials.

The strength of the parent rock, from which the rockfill material originates, also plays a significant role in determining the shear strength of rockfills, especially under higher stress levels. Rock

particles with greater strength tend to experience less breakage and exhibit higher dilation, leading to increased shear strength (Vardarajan et al., 2003).

In this article, two important shear strength models from Rock Mechanics are used to evaluate the peak friction angle of rockfills. These models consider all the major factors that influence the peak friction angle of rockfills. Laboratory tests were conducted using large, medium, and conventional direct shear test apparatuses on rock-soil mixtures. The study extends the applicability of shear strength models from Rock Mechanics to rockfills.

3. ROCK MECHANICS MODELS CONSIDERED IN PRESENT STUDY

The shear strength behaviour of rock joints, jointed rock masses, and rockfill materials share many common characteristics. As a result, widely used shear strength models developed for rock joints and jointed rocks are often applicable to rockfill materials as well. In the present study, two models, namely Barton’s JRC-JCS model (Barton, 1973; Barton and Choubey, 1977) and the Modified Mohr-Coulomb (MMC) criterion (Singh and Singh, 2012), are considered to evaluate their effectiveness in predicting the peak friction angle of rockfills.

3.1 Barton’s JRC-JCS Model

Barton’s JRC-JCS model (Barton, 1973; Barton and Choubey, 1977) for rock joints is the most widely used empirical criterion to predict the shear strength of rough rock joints. The model is very simple, and its input parameters can be easily obtained in the field without much difficulty. The model is expressed as:

$$\tau_f = \tan \left[\phi_r + \text{JRC} \log_{10} \left(\frac{\text{JCS}}{\sigma_n} \right) \right] \quad (1)$$

where

- τ_f = peak shear strength of the rock joint,
- JRC = joint roughness coefficient (geometrical component),
- JCS = is joint wall compressive strength (asperity failure component) and
- ϕ_r = is the residual friction angle of the joint.

The residual friction angle (ϕ_r) is approximately equal to the basic friction angle (ϕ_b) of the rock. The basic friction angle for different rock types can be obtained from Barton (1976).

From Barton’s model (Eq. 1) the peak friction angle of the rockfill can be obtained as:

$$\phi_p = \phi_r + \text{JRC}_{\text{eq}} \log_{10} \left(\frac{\text{JCS}_{\text{eq}}}{\sigma_n} \right) \quad (2)$$

where

- ϕ_r = peak friction angle of the rockfill,
- JRC_{eq} = equivalent JRC of the rockfill,
- JCS_{eq} = equivalent JCS of the rockfill and

σ_n = the normal stress at failure plane.

3.2 Modified Mohr Coulomb Criterion for Jointed Rocks (Singh and Singh, 2012)

A non-linear criterion, Modified Mohr Coulomb (MMC) criterion, for assessing strength of intact rocks under confined conditions, was suggested by Singh et al. (2011). The criterion was based on critical state concept of rocks suggested by Barton (1976). The critical state concept (Fig. 2) postulates “critical state for an initially intact rock is defined as the stress condition under which Mohr envelope of peak shear strength of the rocks reaches a point of zero gradient. This condition represents the maximum possible shear strength of the rock. For each rock, there will be a critical effective confining pressure above which the shear strength cannot be made to increase”. The MMC criterion (Singh et al., 2011) was expressed as:

$$(\sigma_1 - \sigma_3) = \sigma_{ci} + \frac{2 \sin \phi_{i0}}{1 - \sin \phi_{i0}} \sigma_3 - \frac{1}{\sigma_{crti}} \frac{\sin \phi_{i0}}{(1 - \sin \phi_{i0})} \sigma_3^2 \text{ for } 0 \leq \sigma_3 \leq \sigma_{crti} \quad (3)$$

where

σ_1 = major principal stress at failure,

σ_3 = major and minor principal stress at failure,

σ_{ci} = UCS of intact rock,

σ_{crti} = critical confining pressure of the intact rock, and

ϕ_{i0} = the limiting value of the friction angle obtained from triaxial strength tests conducted at very low confining pressure ($\sigma_3 \rightarrow 0$).

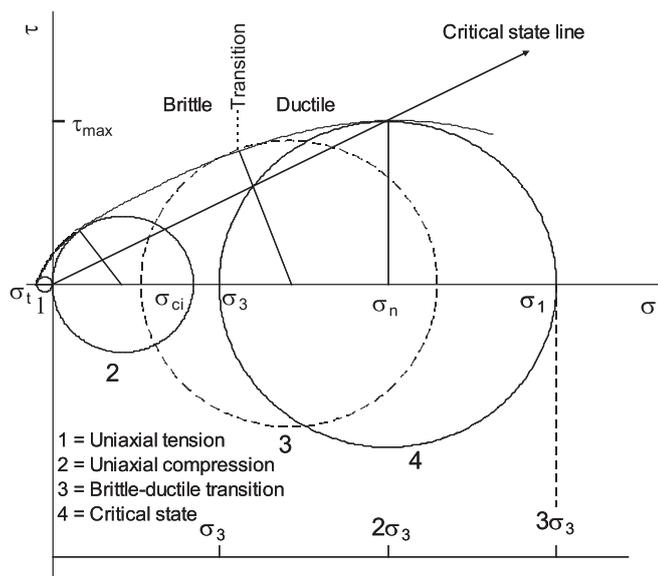


Figure 2 - Critical state concept of rocks (Barton, 1976)

The concept suggests that with increasing confining pressure the rock passes through different stages i.e. brittle, ductile and then critical state. Once critical stage is reached, there is no further increase in shear strength with increase in confining pressure.

The authors (Singh et al., 2011) analysed more than about 150 triaxial test data sets collected from world-wide literature and concluded that critical state of intact rocks is reached when the confining pressure becomes nearly equal to the uniaxial compressive strength (UCS) of the intact rock. Considering critical confining pressure σ_{crti} equal to the UCS of intact rock σ_{ci} , the following form of the Modified Mohr Coulomb criterion was proposed:

$$(\sigma_1 - \sigma_3) = \sigma_{ci} + \frac{2 \sin \phi_{i0}}{1 - \sin \phi_{i0}} \sigma_3 - \frac{1}{\sigma_{ci}} \frac{\sin \phi_{i0}}{(1 - \sin \phi_{i0})} \sigma_3^2 \text{ for } 0 \leq \sigma_3 \leq \sigma_{ci} \quad (4a)$$

$$(\sigma_1 - \sigma_3)_{\max} = \frac{\sigma_{ci}}{1 - \sin \phi_{i0}} \text{ for } \sigma_3 > \sigma_{ci} \quad (4b)$$

Later Singh and Singh (2012) showed that critical state concept is also applicable to jointed rocks and rock masses (Fig. 3). The Modified Mohr Coulomb criterion, initially proposed for intact rocks, was extended to jointed rocks and rock masses. The MMC for jointed rocks is expressed as:

$$(\sigma_1 - \sigma_3) = \sigma_{cj} + \frac{2 \sin \phi_{j0}}{1 - \sin \phi_{j0}} \sigma_3 - \frac{1}{\sigma_{crtj}} \frac{\sin \phi_{j0}}{(1 - \sin \phi_{j0})} \sigma_3^2 \text{ for } 0 \leq \sigma_3 \leq \sigma_{crtj} \quad (5a)$$

$$(\sigma_1 - \sigma_3)_{\max} = \sigma_{cj} + \frac{\sin \phi_{j0}}{1 - \sin \phi_{j0}} \sigma_{crtj} \text{ for } \sigma_3 > \sigma_{crtj} \quad (5b)$$

where

σ_{crtj} = critical confining pressure of the jointed rock,

σ_{cj} = anisotropic UCS of the jointed rock in the direction of major principal stress and

ϕ_{j0} = limiting friction angle of the rock mass for the condition $\sigma_3 \rightarrow 0$.

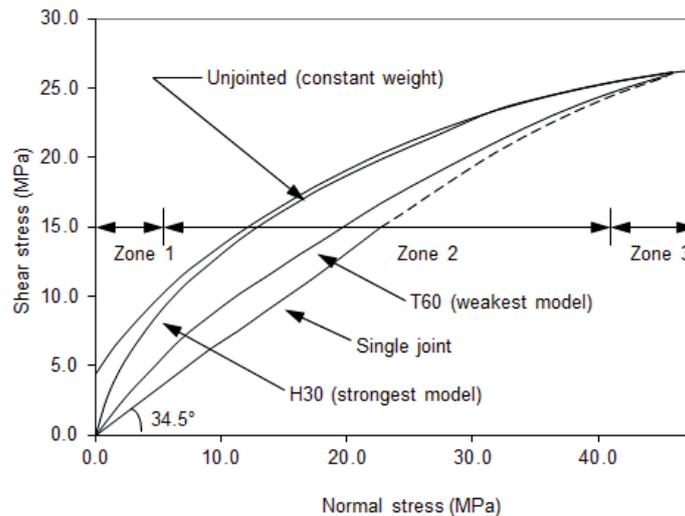


Figure 3 - Failure envelopes for intact and jointed rock mass specimens tested by Brown (1970)

The failure envelopes merge with each at sufficiently high confining pressure (zone-3) indicating that the jointed rocks also follow critical state concept in the same manner as the intact rocks do.

The UCS of the rock mass, σ_{cj} is an important input parameter in MMC criterion for jointed rocks. For Rock Mechanics applications, the value of σ_{cj} may be obtained from the following approaches:

- i. RQD (Zhang, 2010),
- ii. Rock Mass Rating (Bieniawski, 1973),
- iii. Rock Mass Quality Q (Barton et al., 1974),
- iv. Joint Factor (Ramamurthy, 1993; Ramamurthy and Arora 1994; Singh, 1997; Singh et al., 2002),
- v. Fracture Tensor (Kulatilake et al., 1997; Kulatilake et al., 2001) and
- vi. Rock Mass Index, RMi (Palmström, 1997).

Singh and Singh (2012) argued that the most reliable results can only be obtained through field testing. However, loading a rock mass to failure in the field is extremely difficult, they proposed an alternative approach. According to this approach, the rock mass is loaded only up to a predefined stress level, and its modulus is determined. The uniaxial compressive strength (UCS) of the rock mass can then be estimated from the modulus, as suggested by Singh (1997). Singh (1997) conducted a large number of uniaxial compression tests on model rock mass specimens (Fig. 4). Based on these results, Singh and Rao (2005) demonstrated that, for rock masses with varying degrees of fracturing, there is a strong correlation between the reduction in modulus and the reduction in strength (Fig. 5). This correlation was expressed as:

$$\text{SRF} = (\text{MRF})^{0.63} \tag{6}$$

where

- SRF = Strength Reduction Factor (σ_{cj}/σ_{ci}),
- MRF = modulus reduction factor (E_j/E_i),
- E_j = elastic moduli of the rock mass and
- E_i = elastic moduli of the intact rock.

Intact rock modulus, E_i may be obtained from laboratory tests. Rock mass modulus E_j may be obtained from uniaxial jacking tests as explained by Singh and Rao (2005) (Fig. 6). Uniaxial jacking tests (IS: 7317:1974) or cyclic plate load tests may be performed in the field, and the field modulus may be obtained as:

$$E_e = \frac{m(1-\nu^2)P}{\sqrt{A} \delta_e} \tag{7}$$

where

- E_e = field modulus of elasticity of the rock mass (kg/cm^2),
- δ_e = elastic deformation of the rock face (cm),
- $m = 0.96$ for circular plate of 25 mm thickness,
- ν = Poisson's ratio (assumed as 0.2),
- P = load on the test plate (kg), and
- A = area of the plate (2828 cm^2 for 60 cm diameter plate).

Four typical failure modes were observed depending on joint configurations, namely: splitting, shearing, rotation and sliding.

4. EXPERIMENTAL PROGRAM

4.1 Equipment

A series of direct shear tests were conducted using large, medium, and conventional-sized direct shear test apparatuses (Venkateswarlu, 2022). The tests were carried out on various compositions of granular rock-soil mixtures.

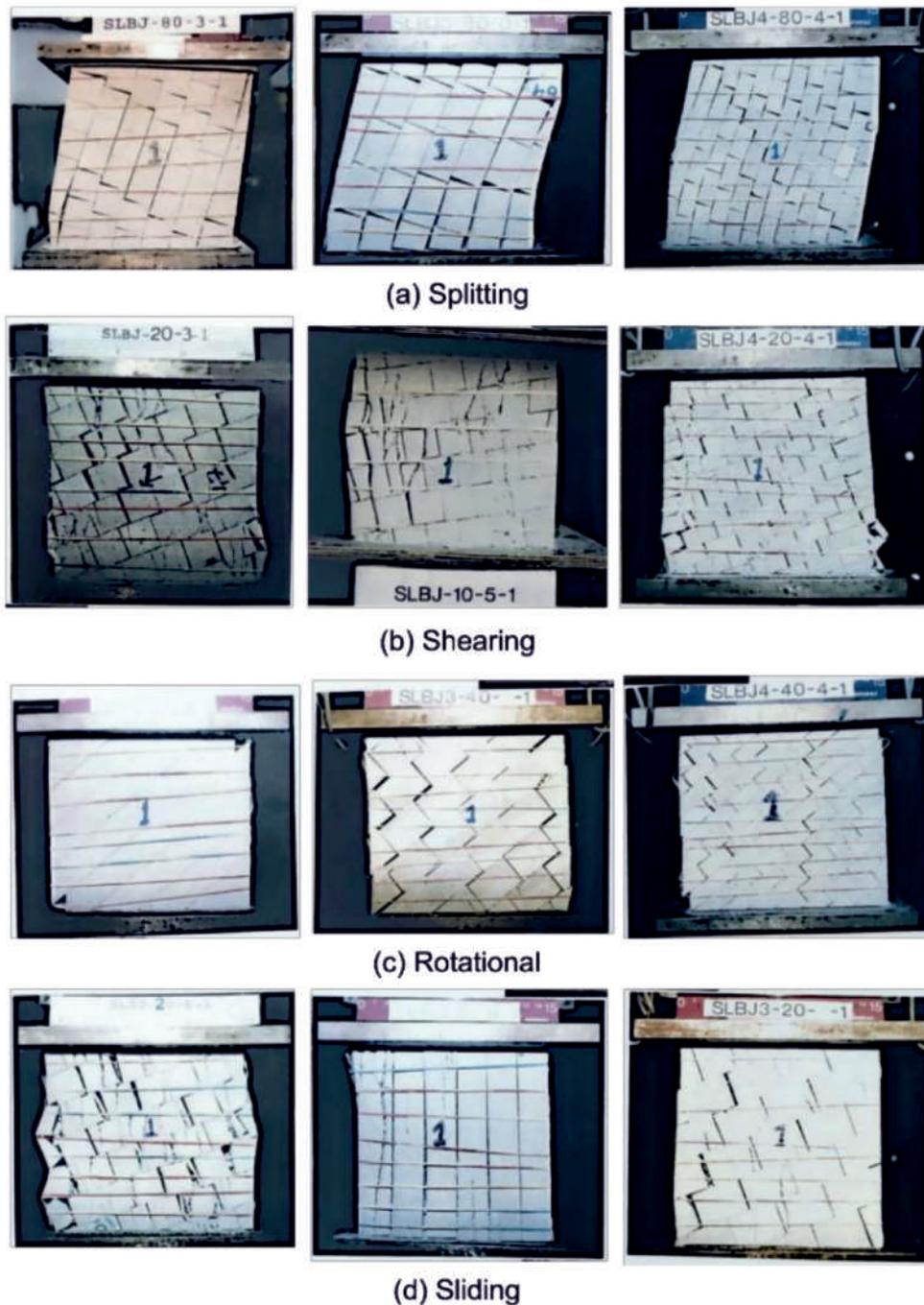


Figure 4 - Typical rock mass specimens tested by Singh (1997) under uniaxial loading condition

To study the effect of particle shape, particles ranging from very angular to sub-rounded were selected for investigation. The tests were performed by using three different sizes of direct shear test apparatus.

- i. Large size direct shear test apparatus: The size of the shear box is 750 mm × 750 mm × 1000 mm height (Fig. 7). The maximum normal and shear load capacities of the apparatus are 1500 kN and 2000 kN respectively. The maximum particle size used was 125 mm.
- ii. Medium size direct shear test apparatus: The size of shear box is 300 mm × 300 mm × 200 mm (height) (Fig. 7) with maximum particle size of 40 mm.
- iii. Conventional direct shear test apparatus was used for testing sand fines passing through 4.75 mm sieve of rock soil mixtures. The size of the shear box size was 60 mm × 60 mm × 25 mm (height).

The number of tests and range of normal stress used are shown in Fig. 8.

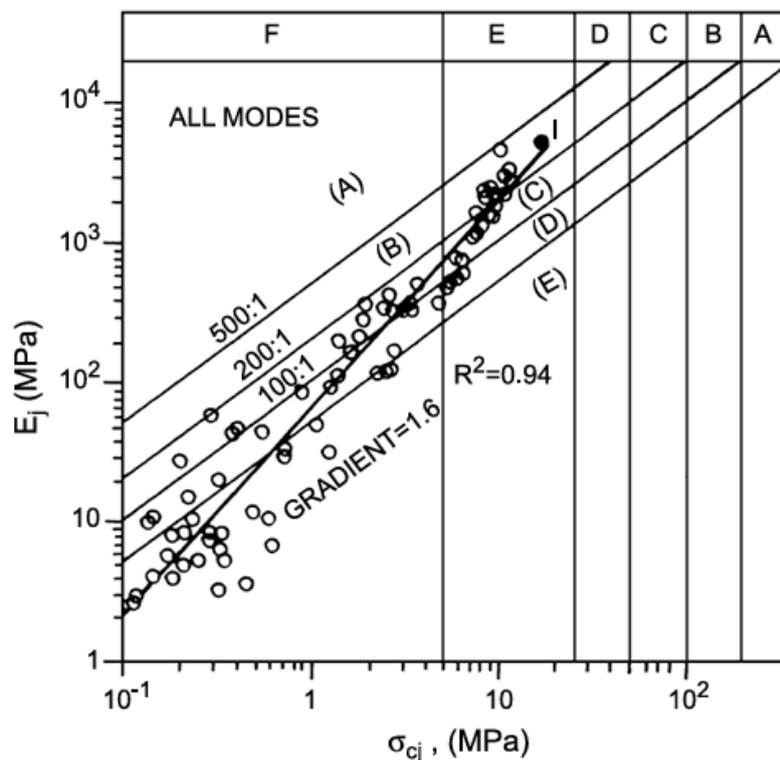


Figure 5 - Strength and modulus values of rock mass

Specimens tested by Singh (1997) plotted on Deere-Miller Classification chart. The letter “I” denote the intact rock position. Plot indicates that modulus reduction and strength reduction due to jointing are closely related with each other.

The plot can be used to obtain the field modulus of elasticity. Cyclic plate load may also be conducted in the field in similar manner to obtain the field modulus.

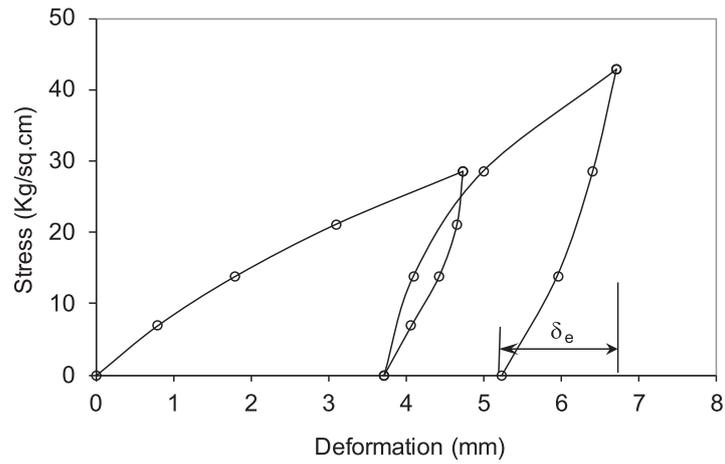


Figure 6 - Typical results of uniaxial jacking test conducted in field

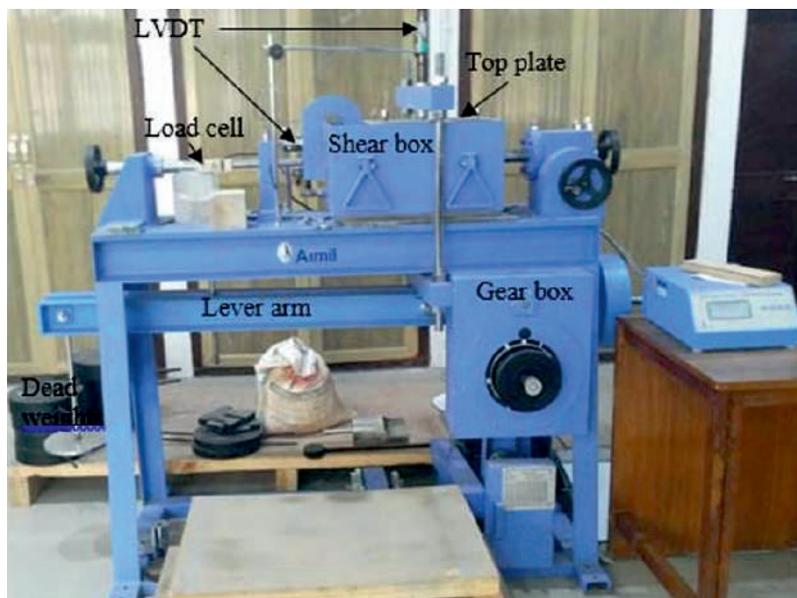


Figure 7 - Large and medium size direct test apparatus used for direct shear tests of rock-soil mixtures

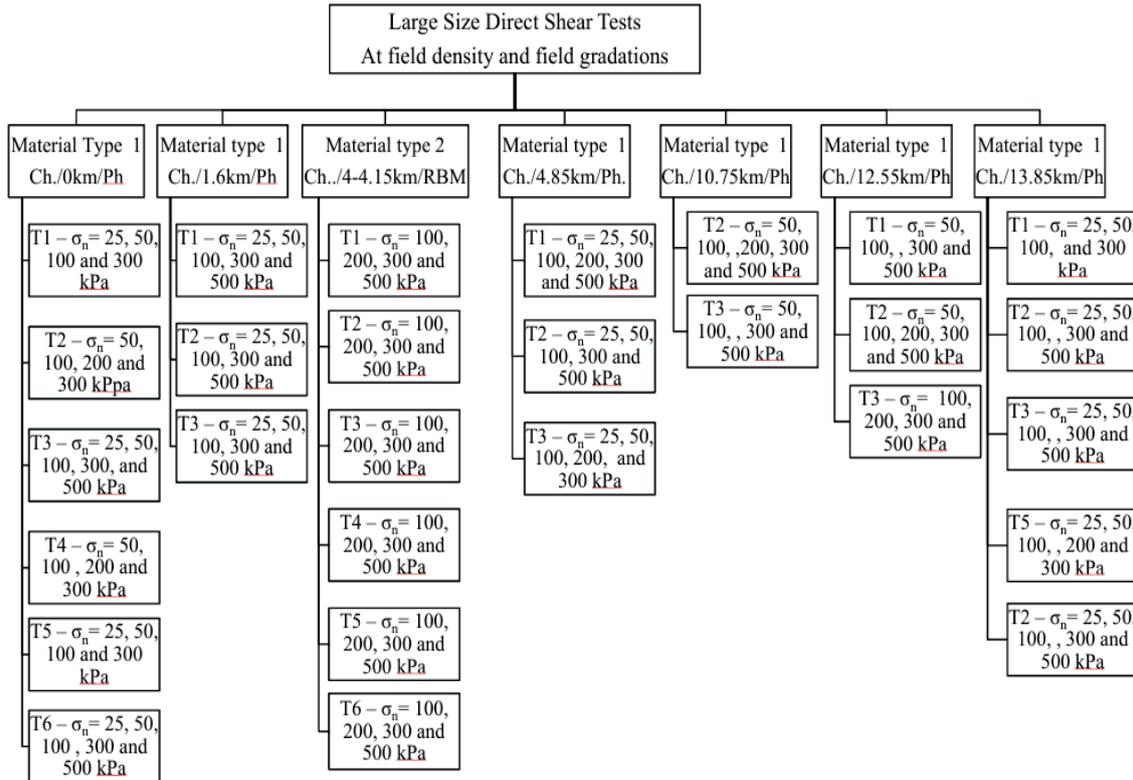


Figure 8a - Details of large size direct shear tests

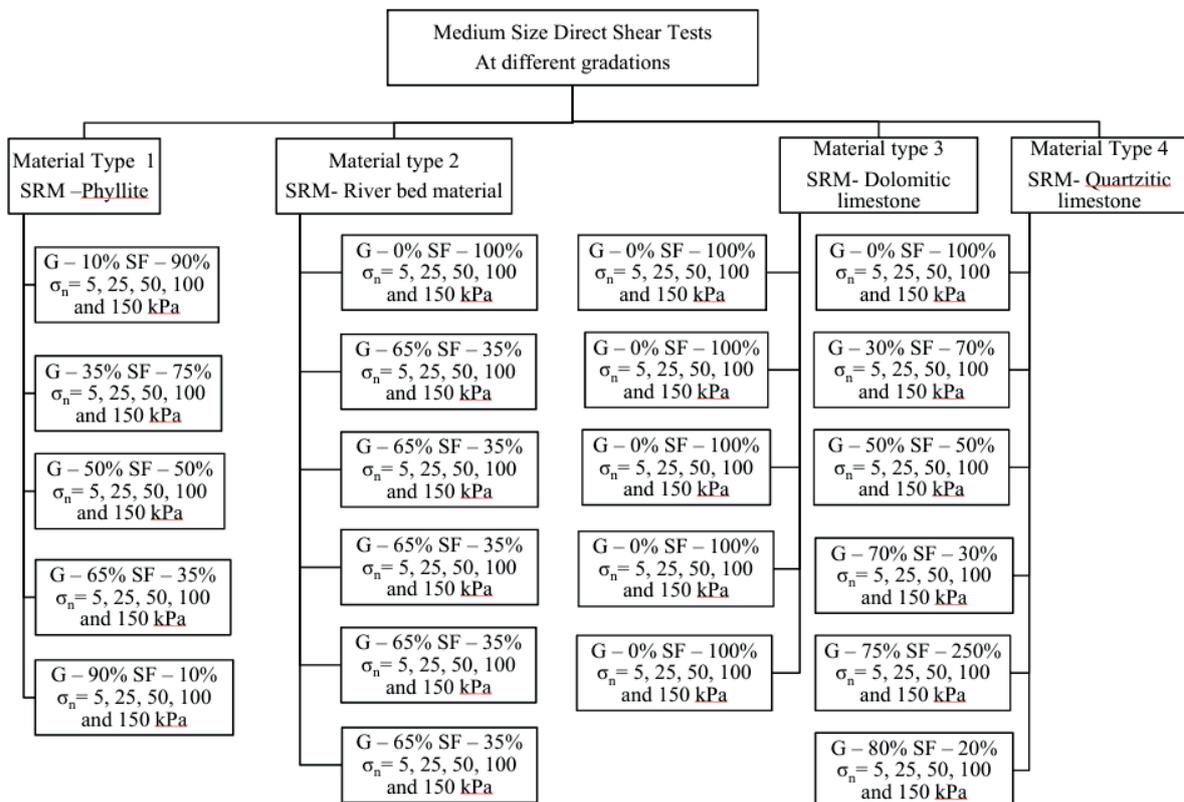


Figure 8b - Details of medium size direct shear tests

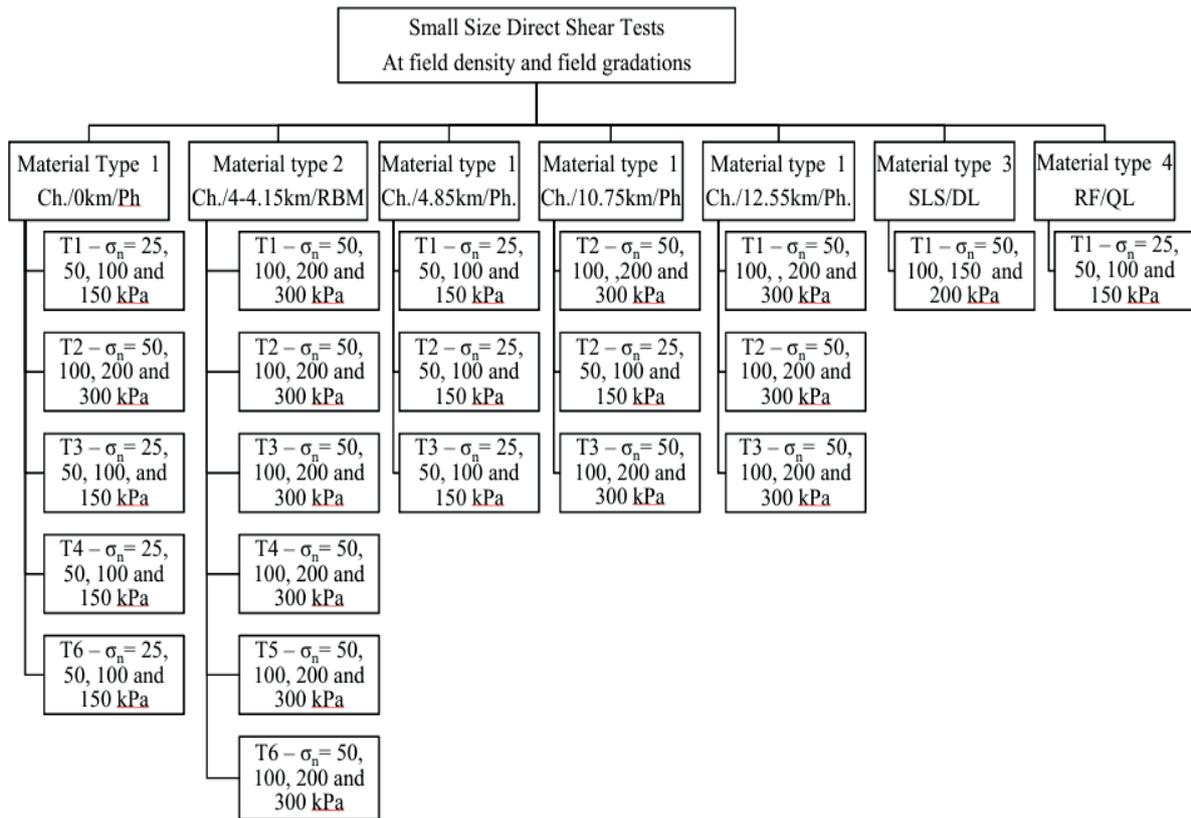
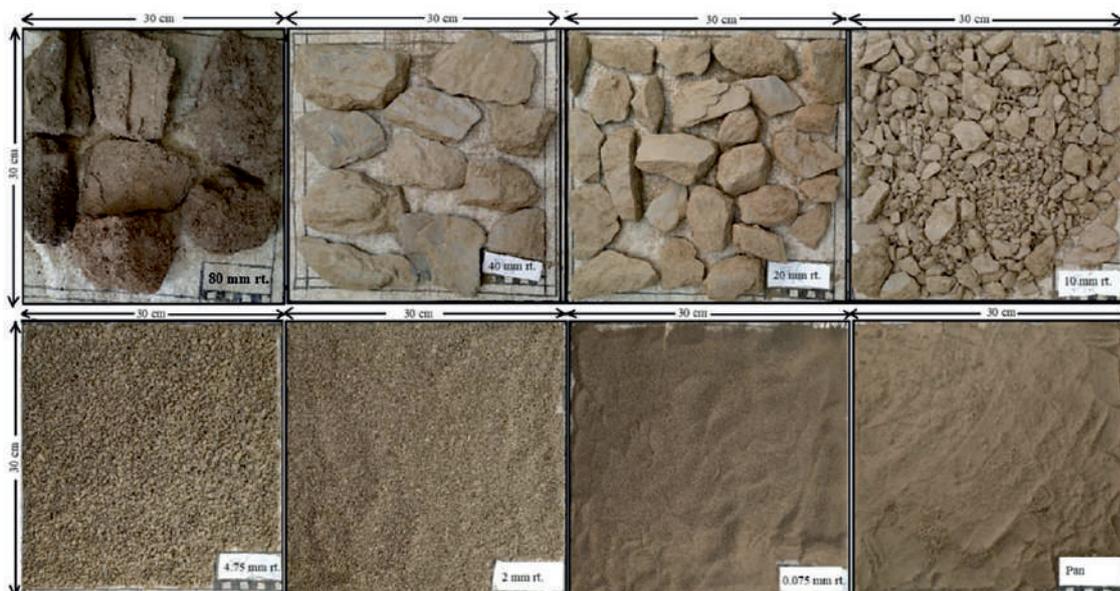


Figure 8c - Details of conventional direct shear tests

4.2 Materials

Four types of materials were used for testing. The sieved fractions of the materials are shown in Figure 9. Combinations of different fractions of gravels were mixed with sand to form the specimens for medium size tests. The following four material types were prepared for testing.



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Figure 9a - Sieved fractions of phyllitic rock-soil-mixture (Material type -1)

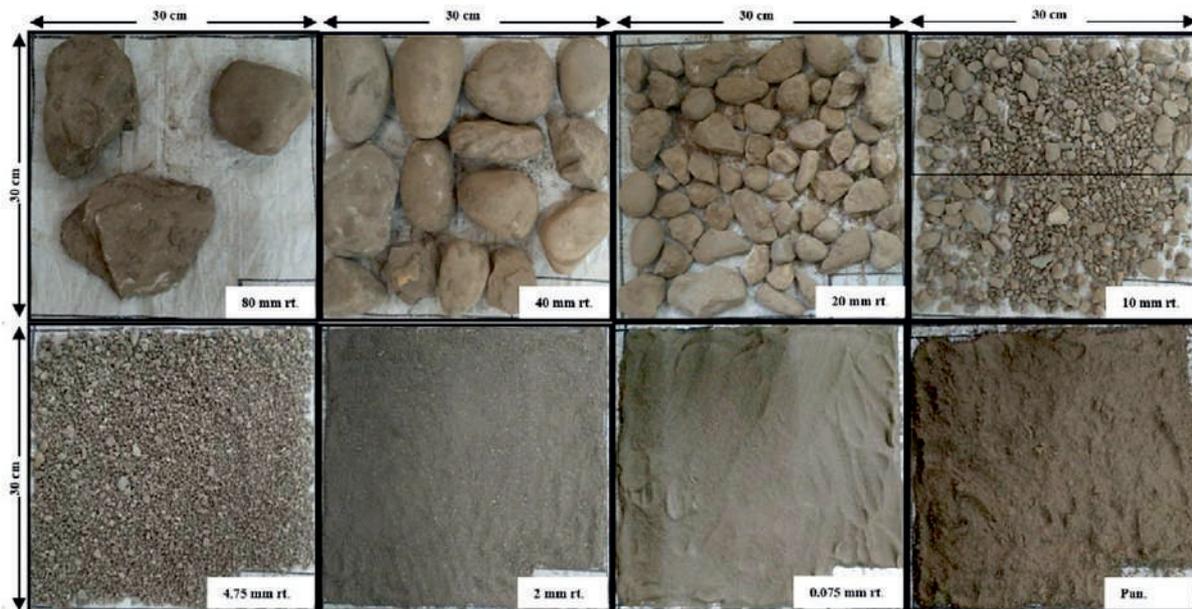


Figure 9b - Sieved fractions of RBM (Material type -2)

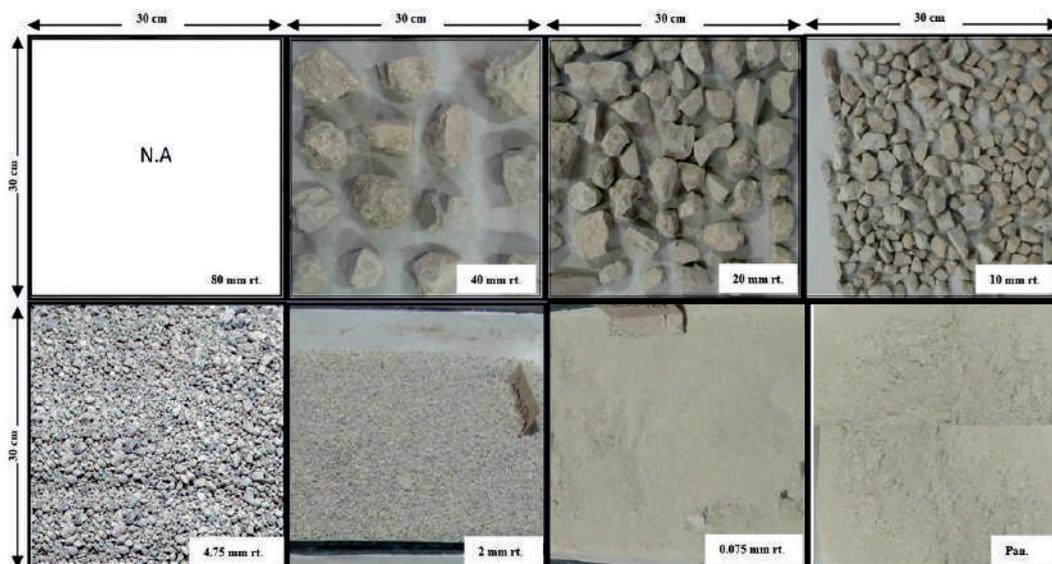


Figure 9c - Sieved fractions of dolomitic limestone rock-soil-mixture (Material type -3)

Material Type 1: Debris material consisting of a heterogeneous mixture of phyllitic rock fragments and soils, collected from talus slopes near the Koteshwar Dam site in the Himalayan region of Uttarakhand, India.

Material type 2: Riverbed material (RBM) collected from the slopes adjacent to Koteshwar dam site. The particle shape is sub-rounded to rounded.

Material Type 3: Debris material consisting of colluvium collected from a landslide near Surbhi Resort, NH-92, Mussoorie, Uttarakhand, India. This debris comprises a mixture of fine-grained materials and rock fragments of dolomitic limestone (DL).

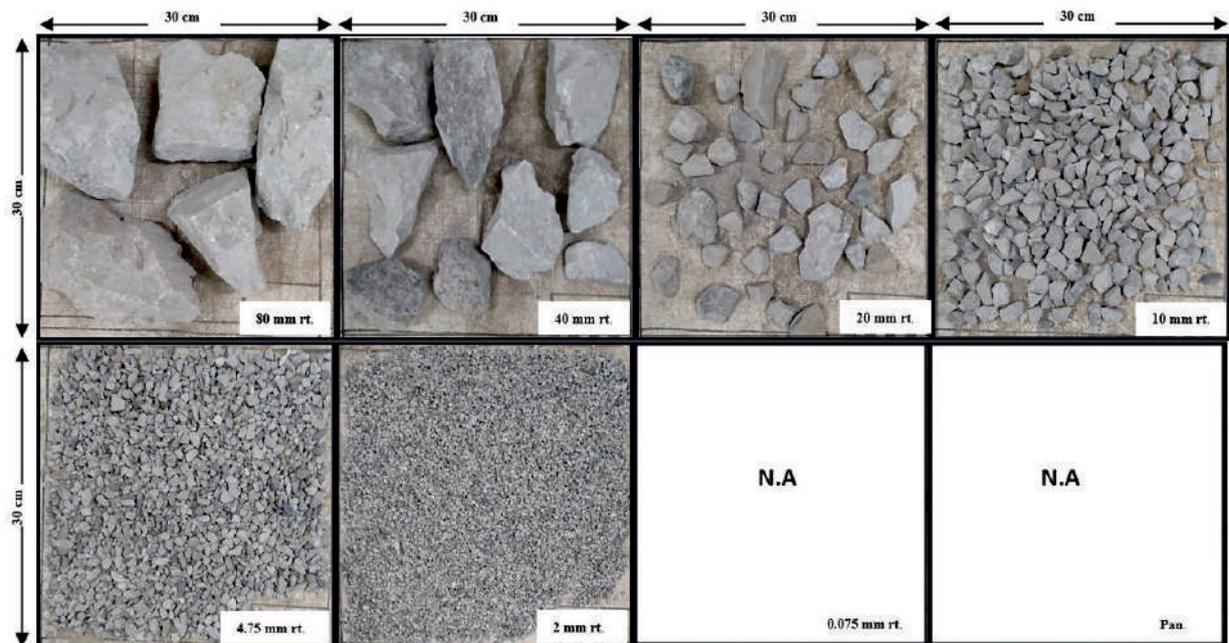


Figure 9d - Sieved fractions of quartzitic limestone rock-soil-mixture (Material type -4)

Material Type 4: Rockfill (RF) collected from a quarry site in western India. The material consists of quartzitic limestone (QL) with very angular particles featuring sharp edges.

The grain size distributions of different material types are shown in Figure 10.

5. RESULTS

Typical results of direct shear tests are shown in Figure 11. The results were obtained in the forms of shear stress vs. horizontal shear displacement plots and dilation vs horizontal shear displacement plots. Peak shear stress was taken as shear strength of the material. Typical failure envelopes in the form of shear strength vs normal stress plots are presented in Figure 12. Summary of failure envelopes obtained from large and medium size direct shear tests is presented in Figure 13. The results show high non-linearity of shear strength response with normal stress for rock-soil mixtures. The peak friction angle at given normal stress was obtained by dividing the shear strength by the respective normal stress. A summary of the values of peak friction angle from the tests is presented in Figure 14, where the angle is plotted against normal stress. It is seen that the angle varies substantially with change in normal stress. In all the cases, the peak friction angle decreases with increase in normal stress.

The results indicate that linear Mohr-Coulomb failure criterion has very limited applicability for assessment of shear strength of rock-soil mixtures. Therefore, non-linear shear strength models should be used when evaluating shear strength of rockfill or determining the peak friction angle.

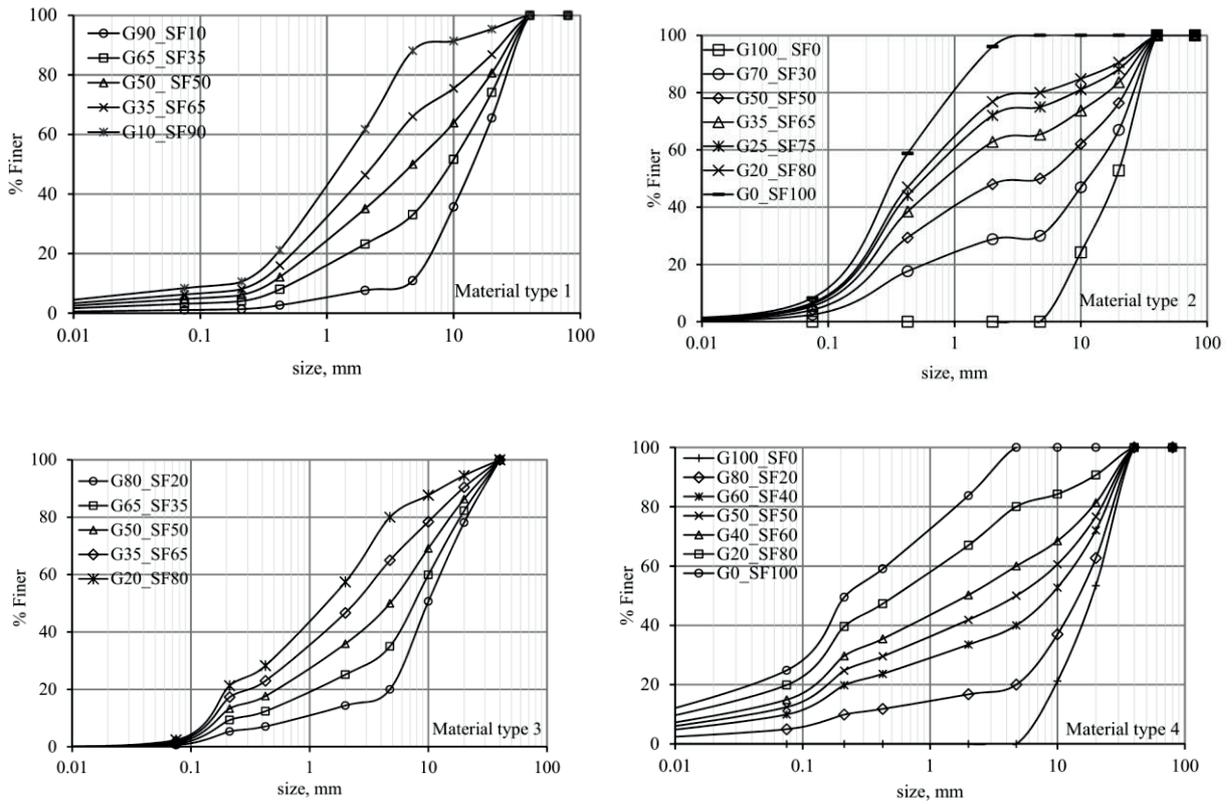


Figure 10 - Grain size distributions of different material types

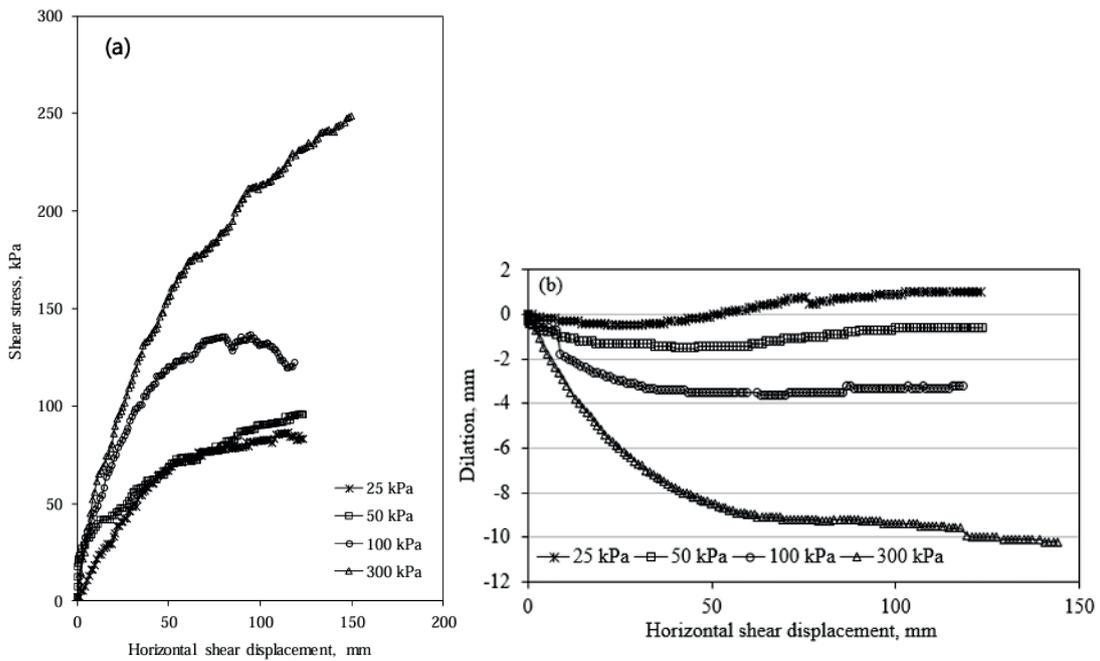


Figure 11 - Typical shear stress vs shear displacement and dilation vs shear displacement plots from large size direct shear tests

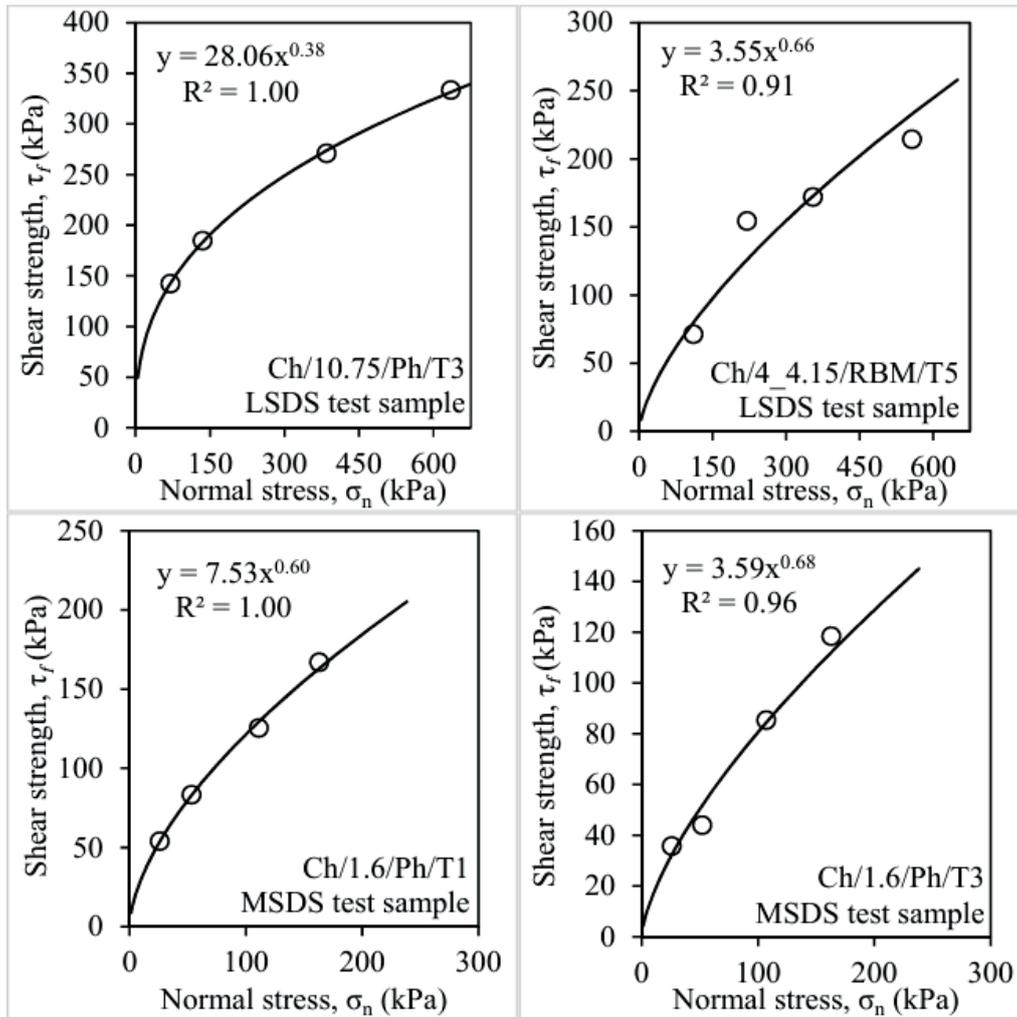


Figure 12 - Typical failure envelope plots from direct shear tests

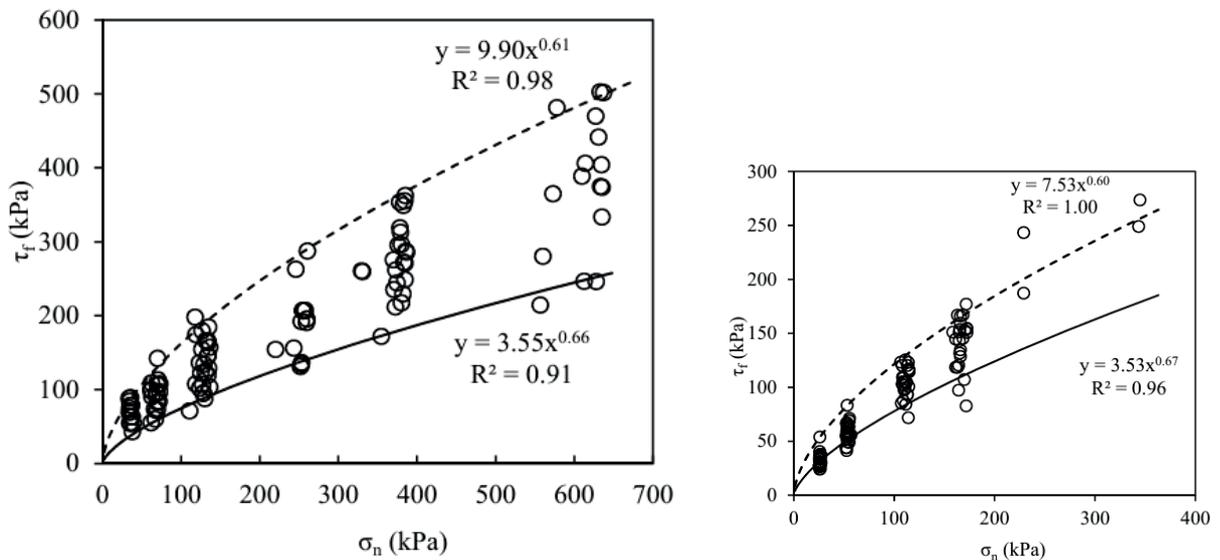


Figure 13 - Summary of failure envelopes from large and medium size direct shear tests

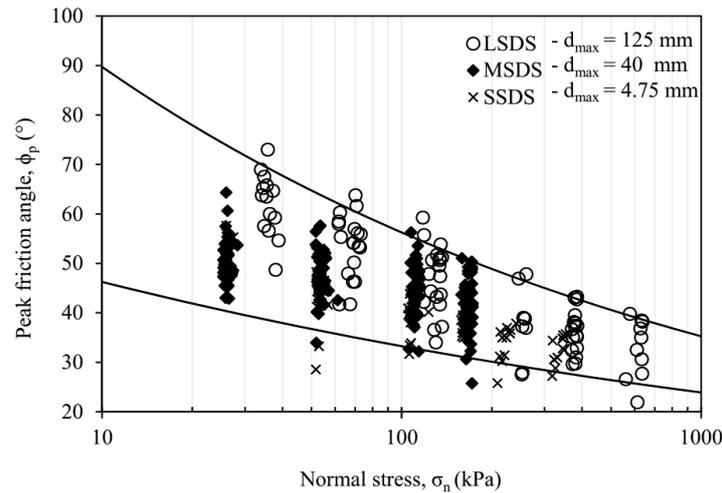


Figure 14 - Variation of peak friction angle with normal stress for large, medium and conventional direct shear tests

6. APPLICATION OF ROCK MECHANICS MODELS

Two shear strength models from Rock Mechanics namely Barton’s JRC-JCS model and Modified Mohr Coulomb criterion, have been suggested to assess the peak friction angle of rockfills. The results from experimental program were analysed and several trials were made. Finally, the following step by step procedure is suggested to obtain peak friction angle of rockfills:

6.1 Barton’s JRC-JCS Model

The following inputs are required for using this model:

- i. UCS of parent rock, σ_{ci} ,
- ii. Gravel fraction in percent, $G\%$ (20 to 80%),
- iii. Maximum particle size (represented by d_{90}), and mean particle size, d_{50} .

The peak friction angle corresponding to normal stress, σ_n , is obtained as:

$$\phi_p = \phi_r + JRC_{eq} \log_{10} \left(\frac{JCS_{eq}}{\sigma_n} \right) \quad (8)$$

where equivalent JCS and JRC are computed as

$$JCS_{eq} = \frac{\sigma_{ci}}{10} \frac{G\%}{100} \quad (G = 20 \text{ to } 80\%) \quad (9)$$

$$JRC_{eq} = \alpha \left(\frac{d_{50}}{d_{90}} \right) \leq 20 \quad (10)$$

$$\alpha = 0.3 \times d_{90} + 16 \leq 50 \quad (11)$$

where d_{50} and d_{90} are in mm.

6.2 Modified Mohr-Coulomb (MMC) Criterion (Singh and Singh, 2012)

The following step-by-step procedure is suggested:

i. Estimation of σ_{cj} of rockfill: Perform deformability test (cyclic plate load test) in the field to get deformation modulus E_j , and obtain hypothetical (apparent) rock mass strength σ_{cj} using following expression:

$$\frac{\sigma_{cj}}{\sigma_{ci}} = \left(\frac{E_j}{E_i}\right)^n \tag{12}$$

where

E_i = intact rock modulus and
 $n \approx 0.94$ to 1.1 (with average value of 0.92).

ii. Compute friction angle ϕ_{j0} (corresponding to $\sigma_3 \rightarrow 0$):

$$\phi_{j0} = 16.5 \left(\frac{G\%}{100}\right) + 16.54 \log_{10} \left(\frac{JCS_{eq}}{5 \text{ kPa}}\right) - 10.3 \text{ degrees} \tag{13}$$

where

$$JCS_{eq} = \frac{\sigma_{ci}}{10} \times \frac{G\%}{100} \quad (G = 20 \text{ to } 80\%)$$

iii. Compute triaxial strength for a set of σ_3 values through following expressions:

$$(\sigma_1 - \sigma_3) = \sigma_{cj} + \frac{2\sin\phi_{j0}}{(1-\sin\phi_{j0})} \sigma_3 + \frac{1}{\sigma_{crtj}} \frac{\sin\phi_{j0}}{(1-\sin\phi_{j0})} \sigma_3^2 \quad \text{for } 0 \leq \sigma_3 \leq \sigma_{crtj} \tag{14}$$

$$(\sigma_1 - \sigma_3) = \sigma_{cj} + \frac{2\sin\phi_{j0}}{(1-\sin\phi_{j0})} \sigma_{crtj} \quad \text{for } \sigma_3 > \sigma_{crtj} \tag{15}$$

where

σ_3 and σ_1 = minor and major principal stresses at failure respectively and
 $\sigma_{crtj} = 4\sigma_{cj}$.

iv. The non-linear failure envelope can now be derived from the set of minor and major principal stresses obtained above. The peak friction angle corresponding to a given normal stress can then be determined. For a given confining pressure, the peak friction angle, ϕ_p , may be computed as:

$$\sin\phi_p = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 + \sigma_3)} \tag{16}$$

7. COCLUDING REMARKS

The shear strength response of rockfills exhibits significant non-linearity and stress dependence. In present article, the applicability of two shear strength models from the field of Rock Mechanics namely Barton’s JRC-JCS model, and Modified Mohr Coulomb Criterion (Singh and Singh, 2011) has been extended to evaluate the non-linear shear strength response of rockfills. The input parameters for these models can be easily obtained through field and laboratory tests, enabling the assessment of the non-linear shear strength response of rockfills.

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