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BRITTLE FRACTURE OF HARD SOIL - SOFT ROCK MATERIALS

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ABSTRACT

Hard soil - soft rock materials tend to fail along well-defined discontinuities. However, in the common practice of geotechnical engineering, such failure is implicitly modelled as uniform behaviour by smearing, and either based on elastoplasticity or empirically determined from laboratory test results. A model is presented herein which addresses the problem as one of brittle fracture of a three-phase material, where the matric suction exerted by the pore air/water phases on the solid phase is disrupted by tensile or shear loading, or a combination of both. There is therefore the added complication that the fracture toughness of the material medium would vary according to changes in the matric suction which is brought about by the application of test loading. Furthermore, it would be necessary to predict the development of non self-similar crack extension from a sharp corner in accordance with the observed behaviour of test specimens.

Accordingly, the problem of plane strain compression testing has been analysed using a hybrid BEM based on a combination of the displacement discontinuity and fictitious stress methods. The model has, moreover, been established for confirmation against the results of laboratory testing on unsaturated kaolin clay.

KEYWORDS

Brittle fracture, hard soil - soft rock, hybrid BEM, matric suction in plane strain, corner crack, unsaturated kaolin clay.

INTRODUCTION

Brittle hard soil - soft rock is often found in geotechnical engineering works such as tunnels, slopes, etc. These soils contain fissures or cracks which are the result of mechanical, thermal and volume-change-induced stresses. As a result of gravity, earthquake or water-pressure-induced loads, these flaws can develop stress concentrations which result in the non-uniform mobilisation of strength and ultimately lead to the catastrophic failure of the soil body as they propagate. Conventional failure criteria [1] of soils may be appropriate to plastic-yield-dominant behaviour, but not, in principle, to this category of brittle fracture. In view of the existence of fissures and cracks, such soils are non-uniform and therefore not amenable to analysis by continuum mechanics alone. On the other hand, fracture mechanical theory may be used to advantage to replicate their behaviour.

The first quantitative data on the role of fissures on the strength of clay appears to have been presented by Terzaghi [2] from a study of the instability of gentle slopes in fissured clay. Such failure occurred despite the very high compressive strength of intact clay fragments. Terzaghi established that the overall strength of the fissured clay represented a fraction of the strength of the same clay without fissures. On the other hand, Bishop [3] and Skempton et al. [4] were apparently the earliest to suggest that fracture-mechanical concepts might shed light on the progressive failure of slopes made of stiff, fissured clays, although Bjerrum [5] also discussed progressive failure in terms of stress concentration at the tip of a slip surface. Saada [6] and Vallejo [7] subsequently applied the concepts of LEFM to investigate the mechanism of crack propagation in stiff clay.

A basic concept of fracture theory is that crack-like imperfections are inherent in engineering materials. These flaws act as stress raisers that can trigger fracture when subjected to critical loading. Unsaturated hard soil-soft rock materials, on the other hand, are three-phase media comprising air, water and solid. As such, the degree of saturation S of the material, and hence its matric suction (u_a - u_w), could vary as it was loaded. Thus, it would be necessary to keep track of changes in the parameters at all stages of loading, since for brittle fracture to take place, the fracture toughness which is available would depend on their ambient values. In other words, unlike the generally-accepted material behaviour of fracture mechanics, during crack development, the applied loading would not only raise the level of total stresses required to cause further crack extension, but also influence the properties of the soil-rock medium which would determine whether the crack would extend.

In the following discussion, a model will be proposed for the brittle fracture of hard soil-soft rock, which is based on the above considerations. The model will be verified by conducting plane strain biaxial compression tests on a pre-cracked specimen, and thereafter comparing the test results with those obtained by using a hybrid BEM based on a combination of the displacement discontinuity and fictitious stress methods. Furthermore, it will be shown how the development of a secondary crack may be predicted in accordance with observed behaviour.

PROPOSED MODEL

Determination of Matric Suction

The matric suction (u_a - u_w) is defined [8] as the difference between the pore air pressure u_a and pore water pressure u_w, which varies with load. It is required in order to determine the fracture toughness of the hard soil-soft rock test specimen. The pore pressures may, in turn, be deduced from their respective pore pressure parameters B_a and B_w, based on the following relationships:

$$du_a = B_a d\sigma_{ave} \tag{1}$$

and

$$du_w = B_w d\sigma_{ave} \ , \tag{2}$$

where

$$\sigma_{ave} = \frac{\sigma_1 + \sigma_3}{2} \tag{3}$$

and σ_1 and σ_3 are the major and minor principal stresses respectively. The pore pressure parameters are given by

$$B_a = \frac{R_2 R_3 - R_4}{1 - R_1 R_3} \tag{4}$$

and

$$B_w = \frac{R_2 - R_1 R_4}{1 - R_1 R_3} \ , \tag{5}$$

in which

$$R_1 = \frac{R_s - 1 - \left[(1 - S + hS)n / \left(\bar{u}_a m_{1p}^s\right)\right]}{R_s + \left(SnC_w / m_{1p}^s\right)} \ , \tag{6}$$

$$R_2 = \frac{1}{R_s + \left(SnC_w / m_{1p}^s\right)} \ , \tag{7}$$

$$R_3 = \frac{R_a}{R_a - 1 - \left[(1 - S + hS)n / \left(\bar{u}_a \left(m_{1p}^s - m_{1p}^w\right)\right)\right]} \tag{8}$$

and

$$R_4 = \frac{1}{R_a - 1 - \left[(1 - S + hS)n / \left(\bar{u}_a \left(m_{1p}^s - m_{1p}^w\right)\right)\right]} \ , \tag{9}$$

where

$$R_s = \frac{m_2^s}{m_{1p}^s} \ , \tag{10}$$

$$R_a = \frac{m_2^s - m_2^w}{m_{1p}^s - m_{1p}^w} \ , \tag{11}$$

h is the proportion of dissolved air in the water, $\bar{u}_a$ the absolute air pressure, n the porosity, C_w the water compressibility and $m_{1p}^s, m_2^s, m_{1p}^w$ *and* m_2^w the volumetric deformation coefficients which may be evaluated from the compressive indices C_t, C_m, D_t and D_m obtained from the constitutive surfaces of the hard soil-soft rock, as follows:

$$m_{1p}^s = \frac{0.435\, C_t}{(1 + e_0)(s_{ave} - u_a)_{mean}} , \tag{12}$$

$$m_2^s = \frac{0.435 C_m}{(1 + e_0)(u_a - u_w)_{mean}} , \tag{13}$$

$$m_{1p}^w = \frac{0.435 D_t G_S}{(1 + e_0)(s_{ave} - u_a)_{mean}} \tag{14}$$

and

$$m_2^w = \frac{0.435 D_m G_S}{(1 + e_0)(u_a - u_w)_{mean}} \ , \tag{15}$$

in which ($_{ave}$ - $u_a)_{mean}$ and $(u_a - u_w)_{mean}$ are the averages of the initial and final net normal stresses and matric suctions over a load increment.

Determination of Fracture Toughness

At any given stage of crack development, it is necessary to obtain an update on the value of the fracture toughness K_c, which is generally dependent on the matric suction, or alternatively the degree of saturation of the soil medium, by way of the pore size distribution index . It is noteworthy that this dependency may be established fundamentally on the basis of Griffith's analogy of the critical rate of energy release G_c and the surface tension   for glass, in which it may be shown that a relationship may be obtained between G_c, the matric suction $(u_a - u_w)$ and characteristic pore size D_p, given by

$$G_C = k \frac{(u_a - u_w) D_p}{4} , \tag{16}$$

where k is a parameter which reflects the mode of fracture. On this basis, the fracture toughness versus matric suction plot of Figure 1 has been determined by fracture testing of brittle kaolin clay specimens in the mode I of deformation.

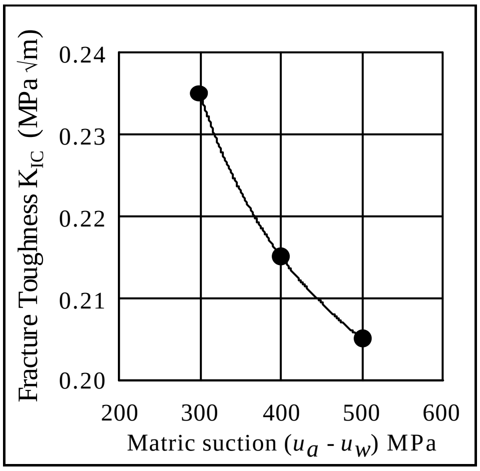

Figure 1: Fracture toughness versus matric suction.

Fracture Criteria

The fracture analysis of tensile loading of materials has been greatly aided by developments in fracture mechanics over the last 40 years or so. However, applied stresses are usually compressive rather than tensile in a geotechnical environment, and the fundamental fracture response of soil structures loaded in compression differs in a number of respects from its counterpart in tensile loading.

In the discussion which follows, the *unified model* [9] will be used as the basis of analysing how a crack would develop in this situation. Accordingly, the modes I and II stress intensity factors with respect to the generalised plane would be given by

$$K_{I\theta} = K_I \cos^3\frac{\theta}{2} - 3K_{II}\sin\frac{\theta}{2}\cos^2\frac{\theta}{2} \qquad (17)$$

and

$$K_{II\theta} = K_I\sin^2\frac{\theta}{2}\cos^2\frac{\theta}{2} + K_{II0}\cos\frac{\theta}{2}\left(1 - 3\sin^2\frac{\theta}{2}\right), \qquad (18)$$

while the criterion of fracture may be stated as

$$\frac{K_{I\theta}^2}{K_{IC}^2} + \frac{K_{II\theta}^2}{K_{IIC}^2} = 1\ . \qquad (19)$$

In mixed mode loading where compression is applied, the stress field due to K_I and K_{II} can be tensile in the vicinity of the crack tip so that fracture can occur in a manner similar to the case of tensile loading, although if $K_{IC} > 1.15\ K_{IIC}$ shear or mixed mode fracture would in principle be possible too. However, unlike the case of the stress-free crack surface due to tensile loading, under combined shear and compressive stresses, the crack tip would develop a singularity due to relative shear displacement of the adjacent crack faces. Hence, some provision would have to be made to prevent the overlap of the material medium at the interface, and friction could also play a part in the fracture of the soil.

VERIFICATION OF PROPOSED MODEL

The problem adopted for verification consisted of a plane strain specimen of brittle kaolin clay, 72mm x 72mm in plan and 36mm thick, which was initially consolidated at 200kPa and then extruded and trimmed to the required size. Thereafter, a pre-crack of length 20mm was formed centrally within the test specimen, and inclined at an angle of 45 as shown in Figure 2, following which the specimen was desaturated under a matric suction of 500 kPa by the application of cell pressure $_3$ = 550 kPa, back-pressure u_w = 50 kPa and pore-air pressure u_a = 550 kPa in a triaxial cell. Subsequently, the specimen was loaded monotonically by applying a constant rate of displacement of 0.5 mm/min under a constant cell pressure of $_3$ = 0.2N/mm^2. This rate of loading had been established from the consolidation stage to be sufficient to maintain an undrained condition in the test specimen.

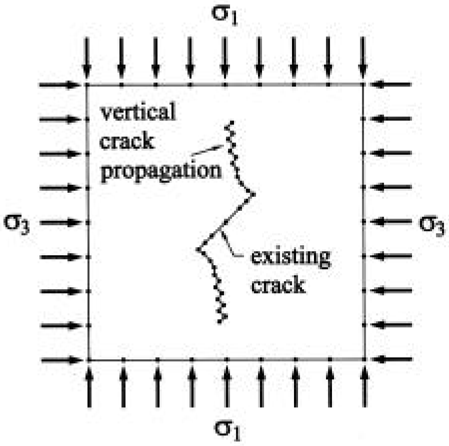

Figure 2: BE analysis of plane strain compression testing of a highly brittle soil.

During loading, the volume change of the soil skeleton, V_s, was monitored continuously by laser sensors and the axial displacement at the top of the specimen recorded automatically via a Wykeham Farrance AT2000 data-logger. Furthermore, the extension of the pre-crack was monitored in tandem with the applied loading. The loading was applied until the test specimen attained its ultimate condition.

A BE analysis was carried out on the extension of a pre-crack in the soil specimen, based on a combination of the displacement discontinuity and fictitious stress methods (Figure 2). The simulation, which was conducted over a total of 12 steps, employed the proposed soil-rock model, and was confirmed against the results of laboratory testing on unsaturated kaolin clay.

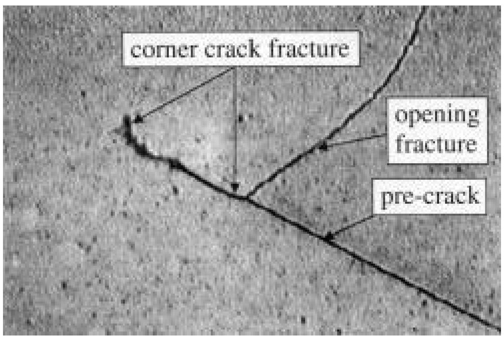

Figure 3: Opening and corner cracking of unsaturated kaolin clay.

Furthermore, it may be shown that the development of non self-similar crack extension from a sharp corner in accordance with the observed behaviour of test specimens (for example, Figure 3) may be determined from the mixed mode criterion

$$\frac{K_{aI\grave{e}}^{2}}{K_{aIC}^{2}}+\frac{K_{aII\grave{e}}^{2}}{K_{aIIC}^{2}}=1\ , \qquad (20)$$

where K_{IC} and K_{IIC} are the modes I and II fracture toughness at the corner, where the corresponding generalised stress intensity factors, K_I and K_{II}, would be given by

$$K_{\alpha I\theta} = f_{11}K_{\alpha I0} + f_{12}K_{\alpha II0} \tag{21}$$

and

$$K_{\alpha II\theta} = f_{21}K_{\alpha I0} + f_{22}K_{\alpha II0} \ , \tag{22}$$

respectively, in which

$$f_{ij} = f(\alpha, \lambda, \theta) \ , \tag{23}$$

$$K_{\alpha I0} = \lim_{\theta=0, r\to 0} [(2\pi r)^{1-\lambda} \sigma_{\theta\theta}(r, \theta)] \ , \tag{24}$$

$$K_{\alpha II0} = \lim_{\theta=0, r\to 0} [(2\pi r)^{1-\lambda} \tau_{r\theta}(r, \theta)] \ , \tag{25}$$

$$\lambda \sin(2\pi - \alpha) + \sin \lambda(2\pi - \alpha) = 0 \ , \tag{26}$$

and α the included angle of the sharp corner.

CONCLUSIONS

Present-day geotechnical models in common usage tend to view the stress-strain behaviour of soils in terms of uniform point-to-point response of the material medium, implicitly. This is reflected in the use of continuum models of elastoplasticity coupled with the effective measurement of stress-strain parameters of soil specimens, when loaded, as smeared values. In an alternative approach, an empirical fit is made to the experimental data although the constraints of uniform behaviour and smeared values still persist.

However, it is a well-observed phenomenon that discontinuities, and hence the departure from uniform behaviour, often do develop in soils (that is, apart from highly plastic soils which exist on the “wet” side of critical state) when subject to loading, and may be expected to influence their stress-strain behaviour significantly. The fracture of brittle hard soils-soft rocks is an important case in point. Accordingly, a model has been proposed to deal with such materials which is based on LEFM, where the fracture toughness is related to the matric suction of the air-water-solid medium. As such, there is a departure from the generally-accepted material behaviour adopted in fracture mechanics, in that the fracture toughness is state-, and hence, load-dependent. The model has been applied to a laboratory test specimen which was subjected to biaxial compression with reasonably good agreement with observed behaviour.

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