The stability of blocks with soil filling the joints

La stabilité des blocs avec un sol de remplissage des joints

A. C. MATOS, Faculdade de Engenharia, Universidade do Porto, Portugal
J. B. MARTINS, Universidade do Minho, Braga, Portugal
P. PIMENTA, Eng. Civil, Universidade do Porto, Portugal

SYNOPSIS A behaviour model and its calculation algoritm are presented for the determination of internal forces and deformation of a set of rigid blocks with soil filling the joints. For small deformations the soil is assumed as an elastic material although with a shear modulus very much lower than the Young modulus. For large deformations the Mohr-Coulomb theory of plasticity is assumed. The reduction of contact area in the joints is also taken into account. The behaviour of the whole mass depends not only of the properties of the filling material but also of the orientation of the joints system. The deformation of the model can be followed until rupture.

INTRODUCTION

The deformation of an homogeneous soil or rock mass is easily analysed by the usual finite element methods, which give the stresses and strains at points inside the elements (the Gauss points) through equilibrium equations stated at the vertices of the network. However, in the most common case of a geological formation with soil filling a system of joints, the deformation and internal forces should be taken at the interfaces of the elements (blocks) and the external equilibrium, including moments, should be taken for the blocks themselves. In this way there will be no discontinuity of stresses at the joints, as it happens in the finite elements solutions. In the model now proposed we consider the blocks connected by an "elastic" layer of soil with a given thickness obtained from the field, and the external forces including self-weight and external moments applied at the center of gravity of each block.

MODELLING THE JOINT BEHAVIOUR

At the joints three kinds of constitutive relationships are considered. First we refer the relationship between shear stress τ and the distortion γ, which depends on the average normal stress $\bar{\sigma}$ ($\tau = F_1(\gamma, \bar{\sigma})$) and can be obtained either by laboratory and field tests (N. Barton, 1974, Norwegian Geotecnical Institute) or by calculation from the geometric and shear characteristics of the joints. The geometric characteristics are the thickness of the joint H, the average roughness amplitude a and the average inclination i_o of the asperities. The shear characteristics are: the friction angle ϕ and cohesion c obtained from "pick" points P (Fig. (1.a)) in the τ, γ curves and residual angle of friction ϕ_r and cohesion c_r obtained from the points of residual strength Q of the same curves. Plotting both Coulomb's law obtained from (c, ϕ) and from (c_r, ϕ_r) we obtain the transition value for the normal stress σ_A, which is given by

$$\sigma_A = (c_r - c)/(\tan\phi - \tan\phi_r) \qquad (1)$$

The beginning of the residual state of shear stress is defined by the shear strain $\bar{\bar{\gamma}}$ which is related to the normal average stress $\bar{\sigma}$. For low levels of $\bar{\sigma}$ during the shear deformation corresponding to the residual state of stress, the contact points in the joints, climb along the asperities and, therefore, the joint dilates. Experience shows that in this case

$$\bar{\bar{\gamma}} = a \tan i_o (1 - \bar{\sigma}/\sigma_T)^4 \qquad (\bar{\sigma} < \sigma_T) \qquad (2)$$

where σ_T is the average transition value of

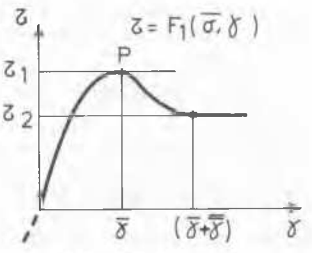

Fig. (1.a)

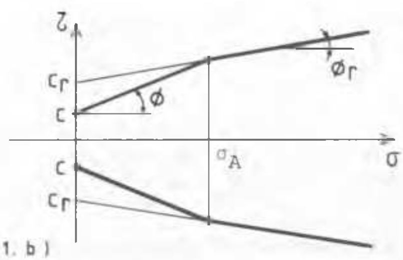

Fig. (1.b)

the normal stress. For values of the average normal stress $\bar{\sigma} > \sigma_T$, the joint does not dilate during plastic (residual) deformation. In this case the normal stress at the joint is such that the rock asperities themselves will be sheared and there will be no climbing of the contact points.

In practice the values of σ_T and σ_A above can be assumed equal.

The function $\tau = F_1(\bar{\sigma}, \gamma)$ in Fig. (1.a) is the following and can be obtained from a simple model (A.C. Matos, internal note, 1984):

$$\tau = \frac{\bar{\sigma}\tan\phi + c}{\left(1 - \frac{k}{\sqrt{1+k^2}}\right)} \left[1 - \frac{\sqrt{H^2 + (m\bar{\sigma} - \gamma)^2}}{m\bar{\sigma}\sqrt{1+k^2}} \right] \tag{3}$$

where $k = H/\bar{\gamma}$, H being the thickness of the joint; c, ϕ are the strength parameters; $\bar{\sigma}$ is the average normal stress: $\bar{\sigma} = N/S_e$ (N=normal force and S_e efective contact length); m is an experimental parameter that relates the pick shear strain $\bar{\gamma}$ and $\bar{\sigma}$. It is assumed that

$$\bar{\gamma} = m\,\bar{\sigma} \tag{4}$$

γ is the current shear strain. It should be noted that for $\bar{\sigma} > \sigma_A$ (Fig. 1.b) we must take $\phi = \phi_r$ and $c = c_r$. From (3) we obtain the shear modulus

$$G = \frac{\partial\tau}{\partial\gamma} = \frac{\sigma\tan\phi + c}{m\sigma(1 + k^2 - k\sqrt{k^2 - 1})} \tag{5}$$

Instead of the tangent value (5) of G, we may use the secant value

$$G = \tau_1/\bar{\gamma} = (\sigma\tan\phi + c)/(m\sigma) \tag{6}$$

Initially we take

$$G_o = \tan\phi/m \tag{7}$$

The function $M = F_2(\theta)$ in Fig. 2, depends on the contact area S_e at the joint and on the average normal stress $\bar{\sigma} = N/S_e$. For $0 < \theta < \theta_1$ there is full contact and for $\theta = \theta_2$ there will be overturning of the block.

It can be easily proved that a function which satisfies those conditions can be written as:

$$M = \frac{1}{2}\left[\ell(N + q_u \cdot \ell) - \sqrt{\frac{8(N + q_u \cdot \ell)}{9E\theta}} \right] \tag{8}$$

for

$$M > M_1 \quad (\text{or } \theta > \theta_1) \tag{9}$$

In this case there will be reduction of the contact area and it can be seen that

$$S_e = 3\,\frac{\ell}{2} - \frac{M}{N} \tag{10}$$

For full contact

$$M = \frac{\ell^3 E'}{12}\,\theta \tag{11}$$

with

$$M < M_1 \quad \text{or} \quad \theta < \theta_1 \tag{12}$$

where

$$M_1 = \frac{N\ell}{6} + \frac{q_u \ell^2}{6} \tag{13}$$

and

$$\theta_1 = M_1 \frac{12}{E'\ell^3} \tag{14}$$

q_u = unconfined undrained strength of the filling soil,

ℓ = length of the joint,

N = normal internal force and

θ = relative rotation of the blocks,

$E' = E/H$, E being the Young modulus and H the thickness of the joint.

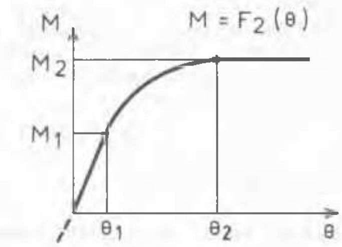

Fig. 2

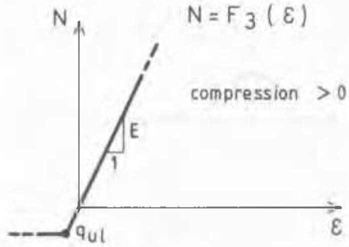

Fig. 3

Concerning the deformation normal to the joint, we shall assume for the function $N=F_3(\varepsilon)$ the following form (Fig. 3):

$$N = E\ell \cdot \varepsilon \qquad (N > -q_u \cdot \ell) \qquad (15)$$

and

$$N \simeq -q_u \cdot \ell \qquad (16)$$

for

$$\varepsilon < -q_u/E \qquad (17)$$

where N and ε are positive in compression. E is the Young modulus for normal laterally confined compression of the filling soil and $\varepsilon=\delta_N/H$ is the average normal strain, δ_N being the relative displacement of the blocks at the middle point of the joint.

ALGORITM FOR THE CALCULATION OF THE INTERNAL FORCES AND DISPLACEMENTS

For the given external loads $\bar{F}$ (self weight and others) at the center of gravity of the blocks we calculate the corresponding vector of external displacements $(\bar{d}, \bar{\theta}_e)^T$, assuming linear elasticity and initial values $E=E_O$ and $G=G_O$. With the external displacements of the blocks the internal (relative) displacements at the middle points of the joints are calculated $(\varepsilon, \gamma, \theta)$ and the initial internal forces N_O, T_O, M_O are calculated thereafter.

Putting the values of $(\varepsilon, \gamma, \theta)$ into $N=F_1(\varepsilon)$ and $M=F_3(\theta)$, we calculate the reduced contact area S_e and from there the value of average normal stress $\bar{\sigma}$ and honce the shear stress $\tau=F_2(\gamma,\bar{\sigma})$ and shear force $T=\tau . Se$. Taking the differences $M=M-M_O$ and $T=T-T_O$ and introducing them with changed sign into the block forces, after transformations to the center of gravity of each block, we can obtain a first correction. At this point we can up to date, if we wish, the values of the shear and Young modulus G and E. We return to the starting point for a new iteration.

As seen this model considers the well know fact that rotation reduces the shear strength of the joints.

COMPUTER PROGRAM AND APPLICATIONS

A computer program has been written for the above algoritm and applications have been made to two problems Figs. (4.a) and (4.b), the second of which had been solved by the finite element method with joint elements of "null" thickness. The problem consists in decreasing the values of the shear strength parameters c and $tg\phi$ of the filling soil by a factor of safety F_s and calculating the displacement of a chosen control point. For very large displacements of the control point we obtained for case of Fig. (4.a) F_s=1,86 with $\phi=\phi_r=30^O$, $c=c_r=0$, $E_O=10^8$kPa,

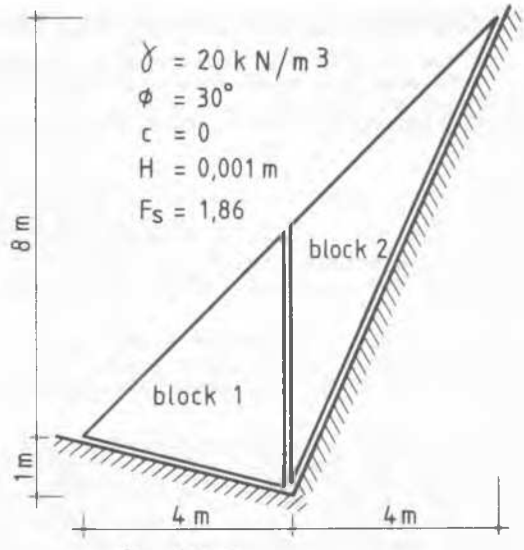

Fig. (4.a)

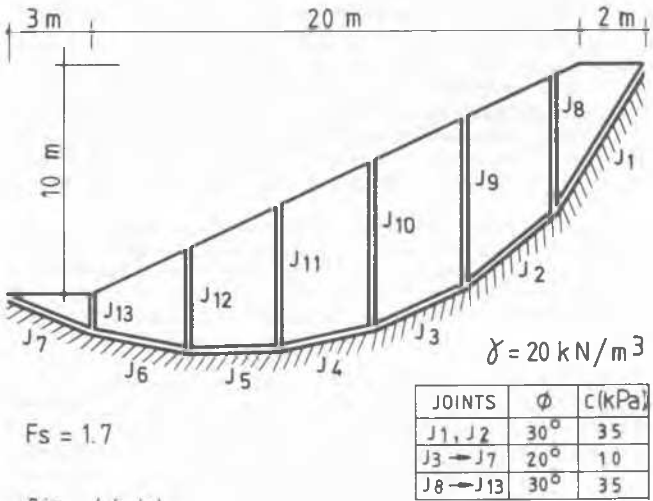

Fig. (4.b)

$G_O=10^5$kPa, $m=5,774\times10^{-6}(\text{kPa})^{-1}$, $H=0,01$ m. These later experimental values obtained by the Norwegian Geotechnical Institute.

For the second case we obtained F_s=1,7 and by the finite element method we obtained F_s=1,9. In this later case convergence difficulties arise, but not in the new method.

CONCLUSIONS

The model proposed can be advantageous on the determination of the safety factor F_s of a rock mass with filling joints. Questions like dilatance, residual strength, rotational shearing and rotational collapse can be easily considered on the analysis of real problems.

ACKNOWLEDGEMENTS

Thanks are due to ETEC Lda for computer facilities. This work has been done with NATO support (Grant nº 004/80).

REFERENCES

Barton, N. (1974). A review of shear strength of filled discontinuities in rock. Norwegian Geotechnical Institute, NR.105, Oslo.

Barton, N. (1974). Review of a new shear strength criterion for rock joints. Norwegian Geotechnical Institute, NR.105, Oslo.

Desai, C.S. (1984). Thin-layer element for interfaces and joints. Int. Journal Num. Anal. Meth. Geom., vol.8, 19-43.

Dowding, C.M. (1984). Dynamic computational analysis of openings in jointed rock. ASCE J. Geotechnical Eng., vol. 109, 1551-1566.

Goodman, R.E. (1968). A model for the mechanics of jointed rock. ASCE J. Soil Mechanics Found. Eng., 637-659.

Martins, J.B., Reis, E.B., Matos, A.C. (1981). New methods of analysis for stability of heterogeneous slopes. 10th Int. Conf. Soil Mechanics, Stockholm.