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The effective stress concept in a jointed rock mass

A literature survey

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PREFACE

This interim report is part of a research project about the effect of hydraulic pressure on rock joint properties. The report is a literature study of the effective stress concept in a jointed rock mass.

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It has been presented at a geo-scientific seminar, “Joints Mechanics“, in March 1997 at the Swedish Nuclear Fuel and Waste Management Company (SKB) in Stockholm, Sweden.

The views and conclusions presented in the report are those of the author and do not necessarily coincide with those of SKB.

Göteborg, April 1997

Roger Olsson

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SUMMARY

The effective stress concept was defined by Terzaghi in 1923 and was introduced 1936 in a conference at Harvard University. The concept has under a long time been used in soil mechanics to analyse deformations and strength in soils. The effective stress σ' is equal to the total stress σ minus the pore pressure u ($\sigma' = \sigma - u$).

The concepts´s validity in a jointed rock mass has been investigated by few authors. A literature review of the area has examined many areas to create an overview of the use of the concept.

Many rock mechanics and rock engineering books recommend that the expression introduced by Terzaghi is suitable for practical purpose in rock. Nevertheless, it is not really clear if they mean rock or rock mass.

Within other areas such as porous rocks, mechanical compressive tests on rock joints and determination of the permeability, a slightly changed expression is used, which reduces the acting pore pressure ($\sigma' = \sigma - \alpha \cdot u$). The α factor can vary between 0 and 1 and is defined differently for different areas.

Under the assumption that the pore system of the rock mass is sufficiently interconnected, the most relevant expression for a jointed rock mass, that for low effective stresses should the Terzagi´s original expression with $\alpha = 1$ be used. But for high normal stresses should $\alpha = 0.9$ be used.

SUMMARY (SWEDISH)

Effektiv spännings begreppet definierades av Terzaghi 1923 och introducerades 1936 vid en konferens vid Harvard Universitetet. Detta har under en lång tid använts inom jordmekaniken för att bl. a. analysera deformationer och hållfasthet i jord. Effektiv spänningen σ' är lika med total spänningen σ minus portrycket u ($\sigma' = \sigma - u$).

Effektiv spännings begreppets giltighet i en uppsprucken bergmassa har undersökts av få. Vid en litteratur studie inom området har en genomgång av flera områden utförts för att skapa en överblick över begreppets användning.

I flera bergmekanik och bergbyggnads böcker rekommenderas att uttrycket är lämpligt för praktiskt ändamål i berg. Dock är det inte helt säkert att man åsyftar en bergmassa utan intakt berg.

Inom andra områden som porösa bergarter, mekaniska normal försök på bergsprickor samt vid bestämning av permeabiliteten används ett något föränrat uttryck där portryckets verkan reduceras ($\sigma' = \sigma - \alpha \cdot u$). Faktorn α kan variera mellan 0 och 1 och definieras olika för olika områden.

Det mest relevanta för en uppsprucken bergmassa, under förutsättning att bergmassans por system har kontakt, är att vid låga normal spänningar bör Terzaghi´s uttryck med $\alpha = 1$ användas. Medan vid höga normal spänningar bör $\alpha = 0.9$ användas.

1. INTRODUCTION

The effective stress concept ($\sigma' = \sigma - u$) in geomechanics advocated by Terzaghi (1936) is useful for understanding the behaviour of geotechnical materials saturated with water. It has been widely accepted for soil mechanics for a rather long time, but its validity for other porous media, such as concrete and rock, has been questioned by many.

Its validity in a jointed rock mass has been less discussed and almost never tested and verified. In rock engineering books, the original effective stress concept by Terzaghi is often recommended for rock. It is only within the Hot Dry Rock (HDR) area were some researchers (Lanyon et al, 1992 and Cornet & Jianmin, 1995) have tried to verify the effective stress concept in a jointed rock mass (as known by the author).

The first substantial contribution to the concept of effective stresses was given by Fillunger in the 1910's. Unfortunately, these findings were almost completely ignored. However, Terzaghi got the credit for the discovery of the effective stress in 1923 but, he did not formulate "the principle" until 1936 when he introduced the concept to the minds of the engineers at the First International Conference on Soil Mechanics at Harvard.

According to Skempton (1962) and others Terzaghi's equation is excellent in the case of saturated soils but it is incomplete for other fully saturated porous materials, e.g. concrete and rock where the equation should have the following appearance:

$$\sigma' = \sigma - \alpha u \qquad (1.1)$$

were α is a reduction factor ($0 < \alpha \leq 1$) of the pore pressure.

The aim of this literature study is firstly to state the validity of the effective stress concept in a jointed rock mass and secondly to state the parameter α.

During the last years Oka (1996), Bluhm & Boer (1996) and Lade & Boer (1997) have proposed corrected expressions for the effective stress principle for porous media. Further, in the begining of their articles they discuss Terzaghi's works in a historical perspective.

2. EFFECTIV STRESS CONCEPT IN SOIL MECHANICS

As mentioned above, the effective stress concept was introduced by Tezaghi (1923). He held his first lecture (Terzaghi, 1936), of the subject in 1936 when he clearly stated the "principle of effective stresses" as follows:

> "The stresses in any point of a section through a mass of earth can be computed from the *total principal stresses* n_I', n_{II}' and n_{III}'which act in the

point. If the voids of the earth are filled with water under a stress n_w, the total principal stresses consists of two parts. One part, n_w, acts in the water and in the solid in every direction with equal intensity. It is called the *natural stress*. The balance, $n_I = n_I' - n_w$, $n_{II} = n_{II}' - n_w$ and $n_{III} = n_{III}' - n_w$, represents an excess over the natural stress n_w and it has its seat exclusively in the solid phase of the earth.

This fraction of the total principal stresses will be called the *effective principal stresses*...

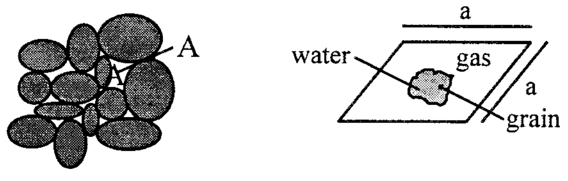
A change of the natural stress n_w produces practically no volume change and has practically no influence on the stress conditions for failure. Each of the porous materials mentioned (*sand, clay,* and *concrete, the authors*) was found to react on a change of n_w as if it were incompressible and as if its internal friction were equal to zero. All the measurable effects of a change of stress, such as compression, distortion and a change of the shearing resistance are exclusively due to changes in the effective stresses, n_I', n_{II}' and n_{III}'. Hence, every investigation of the stability of a saturated body of earth requires the knowledge of both the total and the neutral stresses“.

The expression is strictly valid for an incompressible porous solid filled with an incompressible liquid.

The exact value of the parameter α in equation (1.1) is controversial. Terzaghi suggested on theoretical grounds that α should equal the porosity η, but found experimentally that $\alpha \approx 1$.

Hubert and Rubey (1960) tried to prove theoretically that $\alpha = 1$ by compaering Archimedes buoyancy and Terzaghi buoyancy (an extension to include the case of the porous water-filled rocks), but their proof has been questioned. They also emphasised that they derived an effective stress law only for elastic strain and this law is not automatically applicable to inelastic processes such as those occurring in fractures and frictional sliding.

An other way to demonstrate the effective stress law in soil is done by Sällfors (1994) were he consider the soil as a two phase system were it is important to split up the stresses in different parts. If we have a small area (a x a) through a soil, cutting through a contact point between two grains, pore water and pore gas, as in Figure 1, it is possible to split up the stresses in one part, borne by grains and another part as transferred by water and gas (if gas is present).



a) *b)*

Figure 1. a) Soil grains b) Section A-A between to soil grains (after Sällfors, 1994)

In the section A-A therefore forces act which are taken up by the grains (F_s), pore water (F_w) and pore gas (F_g) and which belong to areas defined as A_s, A_w and A_g.

Following equilibrium equation can be established for the forces between the different phases:

$$\sigma A_{tot} = \sigma_s A_s + \sigma_w A_w + \sigma_g A_g \qquad (2.1)$$

If the soil is saturated, the area $A_g = 0$. Further, the contact area A_s between the grains is small (a few percent) compared to the total area witch means that $A_w/A_{tot} \rightarrow 1$. The equation (2.1) can then be written as:

$$\sigma = \sigma_s \cdot \frac{A_s}{A_{tot}} + \sigma_w \qquad (2.2)$$

If the pore pressure σ_w is called u and the stresses in the soil skeleton is related to the total area and named σ', the equation (2.2) can be written as:

$$\sigma = \sigma' + u \qquad (2.3) \qquad \text{or} \qquad \sigma' = \sigma - u \qquad (2.4)$$

were equation (2.4) is the same as Terzaghi´s.

3. THE EFFECTIVE PRESSURE LAW IN LOW POROSITY ROCKS

The permeability in low permeability rocks, such as crystalline, metamorphic and igneous rocks is according to Brace et al. (1968), Krantz et al. (1979), Brace (1980), Walsh (1981), Bernabe (1986, 1988) and others dependent on the effective pressure. The effective pressure law for permeability can be written as;

$$p_e = p_c - \alpha \cdot p_p \qquad (3.1)$$

where
p_e = effective pressure
p_c = confining pressure
p_p = pore pressure
α = reduction factor of pore pressure

Brace et al. (1968a) tested the strength of intact crystalline rocks and concluded that the law of effective stress (confining pressure minus pore pressure) holds for crystalline rocks of low porosity as long as the loading rate is kept below a certain critical value.

Brace et al. (1968b) measured the permeability of intact Westerly granite as a function of the effective pressure (p_c - p_f).

Krantz et al. (1979) tested the permeability of intact and jointed Barre granite. They pointed out that one should be cautions when applying the term effective stress to jointed media. At least for the hydraulic properties of jointed rock it is not simply the difference between external confining pressure p_c and internal fluid pressure p_f. Because, the permeability is also influenced by surface roughness and stress history and the external pressure produce greater changes on the permeability than the internal pressure.

Walsh (1981) discussed the effect of pore pressure and confining pressure on fracture permeability of Barre granite (Krantz, 1979) by using equation (3.1), with α derived by Robin (1973) where;

$$\alpha = 1 - v_p\beta_s/(\partial v_p/\partial p) \tag{3.2}$$

$v_p =$ pore volume

$\beta_s =$ compressibility of the rock surrounding the fracture

$(\partial v_p/\partial p) =$ rate of change of pore volume with applied hydrostatic pressure for a joint with no pore fluid

Walsh made the approximation that the term within brackets is nearly constant. However, Robin (1973) pointed out that the term is not necessarily constant for rocks. Using data from Krantz (1979) he got the following results;

$p_e = p_c - 0.56 \cdot p_p$ (tension fracture in Barre granite)

$p_e = p_c - 0.91 \cdot p_p$ (polished surfaces in Barre granite)

The confining pressure vs. pore pressure were plotted where the slope of the curves gave the α values.

Bernabe (1986) obtained that α ranged between 0.6 and 0.7 during permeability measurement in intact Chelmsford granite. The tests showed almost no directional effect on α.

Bernabe (1988) compared the effective pressure law for permeability and resistivity formation factor in intact Chelmsford granite. He assumed the effective pressure law as in equation (3.1) and discussed the α values, one for the permeability α_k, and one for the formation factor α_F. The formation factor α_F is obtained from electrical resistivity measurements. There was no significant difference between α_k and α_F .

4. EFFECTIVE STRESSES IN A POROUS ROCK.

In this chapter considers only porous material with isotropic response, and not anisotropic behaviour (Carroll, 1979). For most applications the effective stress tensor σ_{ij}' has the form;

$$\sigma_{ij}' = \sigma_{ij} - \alpha P_p \delta_{ij} \qquad (4.1)$$

where α = a constant which depends on the particular application
δij = Kronecker delta ($\delta = 1$ when $i = j$; $\delta = 0$ when $i \neq j$)

For linear elastic deformation of a porous medium α can be defined as;

$$\alpha = 1 - (K/K_s) \qquad (4.2)$$

where K = the bulk modulus of the dry porous material
K_s = the bulk modulus of the material without pores (the solid or grain bulk modules)

The equation (4.2) is called the effective pressure law (EPL) for bulk volumetric strain and was first suggested by Geertsma (1957) and by Skempton (1961) on empirical grounds.

Nur & Byerlee (1971) showed that the expression (4.2) is theoretically exact and when the effective compressibility of the dry aggregates is much greater than the intrinsic compressibility of the solid grains ($K << K_i$), which is often the case in natural aggregates, then $\alpha \approx 1$.

Robin (1973) were considered both volumetric strain and variation of pore volume on linear elastic, isotropic porous solids made up of an isotropic linear elastic material. He also assumed that all the pores of the solid was interconnected and that the fluid pressure in them was at equilibrium. During consideration of the pore volume variation the α could be defined as:

$$\alpha=[1-\rho \cdot K/(K_i-K)] \qquad (4.3)$$

where ρ = porosity

But he also says that equation (4.2) and (4.3) should not be accepted for rocks! The proof for this was made by calculating the α value with the same parameters from a sandstone (Nur & Byerlee, 1971) Equation (4.2) gave $\alpha = 0.64$ and (4.3) gave $\alpha = 0.97$.

Zoback & Byerlee (1975) measured permeability of Beaea Sandstone and their investigation showed that the permeability is not a simple function of the effective stress, i.e. α is not equal unity.

Boitnott & Scholz (1990) developed a technique for direct measurement of the effective pressure law and applied it to joint closure. It can not take into account hysteresis or other forms of nonlinearity in the relationship between pressure and joint closure. They found that α is less than 1.0, and its value is dependent on the ratio of the ambient pressure stiffness (K_a) and the stiffness to external pressure (K_x);

$$\alpha \equiv \frac{\Delta P_c}{\Delta P_p} = 1 - \frac{K_x}{K_a} \qquad (4.4)$$

and the only way for α to be 1 is when K_a is infinite. Their experiments on tension fractures in a diabase and a quartzite showed no decrease in α with increasing intercept pressure p_i (p_i is the value of p_c at $p_p = 0$) and α where just under 1. One very interesting point was that α is not dependent of the properties of the used fluid which perhaps means that the fluid pressure acts on the entire area, e.g. $\alpha \approx 1$.

5. NORMAL COMPRESSIVE TESTS ON ROCK JOINTS IN LABORATORY

Fluid flow through joints in low-permeability rocks depends on the state of stress in the rock mass. When a joint is stressed, the void space deforms and changes in contact areas occur, affecting the hydraulic and mechanical properties of the rock. Several researcher have investigated the normal displacement of the joints and the increase in contact area as function of applied normal stress. If we is calling the contact are A_0 and the area A_{tot} the α value defines as;

$$\alpha = 1 - A_0/A_{tot} \qquad (5.1)$$

Krantz (1979) reported results from Iwai's (1976) investigation about the effects of contact areas and asperity geometry on permeability. Iwai found that at low normal pressure (0.26 MPa) the real contact area of granite was less than 0.1% of the apparent total area and at high normal pressure (20 MPa) the contact area was around 10 - 20 %. This means that the value of α was less than 0.99 respectively 0.8 - 0.9.

Barton & Choubey (1977) observed that the contact area ratio $A_0/A_{tot} \approx \sigma'_n/JCS$, according to the damage visible at the end of around 1 mm shearing.

Barton et al (1985) reported that Bandis (1980) used a 12 µm polyester film inserted between the mating faces of joints to record the different distribution of contact points. Rough joints gave a non-uniform distribution of large individual contact areas, while planar joints gave a uniform distribution of numerous small contact areas. Performed normal closure tests conducted on joints representing five different rock types indicated ratios of A_0/A_{tot} between 0.4 - 0.7 and corresponding σ'_n/JCS between 0.3 - 0.7, i. e. the values of α were between 0.3 - 0.6.

Witherspoon et al (1981) used results from Iwai's (1976) laboratory investigations on mechanical and hydrological properties of tension joints in basalt and granite to test the validity of their developed "asperity model". By using Iwai's measurements of the normal displacement and estimating the contact area of the fracture, the flow was calculated for different normal stresses. The results were then compared with Iwai's experimental data. Witherspoon et al (1981) found that at a normal pressure of 20 MPa, the contact area should be around 15%, i. e. the value of α approximately 0.85.

Logan & Teufel (1986) investigated the true area of contact during friction sliding as a function of the normal stress. The experiments were performed in a triaxial equipment on specimens with saw-cut surfaces orientated 35° to the load axis. The contact area was determined by the use of thermodyes, applied as a lacquer on the sliding surface before testing. During sliding, heat is generated only at the asperity contacts and the dye gets different colours and textures which may be identified microscopically after sliding. The average, truel contact area was 16% at a normal stress of 374 MPa in sandstone and 18% at a normal stress of 25 MPa in limestone, i. e. the values of α were 0.84 and 0.82 respectively.

Pyrak-Nolte et al (1987) carried out a comprehensive laboratory study of the permeability, mechanical displacement and void geometry of single rock joints in a quartz monzonite (Stripa granite). During the experiments a metal injection technique was developed to study the fracture void and contact area as a function of stress. A molten metal was pumped into the void space until the desired pore fluid pressure was obtained. This was done during different fluid pressures and axial loads. The two halves of the sample were separated and the distribution of metal was examined by using both a scanning electronic microscope and photographic techniques. An image analysis were used to obtain the contact area at different stress levels. The results from theses are showed in table 5.1.

Table 5.1 Contact area (A_0) as a function of effective normal stress for sample E30 and E32.

σ_n' (MPa)	A_0 (E30)	α (E30)	A_0 (E32)	α (E32)
3	8%	0.92	15%	0.85
33	15%	0.85	42%	0.58
85	30%	0.70	42%	0.58

6. FORCE EQUILIBRIUM

Since rock materials can be considered as a multiple phase system there is reason to separate the stresses in de different parts (Pusch, 1974 and Nordlund & Rådberg, 1994). Figure 6.1 shows the different forces who acts in a partly saturated system.

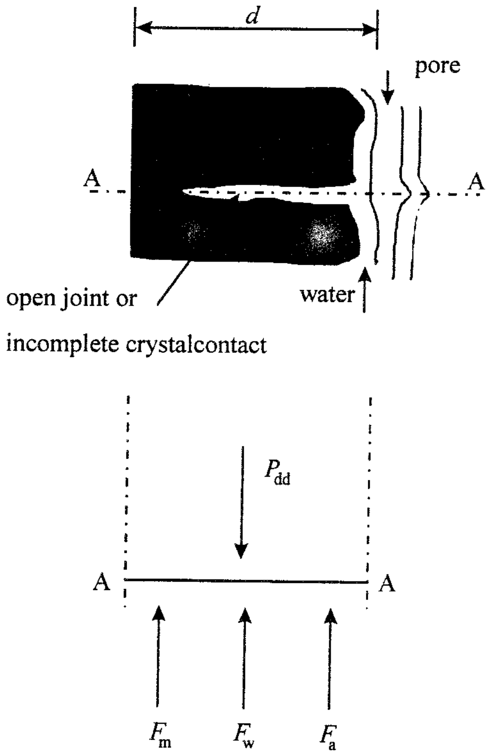


Figure 6.1. Forces in a rock element (Modified after Pusch, 1974)

Equilibrium prevails in section A-A according to the following:

$$P_{dd} = \sigma A = F_m + F_w + F_a \tag{6.1}$$

where σ is the total normal stress and A denotes the total cross section. The forces on the right side of equation (6.1) are taken up by the mineral contact (F_m), water (F_w) and gas (F_a). Equation (6.1) can also be written as:

$$\sigma A = p_m A_m + u_w A_w + u_a A_a \tag{6.2}$$

where A_m represents the area in part of the section where mineral contact prevails and A_w and A_a are corresponding areas through the water and the gas, respectively.

If the partial areas are divided with the total area A, one obtains:

$$\sigma = p_m a_m + u_w a_w + u_a a_a \tag{6.3}$$

or

$$\sigma = p_m a_m + u_w a_w + u_a(1 - a_m - a_w) \tag{6.4}$$

The magnitude of a_m, a_w and a_a depends on the porosity and the grade of discontinuity in the crystal system. In a saturated rock $u_a = a_a = 0 \Rightarrow a_m + a_w = 1$, and equation (6.4) can be written as:

$$\sigma = p_m a_m + u_w(1 - a_m) \tag{6.5}$$

In a very fine-grained rock without macroscopic joints, such as a basalt, a_m is close to 1. But in a macroscopic joint plane a_m has a very low value. Consequently, if the joint plane is saturated, $(1 - a_m) \rightarrow 1$. The pressure p_m in the small mineral contacts is very high and can not be neglected. Let $p_m \cdot a_m$ be represented by σ', use u for u_w, and equation 6.5 can be written as:

$$\sigma = \sigma' + u \tag{6.6}$$

7. JOINTED ROCKS - HOT DRY ROCK (HDR)

Hot Dry Rock (HDR) is a concept of extracting heat from low permeability resources, as e.g. crystalline rocks, with high temperature. The latest concept is based on the assumption that there exists a natural joint network in the rock which can be used for fluid circulation after stimulation.

Pine (1986) applied the effective stress concept for soil mechanics on rock joints, were the effective normal stress σ_n' is approximately given by:

$$\sigma_n' = \sigma_n - p \tag{7.1}$$

where σ_n = total normal stress across joint
p = fluid pressure in joint

He considered that this was a reasonable approach for low effective stresses, since natural joint contact points are quit few.

With higher contact pressure greater contact areas apply, and the following equation should therefore be more correct:

$$\sigma_n' = \sigma_n - \alpha \cdot p \tag{7.2}$$

After discussing results of different authors he concluded that in joints at depth in the Carnmenellis granite in Cornwall, where the effective normal stress is of the order of 10 MPa, the α value should be close to 0.9.

Evans et al (1992) used the following expression for the effective stress at fracture closure, proposed by Robin (1973):

$$\Delta\sigma_n^f = \Delta\sigma_n - \alpha^f \Delta P_f \tag{7.3}$$

where Δ = incremental
α^f = local gradient of constant aperture contours, plotted in σ_n versus P_f space and varies in general (when the range of variation of σ_n and P_f is not large as in a HDR system, α^f can be taken as a local constant)

According to Robin (1973) α^f can be written as :

$$\alpha^f = 1 - (K_{fv}/K_i) \tag{7.4}$$

where K_{vf} = the fracture void stiffness under drained conditions
K_i = the intrinsic modules of the solid component

Evans et al (1992) say that it is commonly assumed that the value of α^f for stress states of interest to HDR reservoirs is essentially unity. This is an exception supported by the contact theory, see chapter 5. Unfortunately is there no relevant experiment which verifies this. Nevertheless, even Evans et al (1992) take α^f as unity.

Lanyon et al (1992) applied a Discrete Fracture Network Models to a Hot Dry Rock geothermal system (Rosemanowes in Cornwall, UK). In the shear dilation model, Coulombs´s law with a constant friction angle φ was used. The stresses on each joint were calculated using an effective stress law with a contact area α and the used equation was the same as equation (1.1):

$$\sigma'_n = \sigma_n - \alpha P_0 \tag{7.5}$$

The model was calibrated to find the value for α and φ by using the pressure at which micro-seismic events were first observed. The calibration gave a $\alpha = 0.9$ and a $\varphi = 45°$.

In a deep well in France (Le Mayet de Montagne), both data from the HTPF (Hydraulic Tests on Pre-existing Fractures) method and focal plane solutions (focal mechanics of induced seismicity) were used for stress field determination and pore pressure mapping by Cornet & Jianmin (1995).

The HTPF method consists of conducting hydraulic tests on pre-existing fractures of known orientation for determining the normal stress σ_n supported by the fracture plane. From an inversion of the focal plane solutions of seismic events (during injection) they obtained two slip vectors from which one were chosen by help of the HTPF results.

By plotting the ratio dp/p_i on the ordinate vs. the value of the ratio τ/σ'_n on the abscissa, Cornet & Jianmine (1995) were able to verify Terzaghi's effective stress concept. Here,

dp=the pressure increments required to induce slip

p_i =the injection pressure at the well heaf

τ = the tangential stress supported by the slip planes

σ'_n = the effective normal stress

8. OTHER RELEVANT LITERATURE

In many geotechnical wellknown textbooks, the effective stress concept is mentioned assuming α equal to unity.

Jaeger & Cook (1969) discuss the results from Terzaghi and Hubert & Rubey (1960), addressing the question whether α should be equal to unity or not. Nevertheless, they chose α equal to unity.

Hoek & Brown (1980) takes the view that the original expression of effective stress by Terzaghi, is satisfactory for practical purpose, i. e. $\sigma' = \sigma - u$. They do this during the following circumstances; 1) the pore structure of the rock is sufficiently interconnected, 2) the loading rate is sufficiently slow to permite internal fluid to equalise during testing.

Hoek & Bray (1981) and Price & Cosgrove (1990) use Terzaghi's original effective stress concept without discussion.

Brady & Brown (1985) and Brown (1993) say that experimental evidence and theoretical argument suggest that, over a wide range of material properties and test conditions, the response of rock depends on the following equation, which is the same as equation (4.1):

$$\sigma_{ij}' = \sigma_{ij} - \alpha u \delta_{ij} \tag{8.1}$$

where $\alpha \leq 1$, and is a constant for given case. For highly porous rocks, soils and joints, the value of α approaches unity. For low porous rocks, the α value reduces considerable below unity.

9. DISCUSSION

The effective stress concept in a jointed rock mass has not been investigated by many. It is primarily in the HDR area, were the rock mass is exposed to a high fluid pressure, where it has been tested and verified. The value of the α-factor in equation (1.1) has been suggested to be between 0.9 to 1 dependent on the effective normal stresses acting on the joint plans.

Most rock engineering books assume Terzaghi's original concept ($\sigma' = \sigma - u$) to be valid but it is not really clear if the author(s) means rock or rock mass.

Obtained values of α from normal compressive tests on rock joints in the laboratory are often low. The mechanical contact points are of interest when fluid flow through the joint is studied. But, during pressurisation with a fluid, e.g. water, the fluid must intrude near the contact points and the "hydraulically dry" contact points must be very few. These means that the α value must be nearly unity.

The results and recommendations by Pine (1986) seem to be the most relevant for jointed rock masses during the assumption that the pore system of the rock mass is sufficiently interconnected. For low effective stresses Terzaghi's original expression where α is equal unity should be used. But, for high normal stresses a α value of around 0.9 should be used.

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