PHYSICAL MODELING OF A ROCK MASS UNDER A TRUE TRIAXIAL STRESS STATE

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Abstract: A study was planned in which several (77 nos.) scale free 15 cm size block mass models having three orthogonal continuous smooth saw cut joint sets and varying inclination '0' of joint set-I with horizontal were tested in true-triaxial system (TTS) developed by Rao and Tiwari (2002). The specimens were prepared with sand-lime brick model material of uniaxial compressive strength 13.5 MPa and representing 'EM' category on Deere-Miller's classification chart. The test results show that with increase in \(a_2\) in dip direction of joint set-I, the strength of rock mass \(\sigma\) and deformation modulus \(E_d\) increase significantly which is confirmed by fracture shear planes developed on \(a_3\) face of specimen, dipping in \(a_1\) direction and steeping with increase in \(a_2\) values. The strength enhancement was maximum (309.2\%) for inclination, \(0 = 60^\circ\) and minimum (24.2\%) for \(0 = 90^\circ\). The axial stress-volumetric strain curves show onset of dilatancy in rock mass and is increasing function of \(a_2\). Most of the specimens failed in shearing with sliding in some cases. The effect of interlocking \(s\) and rotation of principal stresses \(c_2\) and \(c_3\) on strength and deformation response was also investigated for few specimens with \(0 = 60^\circ\). Finally, suitable failure criteria were also evolved in both triaxial and true-triaxial stress states.

Keywords: True triaxial system, Rock mass, Physical modeling, Intermediate principal stress, Dilatancy, Scale effect

1. INTRODUCTION

Civil engineering activities at the ground surface are mainly governed by uniaxial stress condition where confining pressures i.e. minor \(a_3\) and intermediate \(a_2\) principal stresses are negligible or zero. However, at depth stress state will be true triaxial \(p_1> G_2>a_3\) or polyaxial. It is observed that as one goes farther inside an approach drift, the joints become tighter and the persistence of joints decreases significantly. Further, the dilatancy in the rock mass is constrained in tunnels causing increase in its modulus value and strength. In literature also the strength enhancement in intact rock (Haimson and Chang, 2000) and rock mass at site (Singh et al. 1998) has been reported due to \(c_2\). Reik and Zacas (1978) carried out an experimental study on jointed model in true triaxial compression and observed strength enhancement up to 200\%. Singh et al. (2002) conducted extensive model studies on jointed block mass with three sets of orthogonal joints in unconfined compression state (UCS). The studies so available are preliminary works on intact and jointed rocks in true triaxial compression.

In the present paper an experimental study was planned in which jointed block mass models similar to Singh et al. (2002) were tested in true triaxial stress state \(G_1>G_2>G_3\) to assess engineering response of rock mass.

2. EXPERIMENTAL PROGRAMME

2.1 Model material and geometry

The model material used as substitute rock was sand-lime brick get manufactured from U.P.
Minerals Products Ltd. Meerut, India. This was formed by mixture of hydrated lime, fine sand and water in the proportion of 1: 4: 0.20 by weight and pressed in dies at about 39.6 MPa followed by autoclaving for 4 hours at 180°C and air curing for three weeks. Its suitability to use as brittle model material was verified (Tiwari and Rao, 2003). The model material properties have been listed in Table 1. The specimens were prepared by cutting model material bricks into small cubes of size 2.5 cm and arranging the small cubes to form the test specimen of size 15 cm x 15 cm x 15 cm having three sets of orthogonal joints. The joint set-I was continuous and inclined at various angles D=0°, 20°, 40°, 60°, 80° and 90° with X-axis. The joint set-II was perpendicular to set-I and was kept at stepping (s) = 0.5 of width (b) of small block. The set-III was always continuous (Fig.1). Each specimens were given unique identification number viz. A60/0.31/1.62 which indicates Type-A sample of 0=60° and is acted by principal stresses, \( G^1 = 0.31 \) and \( C_T^2 = 1.62 \) along Y and X-axis respectively.

Table 1. Material properties of sand- lime brick

<table>
<thead>
<tr>
<th>Property</th>
<th>Range</th>
<th>Av. value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry density, ( y_d ) (KN/m³)</td>
<td>16.10-17.80</td>
<td>16.86</td>
</tr>
<tr>
<td>Specific gravity, ( G )</td>
<td>2.66-2.67</td>
<td>2.66</td>
</tr>
<tr>
<td>Absolute porosity, ( n ) (%)</td>
<td>32.70-39.30</td>
<td>36.50</td>
</tr>
<tr>
<td>UCS (intact cyl.), ( a_{ij} ) (MPa)</td>
<td>11.10-15.90</td>
<td>13.50</td>
</tr>
<tr>
<td>Tensile strength, ( a_{ij} ) (MPa)</td>
<td>1.63-2.17</td>
<td>1.89</td>
</tr>
<tr>
<td>Failure strain UCS, ( s ) (%)</td>
<td>0.43-0.63</td>
<td>0.53</td>
</tr>
<tr>
<td>Poisson's ratio, ( v )</td>
<td>0.18-0.27</td>
<td>0.25</td>
</tr>
<tr>
<td>Modulus, ( E_{ij} ) (MPa)</td>
<td>4782-4866</td>
<td>4866</td>
</tr>
<tr>
<td>Internal friction angle, ( \phi ) (°)</td>
<td>-</td>
<td>44.70</td>
</tr>
<tr>
<td>Cohesion, ( c ) (MPa)</td>
<td>-</td>
<td>3.29</td>
</tr>
<tr>
<td>Joint friction angle, ( \phi_j ) (°)</td>
<td>-</td>
<td>36.80</td>
</tr>
<tr>
<td>Deere-Miller's chart</td>
<td>EM</td>
<td></td>
</tr>
</tbody>
</table>

2.2 Testing equipment

The True Triaxial System (TTS) designed and fabricated at I.I.T. Delhi by Rao and Tiwari (2002) was used in the present study (Fig. 2). The system consists of a 100-ton capacity vertical frame, a biaxial frame of 30-ton capacity fitted with two pairs of hydraulic jacks and platen, constant confining pressure unit for applying, monitoring and maintaining horizontal stresses \( (a_2 \text{ and } C_T^3) \) on specimen faces, eight channel data acquisition system and personal computer with ad cord to record all load and deformation data.

\[
\text{G} = 0^\circ, 20^\circ, 40^\circ, 60^\circ, 80^\circ, 90^\circ \\
\text{S} = 0, 0.25, 0.50, 0.75 \text{ of } b^1
\]

Figure 1. Key sketch of model tested

2.3 Testing methodology

The prepared samples were dried in room temperature for 30 days for testing in TTS. The specimen is carefully transferred on loading platen and platen from all sides of specimen are moved to touch the faces. The friction free, uniform stresses on all six faces is ensured by firstly putting 0.5 mm thick pair of Teflon sheets smeared with high vacuum silicon grease between specimen and platen and secondly by movement of horizontal frame in vertical and horizontal planes using screw levers. The final micro adjustment of platen on specimen lateral faces is done by moving platens in vertical and horizontal planes with help of mini screw levers fitted on biaxial frame.

The three independent stresses were applied initially in quasi-hydrostatic conditions i.e. \( G^1 > G^2 = a_2 \). The \( G^1 \) was applied in vertical direction, \( G_2 \) was along dip direction of joint set-I and \( c_3 \) is in third orthogonal axis. After reaching the predetermined level of \( a_2 \) and \( a_3 \), the \( G^1 \) was increased monotonically from \( a_3 = a_2 \) to a level when sample yields and even in post failure zone till residual strength of specimen. In all cases of testing the strain controlled vertical loading is applied at a rate such that sample fails in 15-20 minutes. The several testing cases have been illustrated in Table 2. The deformation of system and pair of Teflon sheets smeared with high vacuum silicon grease with vertical loading was also determined separately. The correction for this deformation was made in all load deformation data.
Figure 2. True triaxial system used for study (Rao and Tiwari, 2002)

Table 2. Details of testing programme

<table>
<thead>
<tr>
<th>Test cases</th>
<th>9° and s details</th>
<th>$G_1$, MPa</th>
<th>$c_2$, MPa</th>
<th>$c_3$, MPa</th>
<th>$c_2/c_3$ ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type A (54)</td>
<td>s = 0.5, e = 0, 20, 40, 60, 80, 90</td>
<td>0.31</td>
<td>0.31, 0.59</td>
<td>0.78, 1.22, 1.62</td>
<td></td>
</tr>
<tr>
<td>Type B (7)</td>
<td>s = 0.5, 9 = 60</td>
<td>0.31</td>
<td>0.31, 0.59</td>
<td>0.78, 1.22, 2.24</td>
<td></td>
</tr>
<tr>
<td>Type C (16)</td>
<td>s = 0, 25, 50, 75</td>
<td>0.31</td>
<td>0.31, 0.59</td>
<td>1.22, 1.62</td>
<td></td>
</tr>
</tbody>
</table>

3. TEST RESULTS

The stress strain behaviour, dilatancy, strength, modulus and the failure modes were examined for each test in order to analyse the fracture mechanism of jointed block mass.

3.1 Stress strain behaviour

The typical stress strain curves has been plotted in Figs.3 and 4. The curve shows deviotion stress ($p_c$ - $c_3$) plot against axial strain ($s_0$, lateral strain along $G_2$ ($S_2$), lateral strain along $a_3$ ($s_3$), and volumetric strain $s_v$). The stress strain curves reveal some significant features of rock mass behaviour as below:

(i) Brittle failure for samples of type -A case with $9 = 0°, 20°, 80°$ and $90°$ showing steep elastic portion up to almost the peak and gradual fall in stress (strain softening) after failure. The steepness of curve increases in both region with increase in $c_2/c_3$ ratio (Fig. 3).

(ii) The ductile behaviour for type-A $9 = 40°$, $60°$, type -B and type -C specimens showing short elastic range followed by a continuous increase in stress and decrease in tangent modulus with increasing strain up to failure (Fig.4). Beyond peak stress increase is continued (strain hardening) in most cases and constant stress (Plastic) in some cases.
3.2 Failure pattern

The specimens showed different failure pattern depending upon joint configuration, stress ratio, and stress orientation as follows:

(i) Shearing of intact material and joints were observed in case of type-A specimens with \( \theta = 0°, 20°, 80° \) and 90°. The shear planes developed on YZ or face dips along \( c_3 \) direction and fracture dip increases with increasing \( c_2/c_3 \) levels. For A20 specimens shear plane develops at higher \( c_2/c_3 \) levels.

(ii) Joint dilation and shearing of some blocks in type-A, \( \theta = 40° \) and sliding along joint mixed with shearing in \( \theta = 60° \). Mode of failure shifted from sliding, dilation to shearing with increasing \( c_2/c_3 \) ratio. Type -B specimen shows sliding along joint and with higher \( c_2/c_3 \) ratio mixed mode of sliding and shearing.

(iii) Type -C specimen fails in mixed process of joint dilation, rotation and shearing at stepping, \( s=0 \) but as stepping increases samples fails in shearing along with joint dilation.

3.3 Onset of dilatancy

As the dilatancy is defined as the increase of volume relative to elastic changes caused by the deformation, the onset of dilatancy is the point along \( (\delta I - c_3) \) vs volumetric strain \( (s_v) \) plot where the initially linear curve begins to deviate towards a volumetric increase (Figs. 3 and 4). The percentage of major principal failure stress \( \delta I_{peak} \) at which dilatancy starts, is denoted as dilatancy stress, \( \delta I \)

Figure 6. Dilatancy stress for type -A case at \( \theta =0.31 \) Mpa and is found to increase with \( \theta \) levels for all cases, The type -A case has been shown in Fig.6.

3.4 True triaxial strength enhancement

The Fig. 7 shows effect of \( c_2 \) on failure strength at \( c_3 = 0.31 \) for type -A case. The strength enhancement is defined as \( (\delta I_{max} - \delta I_{triax}) / \delta I_{triax} \), where \( \delta I_{max} \) and \( \delta I_{triax} \) are true-triaxial and triaxial compressive strength respectively. The strength enhancement was plotted against angle, \( \theta (=90°-\theta) \), which is angle between direction of joint set-I and vertical loading axis. The maximum strength enhancement for A60 is equal to 309.2 %, which is considerably higher than 24.2% for A90. It occurs at least principal stress \( c_3 =0.31 \) MPa and \( c_2/c_3 = 5.2 \) (Fig.8). The strength enhancement is comparatively low at higher \( c_3 \) level of 0.78 MPa.

Test on Westerly granite by Haimson and Chang (2000) also indicates the increasing trend of failure strength with increase of \( c_2 \). They reported maximum 49 % of strength enhancement for intact rock at \( \theta =10 ° \). So strength enhancement in jointed rock mass samples are found to be more than for intact rock.

Strength anisotropy was also observed which diminishes with increasing \( c_2/c_3 \) ratio. The maximum anisotropy ratio (3.7) was seen at \( c_2/c_3 = 1 \), which gradually diminishes (1.2) at 5.2 (Fig.9). In type -B case some specimens were tested in true triaxial stress state by interchanging the direction of principal stresses \( c_2 \) and \( c_3 \) i.e. applying them along Y and X- axis respectively. The marked reduction in strength enhancement compared to type- A, \( \theta =60° \) was seen.
The strength enhancement reduces from 309.2 % to 116.4 % at $\sigma_3 = 0.31$ MPa and 43.5% to 19.3% at $\sigma_3 = 0.78$ MPa.

In type-C case the specimens were tested at varying stepping, $s = 0, 0.25, 0.5$ and $0.75$ for different $c_2/c_3$ ratios to assess the influence of interlocking on strength and modulus response. The test results show that strength and modulus value increases with increasing $c_2/c_3$ at each stepping but no marked trend in same was seen with variation in stepping ($s$).

### 3.5 Deformation modulus

The results obtained from type -A, B and C cases show that there is an increase in deformation modulus($E_j$) with increased $\sigma_2/\sigma_3$ ratio. Modulus increase is higher at low $\sigma_3$ level when $\sigma_2$ is along dip direction of joint set-I i.e. along X-axis. The modulus value also show anisotropy which reduces with increasing $c_2/c_3$ (Fig. 10).

### 4. STRENGTH CRITERIA

The failure strength values at $\sigma_2 = \sigma_3 = 0.31, 0.78$ and 1.22 MPa were plotted to non-linear Mohr’s envelope for type-A case specimens. The envelope yields to the Eqn. 1.

$$\sigma_1 = \sigma_{eq} + B\sigma_3^a$$

The B and a are joint geometry and material parameters respectively. The values of B and a vary with joint configuration and are listed in table 3. The $G_{CJ}$ is uniaxial compressive strength of jointed rock mass sample.
Based on true-triaxial testing results of type-A case specimens, a true-triaxial strength criterion in pattern of generalized von Mises theory (Mogi, 1971) was evolved as given Eqn. 2.

\[ T_{\text{oct}} = f(<y_{\text{oct}}) \]  

(2)

where, \( x_{\text{oct}} \) is the octahedral shear stress i.e. \( [(G_1-G_2)^2 + (O_2-C_3)^2 + (G_3-G_1)^2]/2 \) at the point when \( a_1 \) attains its peak value \( <\text{J}_1 \) and \( a_{\text{oct}} \) is the mean normal stress acting on failure plane and \( f \) is monotonically increasing function.

It based on assumption that yielding occurs in the entire body and the octahedral shear stress is a function of the full octahedral normal stress, \( a_{\text{oct}} \). The \( a_{\text{oct}} \) is taken as equal to \( (G_1+G_2+G_3)/3 \).

The strength criterion developed as Eqn. 3.

\[ \tau_{\text{oct}} = D\sigma_{\text{a}2} \sigma_{\text{oct}} \]  

(3)

The \( D \) and \( c \) are joint geometry and material parameters. \( D \) and \( c \) systematically varies with joint configuration (Table 3). The \( c_j \) is uniaxial compressive strength of model material. The high correlation coefficients (0.99 on average) indicate the close fit of the suggested relation. The criteria have been derived based on the assumption that joints are smooth (\( c_j = 36.8^\circ \)) and extent of interlocking level (\( s = 0.5 \)) is medium.

<table>
<thead>
<tr>
<th>Type A specimen (s=0.5),</th>
<th>Strength parameters</th>
<th>( R^2 ) av. value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \text{en} )</td>
<td>B</td>
<td>a</td>
</tr>
<tr>
<td>0</td>
<td>7.28</td>
<td>0.78</td>
</tr>
<tr>
<td>20</td>
<td>6.54</td>
<td>0.82</td>
</tr>
<tr>
<td>40</td>
<td>7.61</td>
<td>0.85</td>
</tr>
<tr>
<td>60</td>
<td>4.15</td>
<td>0.75</td>
</tr>
<tr>
<td>80</td>
<td>8.35</td>
<td>0.49</td>
</tr>
<tr>
<td>90</td>
<td>7.51</td>
<td>0.70</td>
</tr>
</tbody>
</table>

5. CONCLUSIONS

(i) The influence of intermediate principal stress has been established on strength and modulus behaviour of rock mass. The \( o_2 \) enhance the strength and modulus values and are dependent upon joint configuration. The enhancement is well confirmed by increase in fracture dip on \( a_2 \) faces of specimens.

(ii) The initiation of dilatancy is influenced by intermediate principal stress \( c_2 \) and dilatancy stress increases with increase of \( a_2 \).

(iii) The different failure modes observed are shearing of intact blocks and joints. The sliding and dilation along joints mixed with shearing is also seen in some specimens at lower confinements.

(iv) The suitable strength criteria have been evolved for jointed rock mass in both triaxial and true-triaxial stress conditions.

6. REFERENCES


