An investigation on the dilative behavior of crushed rock

Une recherché sur la loi de durcissement des roches concassées

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ABSTRACT

The dilative behaviour of a number of crushed rock samples tested in large triaxial apparatus under different confining pressures is reviewed and the dependency of dilatancy angle on material characteristics and test condition is studied.
Abrasion and point load tests were also carried out to evaluate hardness and compressive strength of the aggregate. An IS50–dependent correlation for dilatancy angle has been developed.
It was noted that under low confining pressures samples from various sources exhibited different extends of dilation. Crushed rock samples with higher grain hardness exhibited greater dilatancy under equal confining pressures.

RÉSUMÉ

Le comportement dilaté d'un certain nombre d'échantillons de roches concassées examinés dans un grand appareil triaxial aux différentes pressions de confinement est passé en revue et la dépendance de l'angle de dilatation sur des caractéristiques de matériaux et les conditions d'essai est étudiée.
Des essais de charge ponctuelle et d'abrasion ont également été effectuées pour évaluer la résistance à la compression et de la dureté de l'agrégat. Une IS50-dépendant corrélation pour l'angle de dilatation a été développée.
Il a été noté que sous des pressions de confinement plus faible des échantillons de diverses sources présentaient différents sortes de comportement dilaté. Échantillons de roche concassée avec une dureté supérieur ont montré une plus grande de dilatation sous les mêmes pressions de confinement.

Keywords : rockfill, crushed rock, dilation, triaxial test

1 INTRODUCTION

It is a well recognized fact that the so-called over consolidated granular samples show dilative behaviour during shear (Indraratna et el. 1998).). In this context, the term "over-consolidated" is meant to indicate that the density is high in relation to the surrounding pressure.

Samples with equal densities exhibit different dilative behavior under different confining pressures; dense samples dilate during shearing under low confining pressure, while the same sample could contract under high confining pressures.

However, the amount of volumetric strain is very much affected by a number of other factors such as particle shape, size and gradation (Wan & Guo 1999), as well as the type and strength of the parent rock constituting the grains (Barton & Kjærnsli 1981, Charles and Watts 1980). Hence simple pressure-density dependent dilatancy is not sufficient for development of an appropriate flow rule. The flow rule must take into account the influence of grain shape, size and strength as well.

It may intuitively be acknowledged that during shear, the asperities of less hard grains tend to break under high confining pressures and thus less dilation is observed than for the case of samples with harder grains under similar conditions.

Increased confining pressure causes an increase in loading energy absorption capacity and thus dilation (or work softening) must reduce with increase of confining pressure (Lade 1977).

Particle breakage is a form of energy dissipation and may therefore, be interpreted as prelude to reduction in dilative behaviour. Hence, the easier the particles can break, the less absorption capacity (i.e. confining pressure) is required to avert dilation.

In this article, the influence of the strength and hardness of the grains on the hardening/softening behaviour of crushed rocks is studied by examining the results of point load and abrasion tests, as well as the results of large triaxial tests on a number of different rockfill samples from various dam construction projects in Iran.

2 CONSTITUTIVE MODELS

Under drained conditions, the amount of dilation has a significant effect on the post-failure stress-strain behavior of granular materials. For this reason, many attempts have been made to evaluate and quantify the rate and amount of dilation through experimentation (Wan & Guo 1998).

There are a number of constitutive models available for rockfill materials (Varadarajan et al. 2003). In most of the available models, some reference to non-associative flow rule and stress dependent dilatancy is made.

There are a number of different ways to specify dilatancy; some have defined a dilation angle in terms of volumetric and shear strains while others have derived it from the gradient of plastic potential in terms of strain invariants. In this study, the definition presented by Wood (1991) is used (Equation 1):

$$\tan\beta = \left(\frac{-d\varepsilon_v^p}{d\varepsilon_d^p}\right) = \left(\frac{-d\varepsilon_v^p}{d\varepsilon_1^p - d\varepsilon_3^p}\right) = \left(\frac{-2d\varepsilon_v^p}{3d\varepsilon_a^p - d\varepsilon_v^p}\right) \quad (1)$$

where ε_v^p is the plastic volumteric strain,

ε_d^p is the plastic deviatoric strain,

and ε_v^p is the plastic axial strain.

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The negative sign arises from the convention that contractive volumetric strain and increasing deviatoric strain are taken as positive.

Separation of elastic and plastic components of strains from test results is not straightforward. However, the contribution of elastic strain to total strains may be assumed as negligible when yielding is occurring and the difference between a plastic strain increment ratio and a total strain increment ratio may be small (Wood 1991). Therefore, in the present study the measured strains (i.e. total strains) are assumed to be equal to plastic strains.

3 MATERIAL SPECIFICATIONS

Seven sets of experimental data on five different rockfill samples are used here for evaluation of the influence of grain strength on the dilative behavior.

The experimental data includes the results of consolidated drained triaxial tests on 30cm diameter samples, point load test on saturated aggregates and abrasion test results.

All samples were angular and within GW classification, with very similar gradation curves (i.e. $C_u>16.9$ and $1.2<C_c<2.9$). Parallel scaling was used to limit the maximum particle size to 30mm.

The summary of the rock type and the results of point load and abrasion tests are presented in Table 1.

Table 1: Material Specification

Rock Type	Point Load Test Saturated I_{S50}	Abrasion Test Weight loss %
Andesite	11.0	19
Diorite-Andesite	4.5	28
Diabase	4.9	30
Basalt	8.3	-
Dolomite	4.8	-

4 TRIAXIAL TEST RESULTS

The large-scale static CD triaxial tests were carried out under varying confining pressures ranging mostly from 50 kPa to 900 kPa, in the Building and Housing Research Centre (BHRC) of Iran. Axial loading was imposed with constant rates of prescribed displacement of 0.5 mm/min (Sadeghpour 1998).

The triaxial specimens were all compacted in a similar manner with the same compacting effort to densities ranging from 1.81 to 1.86 ton/m^3. The prepared samples were all subjected to partial vacuum to facilitate saturation and “B” values in excess of 0.95 were achieved.

Usually, in the triaxial tests axial loading is continued up to failure point which is marked by the maximum deviatoric stress and the test is stopped shortly afterwards. However, here most of the tests were continued up to around %15 axial strain. This has helped to develop a better understanding of the post failure behaviour of the rockfill samples.

In the interest of brevity, the results of only one set of the tests that were carried out on the andesite samples are presented here. The variation of deviatoric stress versus axial strain is presented in figure 1 and the variation of volumetric strain versus axial strain is presented in figure 2.

The calculated variation of dilatancy angle with axial strain is also presented in figure 3. The rest of the results may be found in the quoted references.

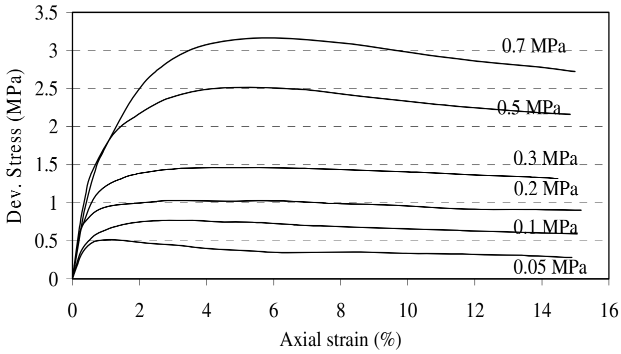

Figure 1. Variation of deviatoric stress versus axial strain for different confining pressures

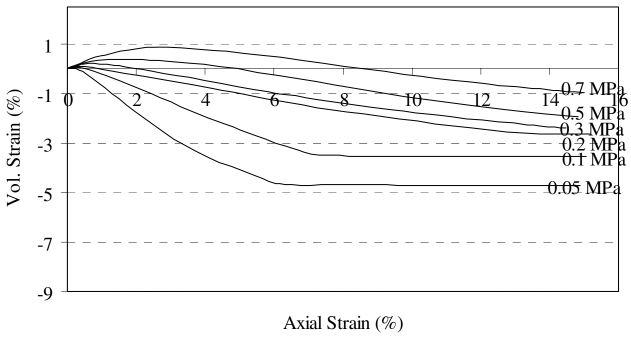

Figure 2. Variation of volumetric strain versus axial strain for different confining pressures

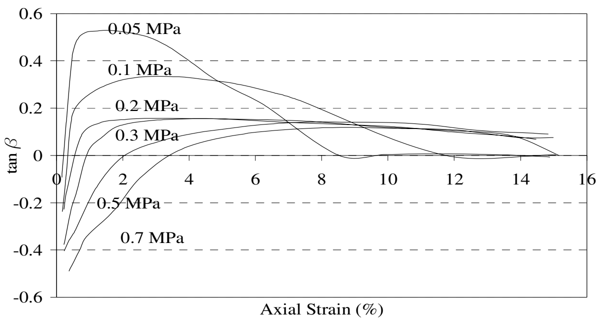

Figure 3. Dilatancy angle versus axial strain for different confining pressures

As is evident from the results, the dilatancy angle varies considerably during axial loading and is very much pressure dependent.

The calculated values of dilatancy angle for axial strains less than %1 is insignificant (Figure 3), since the share of elastic strains in for this state is quite considerable. However, samples under low confining pressures exhibit marked peak values of dilatancy at axial strains of about %2, while samples under higher confining pressures tend to contract initially and later on at larger axial strains dilate slightly. Only one specimen did not dilate at all.

As expected, at large strains, the dilatnacy angle tend to minute values. In general, the obtained results conform to the known trend of behavior.

The angle of friction has also been calculated from the results and its variation with confining pressure is discussed in the following section.

5 DISCUSSION

The failure envelop of coarse-grained soils, in particular, has a pronounced curvature. The non-linearity is very noticeable at low confining pressures and also when shear strengths at very different confining pressures are measured. However, since in practice it is generally preferred to retain the much simpler linear Coulomb failure criterion instead of deploying the more cumbersome non-linear failure envelopes, the reduction in the rate of increase of shear strength with confining pressure is achieved by reduction of the friction angle with increase of the confining pressures.

Many researchers including Indraratna et el (1993) have shown that the friction angle generally decreases with the increase of the normal stress.

The evaluated friction angles in this study are shown (the hatched area) alongside the data presented by Indraratna et el (1993) in figure 4.

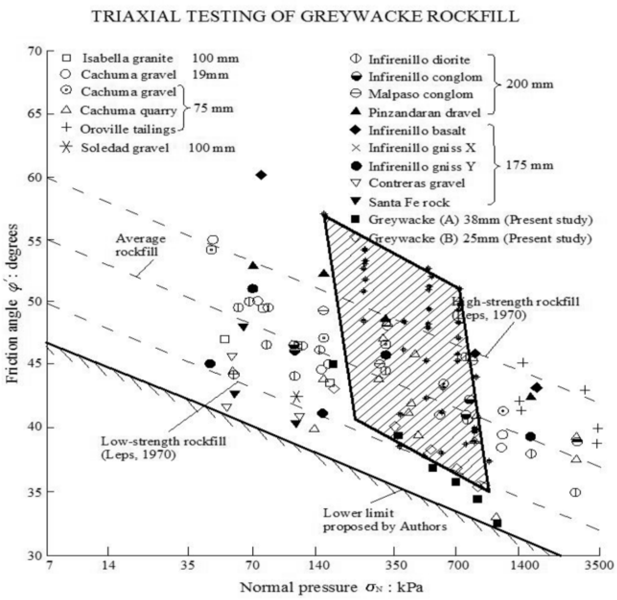

Figure 4. Friction angle versus normal pressure (Indraratna et el 1993)

A logarithmic relationship of the following from would be obtained if the evaluated peak friction angle is plotted against the stress factor (Jalali 1988):

$$\varphi = 16.5 \cdot Ln\beta_f + 30.4 \qquad (2)$$

Where stress factor $\beta_f = \left(\dfrac{\sigma_1 - \sigma_3}{2\sigma_3}\right)_f$

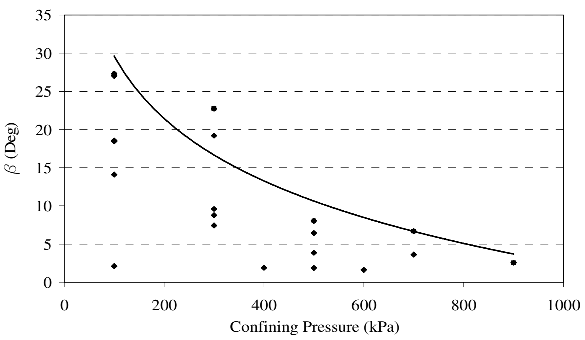

Figure 5. Variation of dilatancy angle versus confining pressure

In spite of the moderate scatter, it can clearly be noted that the trend is positive and dilatancy angle may approximately be expressed by $260I_{S50}/\sigma_3$.

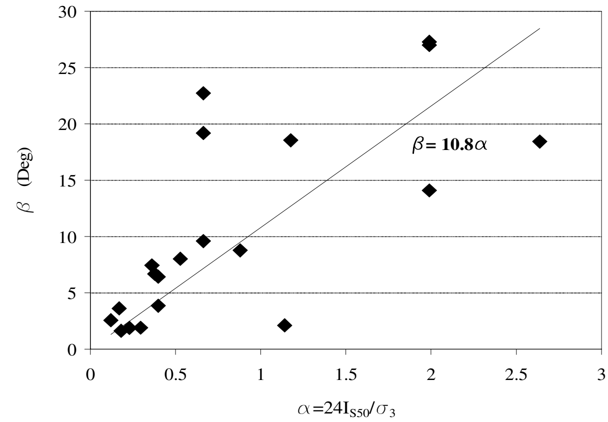

Figure 6. Dilatancy angle versus I_{S50}

Since dilatancy angle is dependent on peak angle of friction, it may be concluded that dilatancy angle must also be inversely related to confining pressure. This hypothesis is confirmed by the experimental data. The variations of maximum dilatancy angle with confining pressure are shown in figure 5.

The abrasion test data was limited to only three of the test samples. However, the expected trend was encountered. The dilatancy angle decreased with increase of abrasion weight loss. The trend is shown in figure 7 for tests with confining pressures of 300kPa.

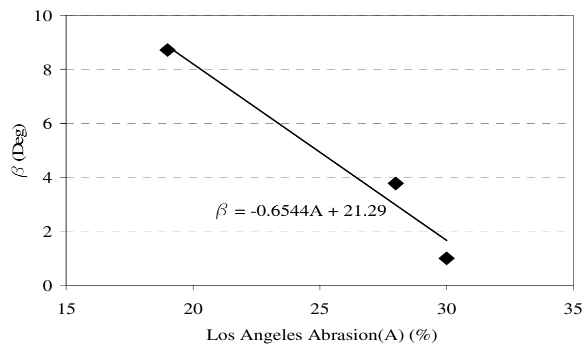

Figure 7. Dilatancy angle versus abrasion weight loss at σ_3=300kPa

6 CONCLUSION

It has been re-exerted that the dilatancy angle varies considerably during loading and is very much dependent on the confining pressure.

It has further been shown that the amount of dilation is directly related to particle strength and its resistance to abrasion for angular shaped particles in rockfill mass. Stronger geo-materials tend to dilate more under a constant (low) confining pressure than less strong rockfills.

This has been attributed to the resistance of the grain to crushing and breakage. Thus, increasing particle breakage causes reduction of dilatancy angle. Particle's resistance to crushing and breakage is very much dependent on the strength and hardness of the grain.

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