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Deformational Mechanisms of a Rock Mass

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SUMMARY In a rock mass if the mechanisms of slip, rotation, and material failure do not occur then the deformational response of the rock mass is represented as an anisotropic composite material. The parameters defining the constitutive relation of the rock mass are given in terms of the joint and intact or rock material volume proportions and physical characteristics. By considering the individual deformation modes and joint orientations the compliance for the rock mass is determined. Aspects of anisotropy and rock bolt reinforcement are then considered. Subsequent to this the mechanisms of slip, rotation, and material failure are considered and the nature of progressive failure introduced. Aspects of stability and instability related to stress redistribution are also considered.

1 INTRODUCTION

By considering the deformational process in an ideal blocky mass the various components of deformation making up the resultant deformation give an appreciation of how and to what degree these deformations effect the stress redistributions. These stress redistributions in turn create the potential for the blocky mass to either stiffen or soften in relation to further deformation. That is, the load deformational response may harden or soften dependent on the constraints related to the stress redistribution. Once the mechanisms related to the components of deformation of the blocky mass are understood it then becomes a reasonable exercise to effectively control the stiffness and consequent strength of the blocky mass.

In order to define the deformational response of a rocky mass a constitutive relation is required. If the mechanisms of slip, rotation, and material failure do not occur in the rock mass it is reasonable to represent the rock mass as an anisotropic composite material which is used as the constitutive relation in applying pseudo-elastic theory to assess the deformational response of the rock mass. The individual deformation modes of the anisotropic rock mass are examined and the analogous assumptions used in infinitesimal elasticity and the anisotropic pseudo-elastic rock mass are considered. It is shown that the parameters defining the anisotropic behaviour of the rock mass are definable in terms of the intact and joint material properties with associated relative proportional volumes and orientations.

Rock mass anisotropies are considered in relation to various numbers of joint sets with the same physical characteristics but different orientations. It is shown that as the joint sets are increased in number the mass stiffness is reduced but the degree of anisotropy is reduced to a stage where the rock mass approaches isotropy.

When the mechanisms of slip rotation and or material failure occur, the resultant deformations due to these mechanisms are superimposed on the deformations due to the deformational response of the anisotropic pseudo elastic rock mass. These mechanistic deformations when they occur require the induced or actual stress conditions arising from the deformational response of the anisotropic rock mass to exceed the strength characteristics of the joint sets and or rock mass. The strength characteristics are defined by the joint and material strengths plus the resultant stress distributions on the joints and blocks making up the rock mass.

The progressive nature of stability or instability in a rock mass is considered in relation to the mechanistic deformations such as slip, rotation, and material behaviour. If a sufficient number of these mechanistic deformations occur a collapse mechanism develops and instability of the blocky mass ensues.

2 DEFINITION OF ROCK MASS MODULUS

For a parallel set of joints in a rock mass it can be shown, Chappell and Bosler (1983a), with respect to deformation that the effect of these joints can be represented as a single joint, Fig. 1. This lumping together of the joint set is not only useful for determining moduli relationships but is also useful in representing a closely spaced joint set as a sparse jointed set in the related computer model. Initially this device is used here to develop moduli relationships.

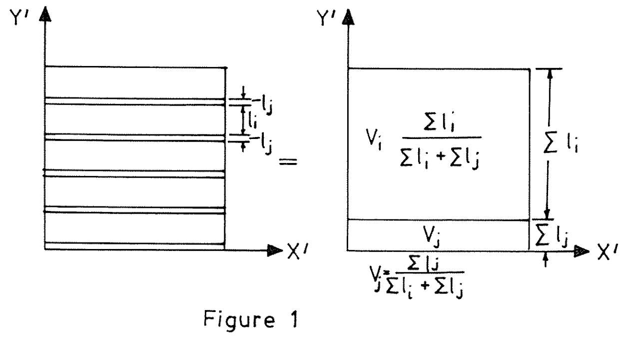


Figure 1

If the deformational mechanisms in a rock mass such as slip, rotation and material failure do not occur then it is appropriate to represent this rock mass as an anisotropic composite material, Chappell loc cit. From this the deformational response of the rock mass is determined by using pseudo elastic theory. The deformational moduli making up the composite modulus is made up of six deformation modes Fig. 2 of the intact rock and joint material. All these deformations and hence composite moduli are definable in terms of the intact and material moduli, Chappell loc cit.

For example

$$\frac{1}{E_L} = \frac{V_i}{E_i} + \frac{V_j}{E_j}$$

$$E_u = V_i E_i + V_j E_j$$

$$\nu_{x'y'} = V_i \nu_i + V_j \nu_j$$

$$\nu_{y'x'} = \nu_{x'y'} \frac{E_L}{E_u}$$

$$\frac{1}{G_{y'x'}} = \frac{V_i}{G_i} + \frac{V_j}{G_j}$$

$$G_{x'y'} = V_i G_i + V_j G_j$$

where V_i and V_j are the volume proportions of the intact and joint materials respectively.

E, G and ν are Young's modulus, Shear modulus and Poisson's ratio respectively and the suffixes i,j refer to intact and joint material and x' y' are the local co-ordinates, Fig. 2.

$$\frac{1}{E_{my}} = \frac{\sin^4\theta}{E_{uy'}} + \frac{\cos^4\theta}{E_{Ly'}} - \left(\frac{\nu_{y'x'}}{E_{Ly'}} + \frac{\nu_{x'y'}}{E_{uy'}}\right)\sin^2\theta\cos^2\theta + \left(\frac{1}{G_{mx'y'}} + \frac{1}{G_{my'x'}}\right)\sin^2\theta\cos^2\theta \cdots\cdots(1)$$

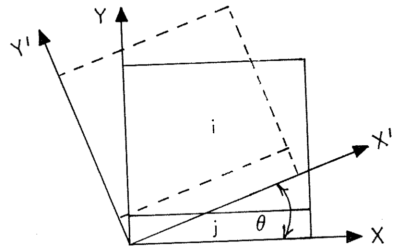


Figure 3

If as in infinitesimal elasticity only simple shear occurs then $\frac{1}{G_{mx'y'}} = 0$ in Fig 2(d) which is reasonable in this instance because the shear stiffness of the rock material is much greater than of the joint material which in turn generally has a much smaller proportional volume than that of the intact material. In addition the relation

$$\nu_{y'x'} = \nu_{x'y'} \frac{E_L}{E_u}$$

exists, Chappell loc cit.

With these relations substituted into equation (1) the relation

$$\frac{1}{E_{my}} = \frac{\sin^4\theta}{E_{uy'}} + \frac{\cos^4\theta}{E_{Ly'}} + \left(\frac{1}{G_{mx'y'}} - \frac{2\nu_{x'y'}}{E_{uy'}}\right)\sin^2\theta\cos^2\theta \ldots\ldots(2)$$

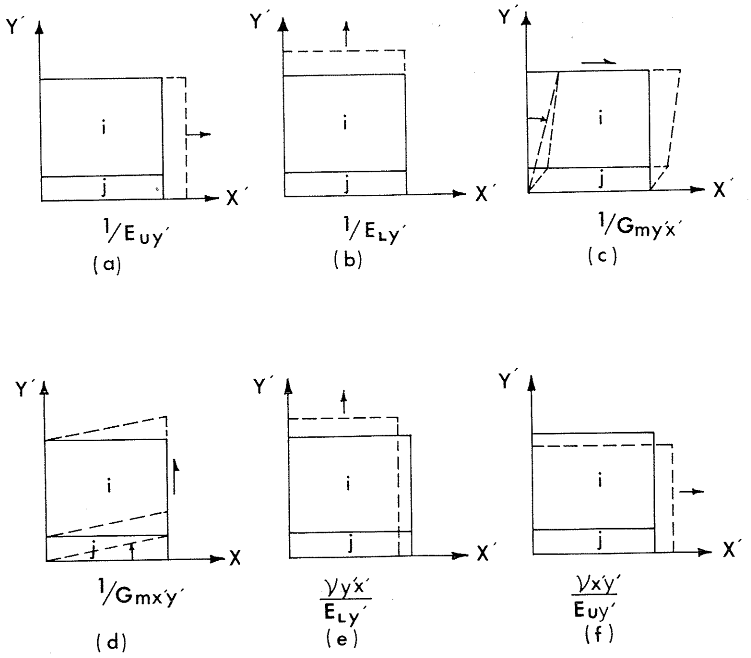


Figure 2

These composite moduli are defined in relation to the intact and joint material in local co-ordinates and are readily transformed into global co-ordinates Fig. 3. The resultant compliance in the y direction when the stress σ_y is acting and $\sigma_x = 0$ is

results, which is Young's modulus in the y direction for a two dimensional orthotropic material, Jaeger and Cook (1979).

If the rock mass is loaded in the x and y direction, Fig. 4 the rock mass compliance is

$$\frac{1}{E_{my}} = \frac{\cos^4\theta}{E_{Ly'}} + \frac{\sin^4\theta}{E_{uy'}} + \left(\frac{1}{G_{mx'y'}} - \frac{2\,\nu_{x'y'}}{E_{uy'}}\right)\sin^2\theta\cos^2\theta$$

$$+ \frac{\sigma_x}{\sigma_y}\left\{\left(\frac{1}{E_{uy'}} + \frac{1}{E_{Ly'}} - \frac{1}{G_{mx'y'}}\right)\sin^2\theta\cos^2\theta\right.$$

$$\left. - \frac{\nu_{x'y'}}{E_{uy'}}\left(\sin^4\theta + \cos^4\theta\right)\right\} \text{..........(3)}$$

It is seen here that the compliance is stress dependent and very much a function of joint orientation.

By using the principle of superposition joint sets are readily combined to give a rock mass with multiple joint sets.

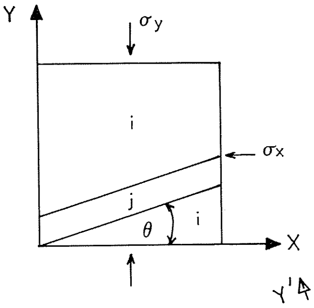


Figure 4

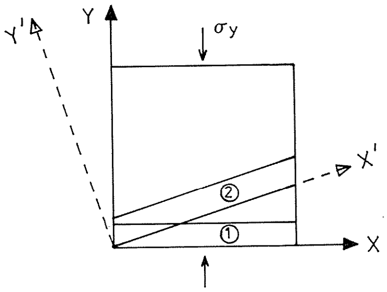


Figure 5

For example if as in Fig. 5 joint set (1) is parallel to the x axis and joint set (2) is inclined at an angle θ to the x axis and if $\sigma_x = 0$ then

$$\frac{1}{E_{my}} = \frac{1}{E_{L(1)}} + \frac{\sin^4\theta}{E_{U(2)}} + \frac{\cos^4\theta}{E_{L(2)}} + \left(\frac{1}{G_{m(2)}} - \frac{2\nu_{x'y'}}{E_{U(2)}}\right)\sin^2\theta\cos^2\theta \text{..........(4)}$$

Where $E_{L(1)}$ and $E_{L(2)}$ are as defined above and (1) and (2) refer to joint sets (1) and (2) respectively. Combinations of joint sets are readily obtained by the above process.

At this stage no mechanism of deformation such as slip, rotation and or material failure has been incorporated into the rock mass deformational response.

3 NUMERICAL MODEL REPRESENTATION OF ROCK MASS

By measuring the attitudes of individual joints, faults, and defects in a rock mass and subsequently evaluating this data with stereographic plotting the statistical representation of the joint sets relative to the attitudes are determined. The frequency or spacing of these joint sets is evaluated from structural zone assessments and histogram analyses. It is fully realised from experience that rock mass joint evaluation especially in relation to factors such as spacing of joints and their continuity are difficult to assess. It is nevertheless possible, Chappell and Klenowski (1983b), to obtain and evaluate representative numerical models of very complex rock masses using the above approach.

One particular difficulty which is effectively overcome in the numerical model of the rock mass is to lump together a specific number of joints so that the spacing of the joints are now further apart and the joints thicker, Fig. 6.

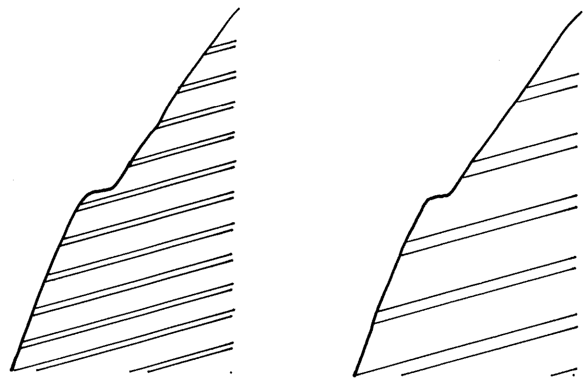


Figure 6 *X - SECTION SLOPE*

This approach in pseudo elasticity is valid as long as the mechanisms of slip, rotation and or material failure do not occur and the neglect of

$$\frac{1}{G_{mx'y'}} \quad \text{deformation mode}$$

and the relation $\nu_{y'x'} = \nu_{x'y'}\dfrac{E_L}{E_U}$

is reasonable.

In many instances the only way to represent a closely jointed rock mass as a numerical model is to lump some of the joints together to obtain a sparsley jointed numerical model.

The lumping together of some of the joints of specific joint sets does not change the magnitude of the anisotropic moduli nor their directional values. Consequently the pseudo elastic representation of the rock mass as a numerical model is reasonably valid. This rock mass representation is a function of certain criteria stated above which in the main are a function of stress gradients within the rock mass. That is, if a significant stress gradient exists then the likelihood of slip, rotation, tension and or material failure is more likely to occur. In addition the lumping of joints together has a dimensional constraint of the dimensional characteristics of the stress gradient imposed on the joint spacing especially if one of the additional deformational mechanisms is imminent.

This problem is partly overcome by limiting the number of joints lumped together in zones where stress gradients are prevalent.

3.1 Rock Mass Anisotropy

Fig. 5 and equation (4) depict and indicate how joint sets are superimposed on the rock mass to give the resultant compliance of the rock mass.

For example, if compliance is required in the y direction, and joint sets (1) and (2) are inclined at α and $(\alpha + \beta)$ to the x axis, Fig. 7, then by superposition

$$\frac{1}{E_{my}} = \frac{\sin^4\alpha}{E_{u(1)}} + \frac{\cos^4\alpha}{E_{L(1)}} + \left(\frac{1}{G_{m(1)}} - \frac{2\nu_{x'y'1}}{E_{u(1)}}\right)\sin^2\alpha\cos^2\alpha$$

$$+ \frac{\sin^4(\alpha+\beta)}{E_{u(2)}} + \frac{\cos^4(\alpha+\beta)}{E_{L(2)}}$$

$$+ \left(\frac{1}{G_{m(2)}} - \frac{2\nu_{x'y'(2)}}{E_{u(2)}}\right)\sin^2(\alpha+\beta)\cos^2(\alpha+\beta) \quad \text{.......(5)}$$

$$\frac{1}{E_{L(1)}} = \frac{V_i}{E_{i(1)}} + \frac{V_j}{E_{j(1)}}$$

$$E_{u(1)} = V_{i1}E_{i(1)} + V_{j1}E_{j(1)}$$

$$\frac{1}{E_{L(2)}} = \frac{V_{i(2)}}{E_{i(2)}} + \frac{V_{j(2)}}{E_{j(2)}}$$

$$E_{u(2)} = V_{i(2)}E_{i(2)} + V_{j(2)}E_{j(2)}$$

$G_{m(1)}$ and $\nu_{x'y'(1)}$ are shear modulus and Poisson's ratio respectively related to joint set (1) and $G_{m(2)}$ and $\nu_{x'y'(2)}$ are shear modulus and Poisson's ratio respectively related to joint set (2).

If the modulus of rock material E_i = 20 GPa and proportionate volume V_i = 0.95 and modulus of joint material E_j = 0.2 GPa and proportionate volume V_j = 0.05 and mass shear modulus G_{mi} = 1.2 GPa and Poisson's ratio $\nu_{x'y'}$ = 0.4 and the joint sets (1) and (2) are equally dimensioned so that there is no anisotropy due to differences of joint dimensions Chappell and Bosler (1983).

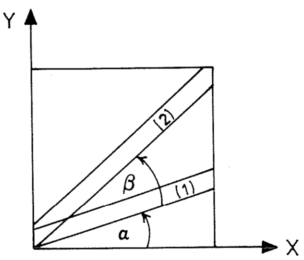


Figure 7

The moduli EL and Eu are 3.9 GPa and 19 GPa respectively. The resultant variation of anisotropy, with direction when neglecting shear modulus and Poisson's ratio Fig. 8, namely curve (1), is different to the same derived curve when including the effects of shear modulus and Poisson's ratio, Fig. 8 curve (2).

Fig. 9 shows the effects of two sets of curves when spaced at 30^o, 45^o and 60^o on the polar diagrams (1), (2) and (3) respectively. Curve (4) shows the resultant stiffnesses when 4 sets of curves are superimposed on the rock mass. These joints are orientated at 30^o, 15^o and 15^o to each other as shown in Fig. 9(b). With an increase in the number of joint sets the resultant polar stiffness diagrams show that rock mass stiffness moduli are reduced and the anisotropy is not as pronounced as for the lesser number of joint sets in the rock mass. When 5 joint sets occur curve (5), Fig. 9 the stiffness is reduced and the material approaches isotropy.

3.2 Rock Mass Reinforcement

It is enlightening to look at some aspects of rock reinforcement like rock bolts in rock mass stabilisation, Chappell and Klenowski (1983b).

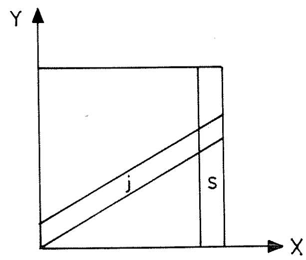
If the same rock mass considered above is rock bolted with a grouted rock bolt with an equivalent modulus of 176 GPa and proportionate volume of rock bolt Vs = 0.002 the upper stiffness modulus is 20.3 GPa and the lower stiffness modulus is 20 GPa. The shear mode now is more like that defined in Fig. 2(d) rather than Fig. 2(c). That is with rock bolts the rock mass shear modulus is 8.14 GPa rather than the 1.2 GPa related to the shear mode of Fig. 2(d).

If now the joint set plus the set of reinforcement as depicted schematically in Fig. 10 is considered the following equation

$$\frac{1}{E_{my}} = \frac{\sin^4\alpha}{E_{u(1)}} + \frac{\cos^4\alpha}{E_{L(1)}} + \left(\frac{1}{G_{m(s)}} - \frac{2\nu_{x'y'(s)}}{E_{u(s)}}\right)\sin^2\alpha\cos^2\alpha + \frac{1}{E_{u(s)}} \quad \text{..........(6)}$$

results.

For the unreinforced rock mass $\left(\frac{1}{G_m} - \frac{2\nu_{x'y'}}{E_u}\right)$ is $\left(\frac{1}{1\cdot2} - \frac{2 \times 0\cdot4}{19}\right)$ while for the reinforced rock mass this is $\left(\frac{1}{8\cdot14} - \frac{2 \times 0\cdot2}{20\cdot3}\right)$.



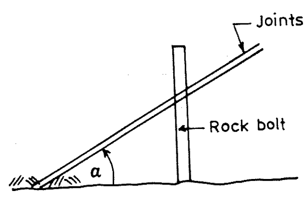


Figure 10

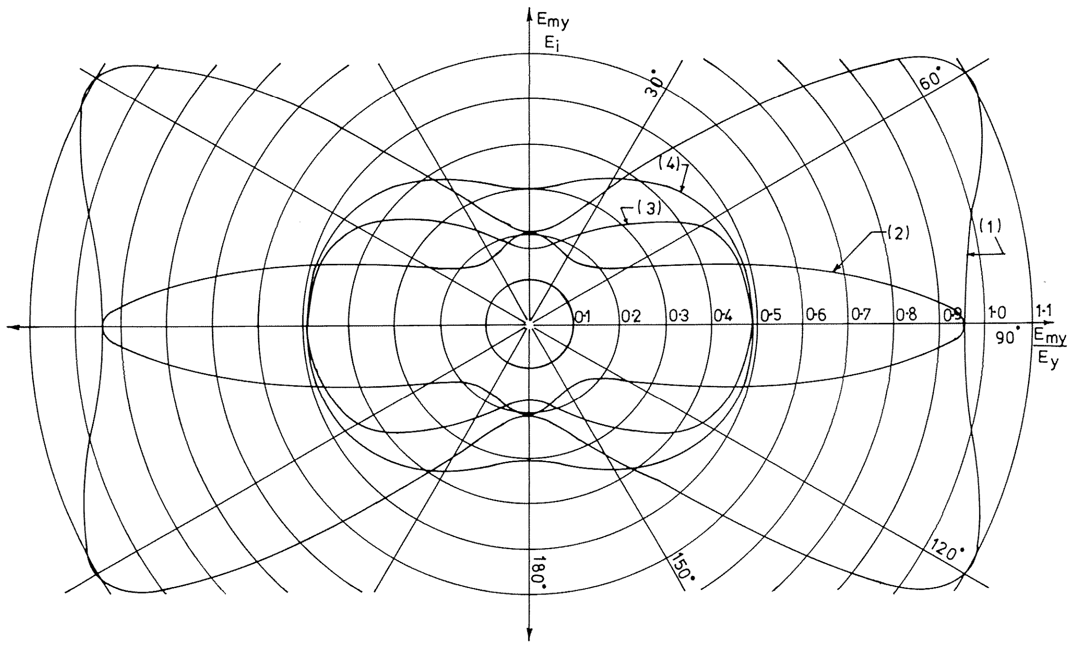


Figure 8 Variation of $\frac{E_{my}}{E_i}$ with single joint set orientation Curve(1) without POISSON's ratio

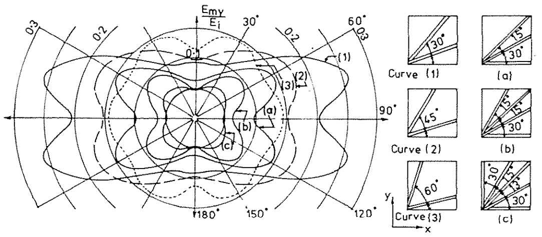


Figure 9 Variation of $\frac{E_{my}}{E_i}$ with orientation of JOINT SETS

Using equation (6) the anisotropy of the rock mass with the unstressed rock bolts is given in Fig. 8 by curve (3). As this bolt has not been pre-stressed but is grouted it is generally called a passive bolt or dowel.

If the bolt is pre-stressed the joints tending to be parallel to the rock surface close up and the joint stiffnesses are increased Chappell and Maurice (1980). If the parameters after pre-stressing are such that the proportionate joint volume V_j = 0.04 and joint stiffness E_j = 0.6 GPa with the other parameters the same as before, curve (4) in Fig. 8 results. It is apparent that pre-stressing increases some directional stiffnesses and therefore reduces the anisotropy of the rock mass.

4 MECHANISMS OF DEFORMATION IN THE ROCK MASS

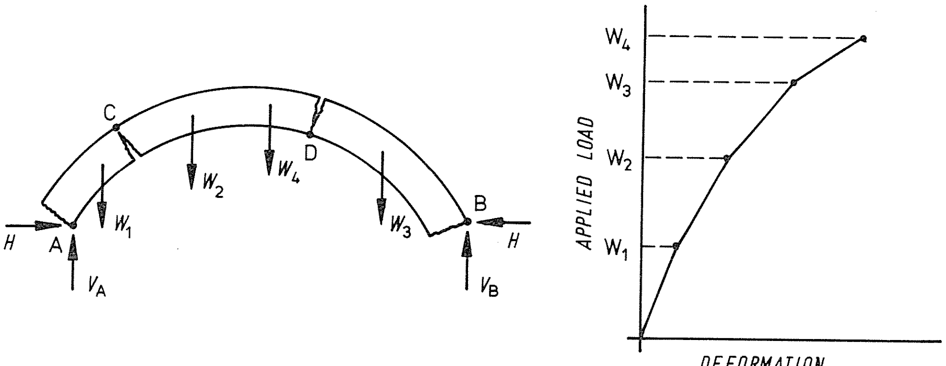
For the mechanism of slip rotation or material failure to occur the stress environment must be propitious and the constraints so disposed as to allow the kinematic movements to occur. These mechanistic deformations are added on to the deformations caused from the response of an anistropic pseudo elastic rock mass. The result of this is that the compliance of the rock mass is increased. An example of this is if an arch of which its load is gradually increased starts to crack, that is a hinge is formed, the compliance is increased, Fig. 11. Pippard and Baker (1943). The same effect of increasing compliance and increasing loads is evident when loading block models, Chappell (1979), and slip between the blocks occur.

An important effect on deformation which has not been mentioned up to now is the joints or defects which cannot sustain tension. This particular phenomenon does not effect the rock mass by causing load redistributions from geometry changes but causes load redistributions from altering or creating boundary conditions within the rock mass. The mechanisms of slip and rotation cause stress redistributions which in the main are derived from geometry changes. Failure of the material seems to have an effect in between that of a no tension characteristic and that of a slip and or rotation characteristic. It could be that failure of the material incorporates all features of a no tension, slip and or rotation characteristic of a blocky rock material. A no tension material is considered as a failure criterion and redistribution of stress determined by an iterative process.

Each of the mechanisms is considered separately here whereas in reality they are very much inter-related.

4.1 Slip

Slip along a joint is generally defined by a Mohr-Coulomb strength criterion with the allowances for joint dilation and roughnesses incorporated into the magnitude of friction angle. Constraints on dilation are allowed for by factoring the apparent cohesion. This strength criterion is superimposed



DEVELOPMENT OF VIRTUAL HINGES

Figure 11

DEFORMATIONAL RESPONSE

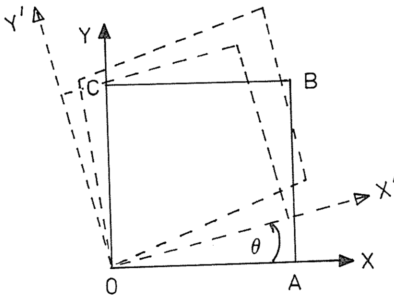


Figure 12

on the consitutive relation on which the additional deformation is also superimposed. The iterative process redistributes the unbalanced loads until equilibrium plus the failure criteria are satisfied.

There are some workers, Gerrard (1983) who incorporates the joint roughnesses and joint continuity characteristics into the compliance matrix of the rock mass. The measurement of joint roughness on the micro meso and macro scale is difficult, Chappell (1984) and at present it is best incorporated into a failure criterion.

For the moment it is best when considering rock mass deformational response as opposed to rock mass failure to consider joint roughness in the strength parameters of the joints rather than in the compliance matrix. The rationale for this is because of the varying sizes of the roughnesses the failure of these roughnesses is taking place at all stages of deformation and is more likely to be controlled by progressive failure of these roughnesses along the joint rather than material deformation of the rock mass.

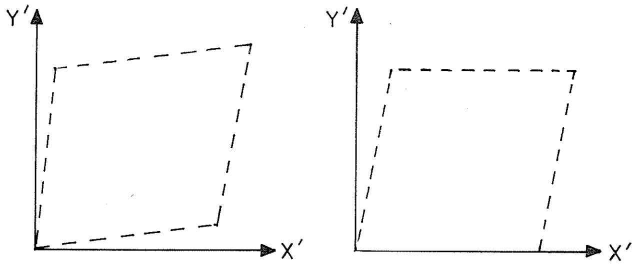


Figure 13

4.2 Rotation

When deformations are finite or excessively inhomogeneous, that is not uniform over a short distance, the aspects of geometry controlling the definition of strain and the equilibrium of stresses require careful consideration. Biot (1965) attacks this problem by considering incremental elasticity which is infinitesimal elasticity on which an allowance for incremental rotation of the reference axes is superimposed, Fig. 12. In infinitesimal elasticity the assumption of homogeneity transforms pure shear into simple shear Fig. 13. In an analogous manner the shear deformations in a composite material are assumed to be made up of the shear deformation mode depicted by Fig. 2(c) while that of Fig. 2(d), namely

$$\frac{1}{G_{mx'y'}}$$

is neglected.

In a jointed rock mass the joints generally and significantly increase the compliance and hence the deformation of the rock mass. If the stress is homogeneous that is there is no significant stress gradient between the blocks, Fig. 14(a) the relative slip between the blocks is minimal and the assumption of homogeneity and neglect of the shear compliance Fig. 2(d) is reasonable. When however the stress gradient is significant, two aspects of deformation are instigated namely the tendency of relative slip between the blocks and rotation of a block Fig. 14(b). If this tendency from the distribution of stresses exceeds the strength of the joints or block constraints slip and or rotation occurs. It is generally near or at a free surface boundary that rotation will most readily occur. This latter geometrical rotation is different to the rotation which though also geometrical is required to transform pure shear to simple shear, Fig. 13.

Relative slip along a joint and rotation redistribute the loads due in the main to geometry changes and rock joint stiffness variations. As the main part of deformation in a rock mass takes place in the joints, Chappell (1973), it is reasonable to model the compliance of the rock mass as a composite material where the deformation takes place in the joints. That is the rock mass is represented as a rigid block system and the joint properties are composite anisotropic pseudo elastic parameters derived from the intact and joint material properties.

If the blocks do not slip or rotate relative to one another then the deformational response of the blocky mass is evaluated using pseudo elasticity with the rock mass represented as an anisotropic material. It must be stressed however that this is valid only if the load is either monotonically increasing or decreasing as the compliances though representing a linear relation are very different for the loading and unloading situtation Chappell, (1979).

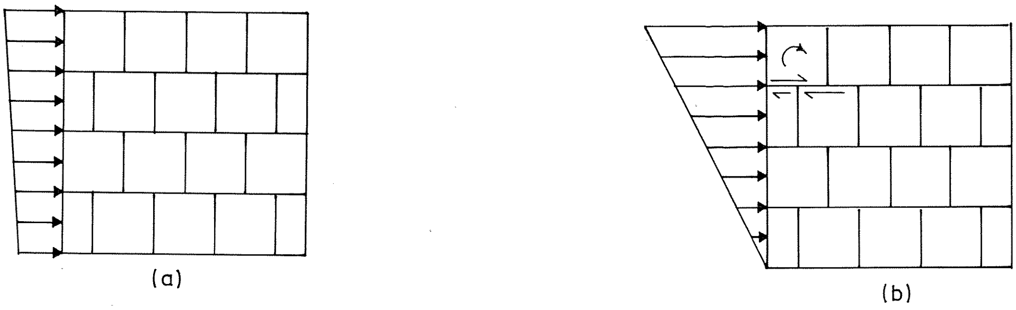


Figure 14

Once the deformations of the joints have been assessed the stresses and hence the loads and resultant moments on each block are determined. Moment equilibrium for each block is required and to achieve this further deformation of the block system mainly in the joints occur. From this the stresses in the joints require re-adjustment, however because no bodily slip or rotation has taken place this operation converges very quickly.

This method of determining the deformational response of a complex rock mass Chappell and Klenowski (1983b) gives very good results. It must be stressed that up to this stage no bodily slip or rotation between the blocks has taken place. When this does happen the loads re-distribute and further slip or rotation may be induced, this is a form of progressive failure. Up to this stage the lumping together of joint sets and rock blocks to give a more readily workable numerical model of the structural geological model has not effected the deformational response. However once slip and or rotation occurs the compliance of the rock mass changes Chappell (1983c) and a relation between moment on the block and rotation of the block is required and a relation between shear force and slip is also required. If these additional deformational modes are superimposed on the deformational modes of the anisotropic pseudo-elastic material the resultant deformations are again realistic.

A situation can and does arise when the load redistributions cause further progressive slip and or rotation and this creates a sufficient number of hinges, Chappell (1983c), to form a collapse mechanism. That is instability ensues. When these slip and rotation mechanism occur with the resultant redistribution of load the lumping together of joint sets and rock blocks is no longer valid because the lumped material impairs the further development of slip and or rotation mechanisms due in the main to kinematic constraints.

4.3 Material Failure

If the interactive stress distribution on a specific block are uniform and equal, ie stress ratio

$$\sigma_y/\sigma_x = 1$$

compressive failure of the block does not occur Fig. 15(a). When however, the stress ratio

$$\sigma_y/\sigma_x$$

increases beyond 3 and 4 local failure of the block can and does occur. In turn when the interactive stress redistributions cause stress gradients across the boundaries of the block, Fig. 15(c), tensile stresses readily occur within the blocks and a failure environment is opportune. The deformation mechanisms which readily give a stress gradient across the boundaries of the blocks is rotation and the associated moments, Chappell (1979). These rotations are generally facilitated at the boundaries of the rock mass where constraints are minimal.

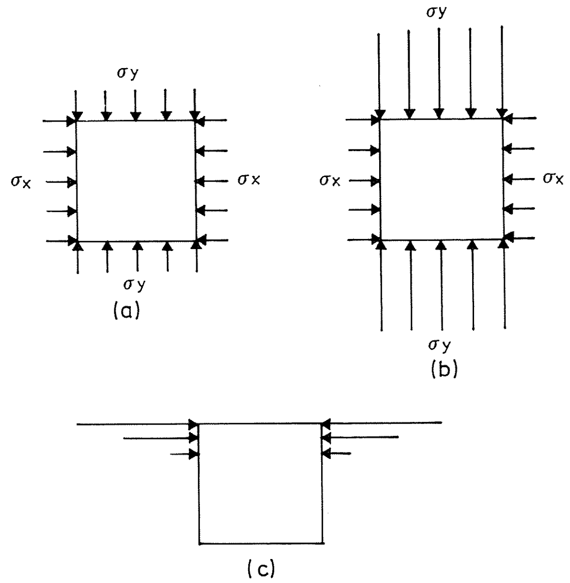


Figure 15

Here again the lumping together of joints of a joint set and associated blocks is not valid as when considering the deformational response of a rock mass. In addition, when a block fails a new joint or joints are introduced and the compliance of the rock mass generally increases. That is a hinge is formed, Chappell (1983c). The energy released from the failure of material and the increased compliance redistributes the loads and further failure may or may not occur dependant on a number of factors such as stress environment, material type and constraints. This is, in effect, stable or unstable progressive failure and is dependant on the difference between the required and actual hinges formed to create a collapse mechanism, Chappell loc cit. If the required hinges to form a collapse mechanism are greater than the actual hinges formed then stability ensues and if not instability results.

5 CONCLUSIONS

Component deformations in a jointed or blocky rock mass are made up of deformations of the rock and joint material on which are superimposed the deformations of mechanisms. These mechanisms are slip, rotation and material failure plus the condition of a no tension stress or tensile strength across the joints.

If the deformation mechanisms of slip, rotation, and material failure with the restriction on tensile conditions do not occur then the deformational response of the rock mass is best represented as an anisotropic composite material on which pseudo-elastic theory is readily applied. The compliance of the rock mass is defined in terms of the compliances and relative volumes of the constituent rock and joint material. This compliance is besides the rock and joint properties a function of joint set orientations and the stress environment. When numerically modelling the rock mass it is possible to lump a number of parallel joints of a joint set together so that a closely spaced laminated joint set is represented by a widely spaced laminated joint set. The deformational response of the rock mass is not greatly effected by this device as long as the deformational mechanisms of slip, rotation, and or material failure do not occur. This implies that the stress gradients within the rock mass are minimal.

If the mechanisms of slip, rotation and material failure do occur then these deformations must be superimposed on those deformations caused by the pseudo elastic rock mass response. These mechanisms of deformation initially are controlled by the induced stress system and strength of the joints. Once the strength of the joint is exceeded slip occurs and consequent rotation may or may not occur depending on the constraints imposed on individual blocks of rock. It appears that the place for rotation of a block to most readily occur is at the unconstrained boundaries of the rock mass. Slip with the associated rotation cause geometry changes and the related redistribution of stresses. This in turn causes stress gradients within the block mass to develop especially at the interfaces of the rock blocks. From this tensile stresses within the rock blocks readily occur with the consequent likelihood of material failure.

Once material failure of the rock block occurs the compliance of the rock mass increases and there is a further redistribution of stress. The degree and magnitude of stress redistribution is very much dependant upon the amount of energy released from the material block failure. This is in the nature of progressive failure.

When the mechanisms of slip, rotation and material failure occur with the restrictions of tensile stress and strength the lumping together of joint sets requires careful consideration. This is because the whole nature of progressive failure is the interaction of the joint and rock material with the mechanisms of slip rotation and or material failure included.

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