Numerical Analysis of Water Inflows to Underground Excavations - Current Status and Future Trends.

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ABSTRACT

For the rational analysis of groundwater flow problems associated with rock engineering projects, it is essential to understand the hydro-mechanical behaviour of jointed rock masses. It is also essential to develop efficient numerical methods capable of simulating initial hydraulic regime and the excitations introduced by the proposed construction or excavation. Difficulties arise in inflow modelling, due to (a) poor understanding of the physical processes, (b) limitations and uncertainties inherent to characterisation of the rock mass and the acquisition of input data, and (c) mathematical problems associated with the simulation of complex phenomena including the initial and boundary conditions. Although there has been a considerable progress in this field during past 2-3 decades, further technical developments towards the understanding of hydro-mechanical behaviour of rock masses are still required. In this paper, after an overview of fundamental aspects related to fluid flow and deformation processes in jointed rock, current techniques in modelling hydro-mechanical behaviour of rock masses and inflow prediction are reviewed. Problems and difficulties are highlighted, requirements for further research are identified and some new ideas are presented. Numerical models more versatile than the currently available models are considered necessary for realistic simulation of complex situations such as longwall mining where large deformations, mechanical and hydraulic anisotropy, transient and/or non-linear flow, periodic roof collapse, post failure behaviour of the goaf and three-dimensional geometries are involved.

INTRODUCTION

The groundwater condition existing within a given rock mass is an important factor that requires due consideration in rock engineering projects. The occurrence of unexpected groundwater flows and/or excessive water pressures can create a number of safety, stability and operational problems. This has more relevance to underground projects since the restricted work space, limited access and dewatering/drainage difficulties inherent to them usually makes the situation worse. These adverse effects can only be mitigated by proper evaluation of the potential groundwater problems in the planing and design stages of the project, and thereby providing adequate measures during design and construction and operational stages. For rational analysis and design, it is essential to understand the hydraulic and mechanical behaviour of the rock mass, the natural mechanisms that govern groundwater flows, and how the hydro-geological system would respond to the changes induced by the proposed construction or excavation. The complex and random nature of geological formations makes it difficult to characterise the mechanical and hydraulic behaviour of the rock mass and formulate reliable prediction procedures adaptable to most practical situations. Although there has been a considerable progress in this field during past 2-3 decades, further technical developments towards the understanding hydro-mechanical behaviour of rock masses and more reliable prediction methods are still imperative.

Analysis of water inflows to underground excavations requires:

a) Identification of potential water sources and sinks, and defining their geometry and storage-conductivity characteristics,

b) Identification of structures including any permeable strata, open or filled joints and other geological features such as faults, dykes, etc. and establishment of their hydro-mechanical characteristics,

c) Evaluation of initial stress fields, and

d) Understanding and simulation of changes in stress, displacements and flow fields induced by excavation processes.

The limited data acquired during investigations are usually available in the form of piezometric levels, bore hole pumping data and laboratory test results. They are interpreted to establish various geometries, material properties and other parameters required for the analysis. Subsequently, conceptual and mathematical models, simulating the real system are developed and solved to establish the potential and flux fields of the initial system. Once the initial system is satisfactorily modelled, the model is then solved under a range of initial and boundary conditions to simulate the changes introduced by excavation, and thereby predict the stress, displacement, hydraulic potential and flow fields. Figure 1 illustrates a flow chart depicting the factors contributing towards water inflows into excavations and the associated risk of failure.

HYDRO-MECHANICAL BEHAVIOUR OF JOINTED ROCK

The success of any inflow prediction method largely depends on our ability to describe accurately the fluid flow process in a given rock mass under different stress and strain fields. Rock masses essentially consist of intact rock (rock matrix) and fractures/joints. Major defects such as faults and dykes may also be present and act as super-conductors. Any significant flow component through each of these features and rock matrix must be taken into account in the prediction model. The main methods adopted to solve flow problems in rock masses are (a) equivalent porous media approach (b) discrete fracture network approach, and (c) the dual porosity approach. The most appropriate method for a particular flow problem is determined by the nature and extent of jointing in the rock mass, relative significance of flow component in the rock matrix and in the joints, and the nature of detailed information required for the design. The availability of computer resources and time restrictions may also influence the selection of the method, hence, a compromise often becomes necessary.

The equivalent continuum approach requires the existence of a Representative Elementary Volume (REV) and a symmetric equivalent permeability tensor [1]. When a REV cannot be found or it is too large making the measurement of hydraulic conductivity in the REV becomes impractical. Therefore, an alternative stochastic continuum representation [2] may be adopted. In this method, hydraulic conductivity tests carried out at sub-REV scale are analysed statistically to obtain representative values. The discrete fracture network method requires detailed information about the distribution of fractures and constitutive relationships describing the flow in individual fractures under various stress-strain conditions and hydraulic potential gradients. The permeability of rock matrix is normally ignored. If the permeability of rock matrix is also significant, then the dual porosity approach is more realistic. For any given site, the detailed information on individual fractures necessary to define them explicitly in the discrete fracture network is often difficult to obtain with currently available field methods. In order to overcome this problem, stochastic discrete models have been developed. However, the potential for implementation of stochastic models in coupled hydromechanical analysis is limited due to extensive computer time and storage requirements [3]. The developments in the discrete fracture network method have been concentrated mainly on two important aspects, namely, (a) understanding and mathematical representation of the deformation and fluid flow processes in a single fracture and, (b) development of fracture network models for simulation of the actual joint systems.

Fluid flow in a single fracture

The "cubic law" based on the assumption of steady laminar (incompressible) flow between smooth parallel plates is normally used to describe the flow through fractures in rock. Many researchers have investigated the validity of the cubic law for flow through fractures in rock using artificial fractures as well as natural fractures. The results of these investigations indicate that the

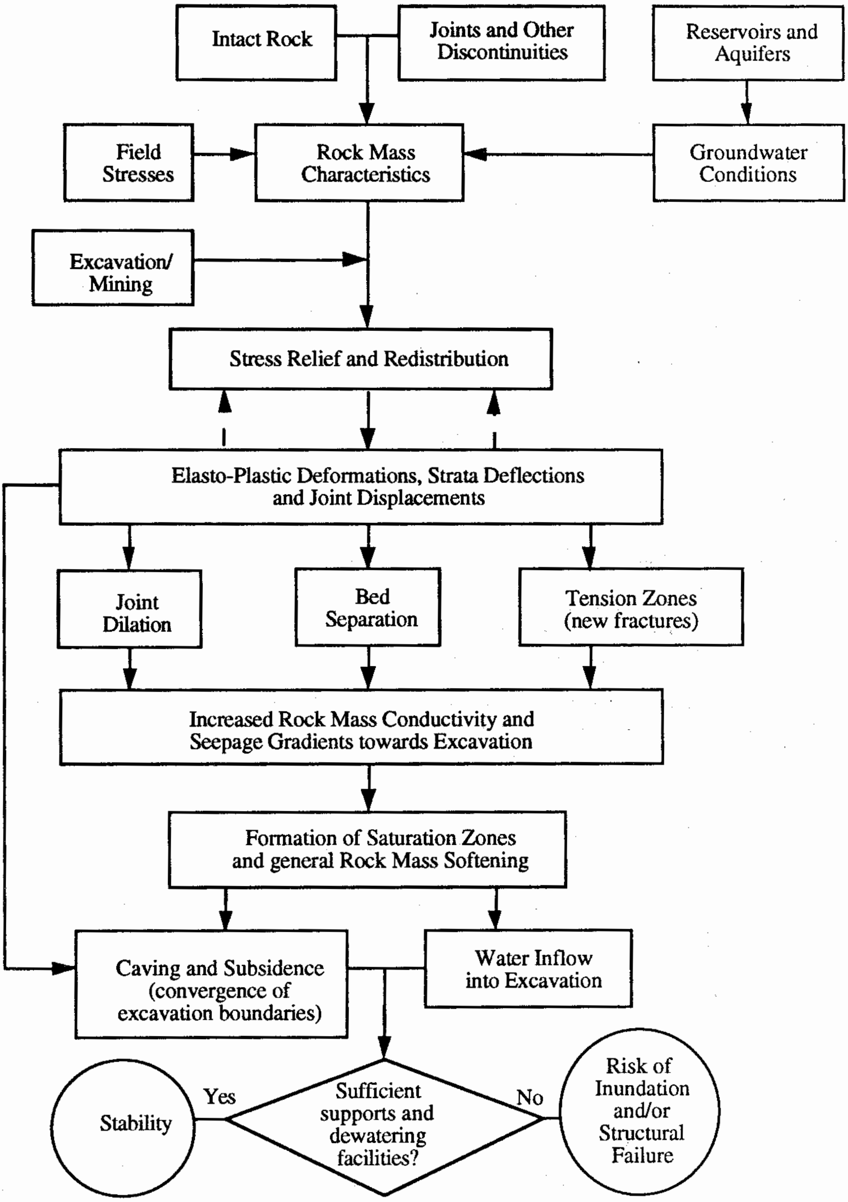

Fig. 1. Factors contributing towards water inflow to excavations and risk of failure.

cubic law is acceptable for smooth open fractures under certain conditions [4]. For rough open fractures, cubic law can be used with an appropriate correction factor to allow for the effect of roughness. For tight and rough discontinuities, the cubic law deviates from accuracy. The flow through tight discontinuities is strongly influenced by the stress changes and at low stress levels, the aperture and flow change rapidly with the stress. At high stress levels, there is no significant changes in aperture, tortuosity and flow with the stress changes [5]. A complete closure of fluid flow is difficult to achieve even at high stress levels. This phenomenon is best explained by the changes in void geometry and contact area between the joint faces under applied stress and resulting reduction in flow area and increase in tortuosity of flow paths. The geometry and the distribution of void volume within the joint interface, together with the geometry and the mechanical properties of the asperities of joint surfaces, control the fluid flow behaviour under different stress conditions. The flow condition is considered non-linear when the flow rate is not directly proportional to the hydraulic gradient. Non-linear flow conditions occur when the inertial effects (due to acceleration, divergence or convergence) and/or the kinematic effects (due to head losses at high velocity or turbulence) become significant under high hydraulic gradients. Non-linear flow conditions can occur in high hydraulic gradient situations such as flow under dam foundations, close to excavations, and in karstic formations. The above situation can be modelled by the following analyses:

(i) Forchheimer Equation: $\nabla\phi = av + bv^2$

(ii) Missbach Equation: $\nabla\phi = cv^m$ or $v = -K\nabla\phi^\alpha$

where, $\nabla\phi$ = Hydraulic gradient,
v = Flow velocity,
K = Hydraulic conductivity,
a, b, c and m are constants, and
α = 1 for laminar flow and 0.5 for turbulent conditions.

Figure 2 shows the typical relation between the flow rate and the excess pressure in a rock fissure. Among much technical literature, the references [4, 5, 7] provide valuable information regarding fluid flow behaviour in rock joints.

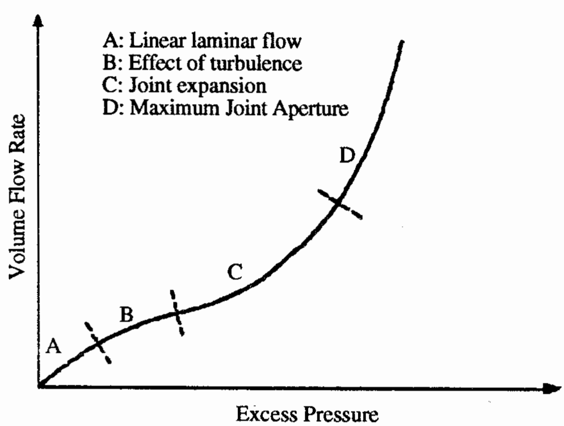

Fig. 2. Typical relationship between flow rate in a fissure and pressure.

Normal stress-closure behaviour of rock joints

The joint deformation behaviour under normal and shear loading conditions has been investigated by many researchers. Two main approaches to quantitatively describe the deformation process are (a) analytical method based on the theory of elastic contacts and (b) the empirical approach based on experimental evidence [4, 9]. Figure 3 shows an example of normal stress-deformation behaviour for an intact rock, and for a rock joint under interlocked and mismatched conditions. Figure 4 shows the normal stress-deformation behaviour under cyclic loading. Some of the important aspects related to normal deformation behaviour highlighted below.

1) Predominantly non-linear nature of normal stress-closure relationship: The normal stiffness is not a single value, but a function of the normal stress, as shown in Figure 3.

2) Hysteretic and inelastic behaviour during unloading: Permanent set reduces upon repeated loading. Maximum closure depends on the loading history, as shown in Figure 4.

3) The lower normal stiffness of the mismatched joints in comparison with the normal stiffness of mated joints: Bandis et al [6] have attributed the lower stiffness of mismatched joints to the stress concentrations over lower contact areas and the lack of asperity confinement. Mismatched joints exhibit larger permanent set during cyclic loading compared to the mated joints.

4) The lack of significant influence of scale effects on normal deformation behaviour [14]: Wei and Hudson [13] have discussed this matter and have pointed out that in the case of mismatched joints the scale effects can influence the normal deformation behaviour. Figure 5 shows the effect of specimen size on the relation between hydraulic conductivity and normal stress, based on some published experimental data.

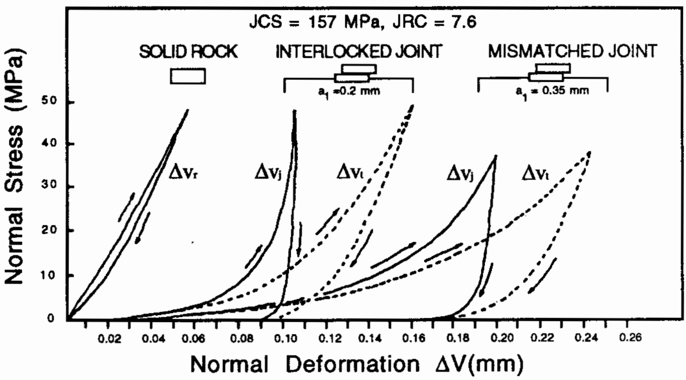

Fig. 3. Examples of normal stress-deformation behaviour for a solid rock and for interlocked and mismatched joint (Bandis et al., 1983).

Goodman [10] and Bandis et al [6] have proposed hyperbolic functions to model normal stress-closure behaviour of interlocked joints. The function propose by Bandis et al. take the form

$$\sigma_n = \frac{\Delta V_j}{b - c\Delta V_j} \quad ;$$

where, c and b are constants defined by initial stiffness (k_{ni}) and the maximum closure (V_m).

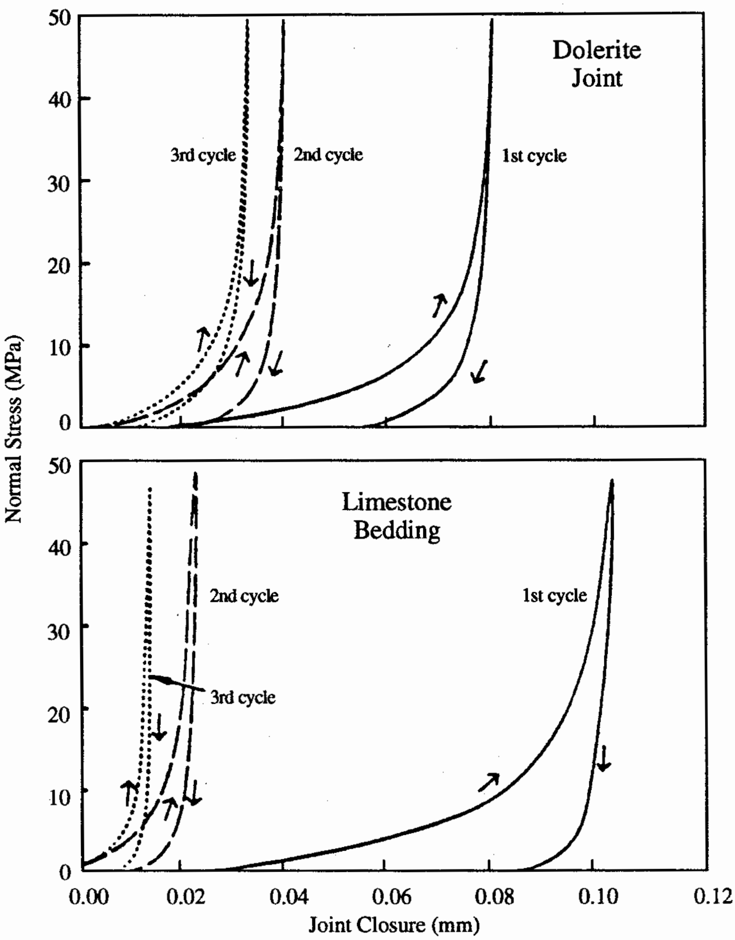

Fig. 4. Normal stress-deformation behaviour under cyclic loading (Bandis et al., 1983).

For the mismatched joints, Bandis et al. found that the semi-logarithmic function gives the best fit with their experimental observations, and proposed the following function:

$\text{Log } \sigma_n = p + q\Delta V_j$; where, p and q are constants. The constant p represents the initial normal stress.

The constant q describes the relationship between the normal stress and the incremental normal stiffness (K_n) in the form $K_n = \dfrac{q\sigma}{0.4343}$.

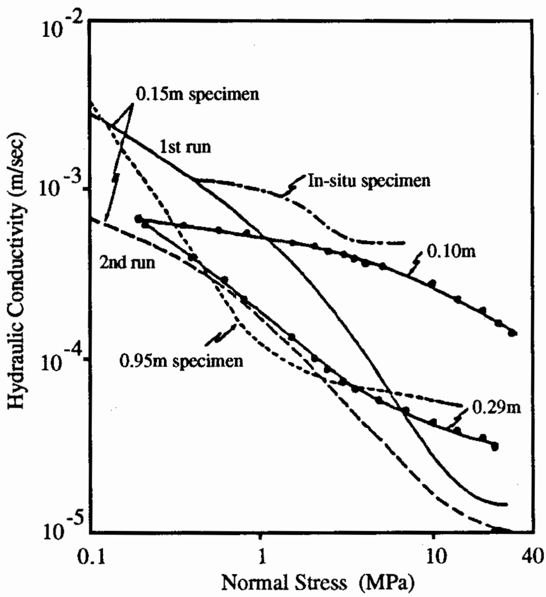

Fig. 5. Effect of specimen size on the relation between hydraulic conductivity and normal stress (Gale and Raven, 1980).

Shear stress-shear deformation behaviour of rock joints

The deformation behaviour under shear loading is more complicated than the normal deformation behaviour. The following aspects are important to note.

1) The dependence of shear behaviour on the normal stress level and the surface characteristics of the joint surfaces. The shear stiffness depends on both the normal stress and the shear stress.

2) The phenomenon of dilation associated with shear deformation has a significant impact on the hydraulic conductivity. The dilation effect also contributes to the frictional resistance of the joint. Dilation begins after a finite displacement of the joint has taken place and continues at an increasing rate as the peak shear strength is approached.

3) The shear behaviour is strongly influenced by the scale effects. With increased joint size, the peak shear strength decreases while the peak shear displacement increases. Consequently, the shear stiffness is seriously affected. The delayed start of dilation also results in an increased joint size due to the enhanced peak-shear displacement.

Kulhawy [11], and Hungr and Coates [12] have proposed hyperbolic functions to describe the shear stress-deformation behaviour of rock joints within the pre-peak shear stress range. Based on Barton's parameters, Sharp [27] proposed a dimensionless model (Figure 6) to simulate the shear stress-displacement process. This model is based on the dimensionless ratios

$$\frac{JRC(mob)}{JRC(peak)} \text{ and } \frac{\delta(mob)}{\delta(peak)}$$

where, JRC is the Joint Roughness Coefficient and δ is the shear deformation. The subscripts (mob) and (peak) indicate mobilised and peak values, respectively.
The model describes the behaviour within the pre-peak shear stress region as well as the post-peak shear stress region. They proposed empirical relationships to correlate the laboratory values with the parameters of natural joints and thereby incorporated scale effects. The model parameters are determined by JCS, JRC, residual friction angle (Φ_r) values and the level of effective normal stress(σ_n).

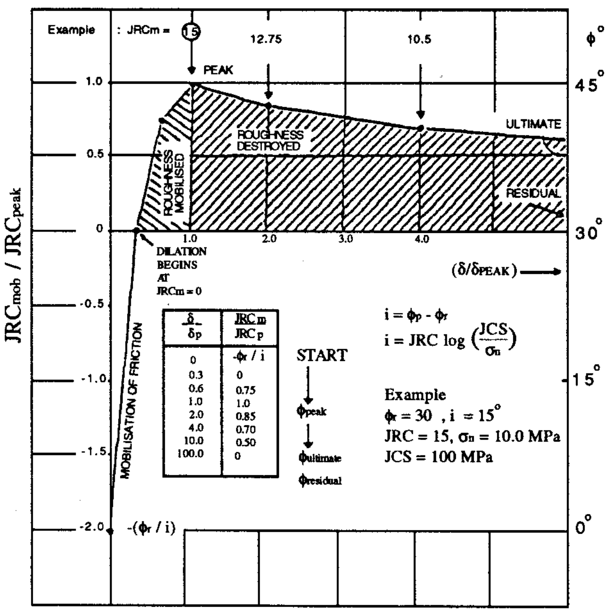

Fig. 6. Dimensionless model for shear stress-deformation modelling (Sharp 1970).

Wei and Hudson [13] proposed the following two functions to describe the post-peak portion of the curves with unequal peak and residual shear strengths.

$$\tau = A + \frac{B}{(D+\delta)} + \frac{C}{(D+\delta)^2}$$

$$\text{where } A = \tau_r \qquad B = \frac{2\tau_r\delta_p(\tau_p - \tau_r)}{\tau_p - [\tau_p(\tau_p - \tau_r)]^{0.5}} \qquad C = \frac{B^2}{4(\tau_r - \tau_p)} \quad D = -\frac{2C}{B} - \delta_p$$

$$\tau = H - \frac{\delta}{C + D\delta}$$

$$\text{where } H = \frac{\tau_p - R_{f2}\tau_r}{1 - R_{f2}} \qquad R_{f2} = 1 - R_f \qquad C = \left[\frac{1}{H - \tau_p} - D\right]\delta_p \quad D = \frac{1}{H - \tau_r}$$

τ = Shear stress	τ_p = Peak shear strength	R_f = Failure ratio
δ = Shear displacement	δp = Peak shear displacement	τ_r = Residual shear strength

In order to overcome the difficulties associated with implementing piecewise linear relationships in numerical models, Cundall and Lemos [25] proposed a continuously-yielding joint model. This model takes into account non-linear compression, non-linearity and dilation caused by shear and the non-linear limiting shear strength criterion [26]. In this model, the progressive reduction in the mobilised friction angle of the rock joint with plastic displacements has been incorporated in the formulations.

Conductivity coupling

To model the fluid flow behaviour in a joint it is necessary to couple the above stress-deformation models with associated conductivity changes. The cubic law or any other relationship describing the fluid flow in a joint is essentially a function of the equivalent hydraulic aperture (e) of the joint. In the models describing the stress-deformation behaviour, the mechanical aperture of the joint (E) is considered. Therefore, it is necessary to find the relationship between E and e, under the complete range of normal and shear stress conditions. The difference between E and e is due to the flow losses caused by tortuosity and roughness effects. In the conductivity coupling proposed by Barton et al.[14], the following empirical relationship has been adopted as illustrated in Figure 7.

$e = \dfrac{JRC^{2.5}}{(E/e)^2}$. Once e (in μm) is determined, the conductivity k can be obtained from the following relationship bases on parallel plate model:

$e = (12k)^{0.5}$.

The mechanical aperture (E) can be estimated from the surface topography. The common approach is to estimate the initial aperture (E_0) and use the relationship $E = E_0 - \Delta E$ to obtain E under different stress conditions. The change in mechanical aperture (ΔE) is either measured or estimated from the corresponding stress-displacement relationship. In the Barton et al.[14] model, the dilation-conductivity coupling is achieved by incorporating the change in mechanical aperture due to dilation, ΔE, given by the relationship $\Delta E = \Delta\delta \tan d_n(\text{mob})$, where $d_n(\text{mob})$ is the mobilised dilation angle and $\Delta\delta$ is the increment of shear displacement. However, the shear-conductivity coupling has not been tested adequately due to experimental difficulties inherent to the measurement of fluid flow under shear deformation conditions.

Wei and Hudson [13] proposed the following linear relationship as illustrated in Figure 8.

$$e = \frac{e_0}{E_0}(E - E_{min})$$

where, e_0 = Maximum hydraulic aperture,
E_0 = Closure of the joint, when the hydraulic aperture is zero.
E_{min} = Residual mechanical aperture, when the hydraulic aperture is zero.

In order to incorporate the hydraulic aperture changes caused by shear dilation, they have proposed the following relationship:

$e = e_i + k E_d$ where e_i is the initial hydraulic aperture (at zero normal stress), k is a constant and E_d is the shear dilation.

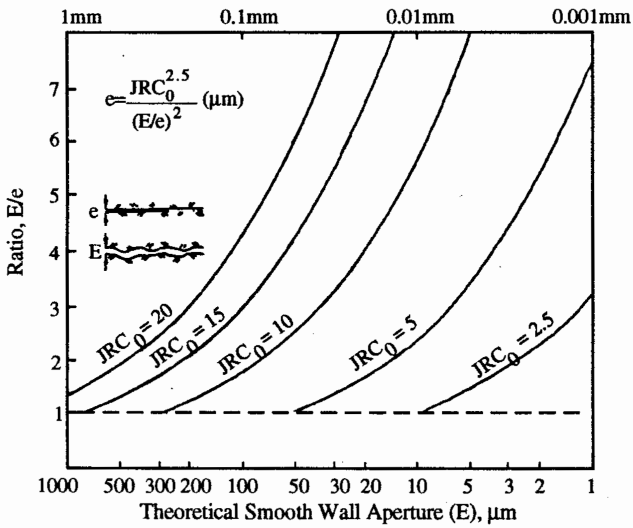

Fig. 7. Empirical relationship incorporating joint roughness and aperture (Barton, 1982).

Olsson et al.[15] measured the normal compliance and fluid flow rates through a natural fracture in Austin Chalk as a function of the shear offset (tangential displacement with joint surfaces separated, and at zero normal stress) and slip (tangential displacement with joint surfaces in contact, and at non-zero normal stress). They measured shear stress, slip, dilation and flow rate under constant normal stress in an initially mated joint. The peak shear strength was achieved after 0.2 mm slip and the residual strength was achieved after about 2.0 mm. The flow rate did not change significantly until the peak shear stress but increased steadily in the post-peak shear regime. They observed that the tangential movement of the fracture walls (mismatch) cause order-ofmagnitude changes in the conductivity. Within the stress range used, the change in conductivity due to normal stress was only by a factor ranging between 2 and 3.

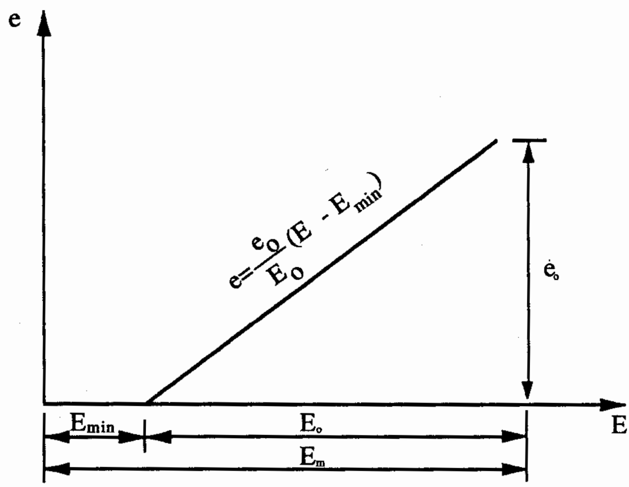

Fig. 8. Relationship between hydraulic and mechanical apertures of a joint (Wei and Hudson, 1988).

JOINT SYSTEM MODELS

In the analysis of fluid flow in the rock mass, flow laws developed for a single joint and the mass balance equation are applied to the system of joints. The resulting sets of equations are solved to obtain the potential and flow fields. The joint system model selected for the analysis should resemble the actual jointing of the rock mass in order to obtain any meaningful results. At present, there are a number of joint system models available for this purpose. The Orthogonal Model, Beacher Disk Model, Veneziano Model, Dershowitz Model and Mosaic Block Tessellation Models are some of these options. In the development of the above mentioned models, consideration has been given to the joint characteristics such as joint shape, joint size, orientation of joint sets and the termination and intersection aspects of joints. These models differ from each other with respect to the assumptions made about the joint characteristics. The joint shape is assumed to be either a rectangle, a circle, an ellipse or a polygon. Joints may either be bounded or unbounded. Joint generation is achieved either deterministically or stochastically. These models are intended to represent most of the common joint geometries. Depending upon the characteristics of the actual joint system in the field, the most appropriate joint model has to be adopted [16,17,18].

FLOW MODELS

Conceptual models

Conceptual models for the fluid flow analysis in rock masses can be divided into two main categories, namely, the equivalent continuum models and the discrete flow models. The Equivalent continuum representation is possible when the rock mass behaves like an equivalent porous medium. In these models, all the variables are defined in an average sense, and their distribution within the fractures cannot be obtained. In the discrete models, the fluid flow in the joints of the rock mass is modelled explicitly. The governing equations are derived by applying the flow laws to individual joints, and by applying the mass balance equations for the complete network of connected fractures. In double porosity models, the rock matrix and the joint network are considered as two distinct but overlapping continua.

Numerical models

Once a continuum or a discrete fracture flow modelling approach is adopted, the numerical models employing the finite different, finite element, distinct element or the boundary element formulations can be used to model the flow under a given set of initial and boundary conditions. For accurate predictions, a realistic set of constitutive relationships must be incorporated in the numerical analysis. Numerical models provide a useful tool for the quantitative analysis of fluid flow, and make the comparative assessment of the alternative engineering solutions somewhat easier.

The formulations in numerical models can be divided in to two main groups, namely, the domain-type formulations and the boundary formulations. The Finite Element, Finite Difference and the Distinct Element methods are in the first category while the Boundary Element method is in the latter category. In domain type methods, the problem domain is subdivided in to elements. In boundary methods, the boundary of the problem domain is subdivided into elements. Hybrid models using a suitable combination of the above methods can be more appropriate under certain situations.

The Finite Element method and the Finite Difference method offer two powerful alternatives for the coupled flow-deformation analysis of unfractured or fractured porous media. The Extensive meshing requirements, and the large number of equations and data input requirements are the disadvantages of these methods. With the Finite Element Method, it is easy to handle irregular shapes and non-linear phenomena involving strong coupling processes. The main drawback of the early Finite Difference models was the restrictions imposed by the rectangular mesh requirements. However, the later models are not restricted to rectangular grids, and capable of handling complex boundary shapes and elements with different properties. When dealing with volumetrically extensive far field conditions, difficulties arise with both the above methods.

The Boundary Element Method allows the efficient representation of infinite and semi-infinite domains and therefore, it is best suited for linear, volumetrically extensive far-field of infinite body problems [19]. The method is not suitable for non-linear problems. The BEM models are relatively easy to modify, compared with other methods.

Although some continuum models have the facility of incorporating discontinuities by interface or "slide lines", difficulties arise when many intersecting interfaces are used. Furthermore, these models are limited to small displacements and/or rotations and they do not have any automatic scheme to recognise new contacts. In contrast, distinct element methods allow finite displacement and rotations of discrete bodies, including complete detachment, and recognise new contacts automatically as the calculation progress [20]. In the distinct element method, a rock mass is represented as an assembly of discrete blocks while the joints are viewed as interfaces between distinct bodies. Deformable blocks are subdivided into triangular, finite-strain elements. In the deformable blocks, motion is calculated at the grid points of the triangular finite-strain elements within the blocks. Then, the application of the block material constitutive laws give the new stresses within the element.

In hybrid models, coupled formulations involving both the domain type methods and the boundary methods are employed in order to ensure more realistic representation of the near-field and far-field environments. When a Representative Elementary Volume exists for the rock mass, but it is of the same size or larger than the size of the structure built on the rock mass, the continuum approach cannot be used within a certain distance from the structure. Wei [3] developed a 3-D Discrete-Continuum model for flow analysis in such a medium. In his model, a discrete element algorithm was employed based on a static relaxation scheme to analyse the deformation and fluid flow in the near-field. The far-field was analysed using the Boundary Integral Equation Method. Pan and Reed [21] developed a coupled Distinct Element-Finite Element model for large deformation analysis of rock masses. Their model was intended for situations where a part of the problem domain can be best represented by an elastic or visco-plastic continuum (eg. weak and soft rock), and the remaining part can be best represented by a discontinuum(eg. jointed hard rock). Lorig and Brady [22] developed a Distinct Element-Boundary Element computational scheme for excavation and support design in jointed rock media. The far-field was modelled as a transversely isotropic continuum by a boundary element scheme and the near-field was modelled as a set of discrete elements. Elsworth [23] proposed a Boundary Element-Finite Element procedure for linear and non-linear fluid flow simulation in porous and fractured aquifers. In his formulation, non-linearity is restricted to turbulent flows at high Reynolds numbers.

Hydro-mechanical coupling

In many practical situations, the strong interdependence between the mechanical behaviour and the hydraulic behaviour of the rock mass makes it necessary to formulate coupled solutions.

In explicit coupling, two separate models are developed and information is transferred between the two models, and iterations performed until convergence. This is more suited for steady state problems, but less successful in transient problems where the solution procedure can rapidly become numerically unstable. In implicit coupling, the formulation is such that the solution for the displacements and pressures are obtained simultaneously during each iteration. With new stress-strain values, the stiffness and permeability tensors can be updated for the next iteration and the procedure is continued until convergence is reached.

PROBLEMS ASSOCIATED WITH INFLOW MODELLING

The problems associated with fluid flow modelling in jointed rock masses can be broadly divided into three main groups, namely,

a) Poor understanding of the various processes involved,
b) Data limitation and uncertainty of input data, and
c) Mathematical problems .

Problems due to poor understanding of physical processes

One of the most important area where the understanding is still lacking is the flow behaviour under shear displacement conditions. This is mainly due to the experimental difficulties associated with flow measurements under shearing conditions. The complex nature of shearing process, during which wearing of asperities, gouge formation and dilation take place, creates additional difficulties in the interpretation of experimental results. Wei [3] has highlighted the importance of studying the conductivity changes in the direction normal to shearing, as well as in the direction of shearing.

Limitation and uncertainty of input data

The selection of appropriate model parameters is extremely important in order to obtain realistic results in any modelling exercise. Difficulties arise when extrapolating small scale test results to represent generally larger problem domains. For example, defining the initial anisotropic permeability tensor for a continuum model, or defining the initial apertures for the distinct element method carries a significant degree of uncertainty due to the difficulty in obtaining precise data. One of the most important, but less investigated aspects is the connectivity of conducting joints.

Mathematical problems

Formation of saturation zones and bed separation zones in the rock mass has a significant influence on the groundwater behaviour of the rock mass. Although the flow regime in jointed rock is generally considered linear, non-linear conditions can prevail near the excavation boundaries. Transient flow can also prevail for a considerably longer period, particularly during construction, due to the progressive nature of excavation process. The conductivity changes in the post-failure region also can be of interest under certain situations. It is also important to be able to calculate directional permeabilities, and relate them with the stain components. Mathematical difficulties arise in the simulation of the above mentioned aspects in numerical models. The identification of failure and the formation of new cracks on one hand, and quantitative representation of associated conductivity changes on the other hand are the major considerations. Defining the effect of cyclic loading, especially under shear also create difficulties. The singularities associated with piecewise linear constitutive relationships which describe failure and deformation processes must also be considered. Numerical instabilities associated with time-dependant phenomena is another class of problems.

CONCLUSIONS AND NEW RESEARCH DIRECTIONS

Due to the historical development of the parallel plate model representation of flow in a single joint, most published work is still based on the equivalent aperture in defining fluid flow in joints. However, the equivalent aperture alone cannot represent fully the effect of joint geometry on the flow behaviour of tight joints. The effects of tortuosity and the reduction of flow area due to asperity contacts can be best represented by the rate of change in void volume with the applied stress. With the developments in laser profilometer techniques [15, 24], it is possible to characterise the surface topography of joint faces of laboratory specimens more accurately, and subsequently compare with the earlier methods. Supplemented with digitising and computer simulation/matching procedures, quantitative analysis of the void volume and contact area changes of the joint interface under different relative displacements of joint faces should become feasible. Taking these developments into consideration, the authors believe that a flow law containing suitable variables to represent the void volume/flow area, in addition to the equivalent aperture is more realistic for describing hydraulic conductivity changes under stress.

There is a great need to investigate the flow behaviour under shear displacement conditions incorporating directional effects. In order to obtain more representative values for directional permeabilities from field tests, the current test methods should be improved. One possible direction for improvement is to incorporate more observation wells, and to make use of 3-D numerical models to simulate the test conditions. A suitable testing procedure incorporating interchanging the injection and withdrawal wells as well as observation wells could provide the data base necessary to establish the link between connectivity and the directional permeabilities.

The limitation due to small strain assumption render the quasi-continuum methods unsuitable for situations, where finite displacements and rotations can occur. Although the distinct element is suitable for such problems, the assumption of impermeable blocks limits its application when the permeability of rock matrix is significant. For example, flow prediction under the complex situations of longwall coal mining requires a versatile numerical model capable of accommodating large deformations, mechanical and hydraulic anisotropy, transient and non-linear flow, periodic roof collapse, post failure behaviour of the goaf and three-dimensional geometries. At the present, most numerical models need simplification of one or several of these conditions. Therefore, increased accuracy in predictive models requires further development of numerical models for coupled hydro-mechanical analysis. These should be further developed and subsequently marketed as user-friendly software for potential users.

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