THE INTERPRETATION OF LABORATORY SOIL TESTS

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ABSTRACT
This paper establishes relations among the friction angle of cohesionless soils tested under direct shear, triaxial compression and plane strain compression conditions based on both theoretical and experimental studies. The stress level as well as the density and fabric of soil are found to have significant effect on the relations between the various angles of friction ϕ. The direct shear friction angle ϕ_{ds} may be 10 to 15% either larger or smaller than the triaxial friction angle ϕ_{tc}, while the plane strain angle ϕ_{ps} may exceed ϕ_{tc} by approximately 10 to 15% for dense sand. The fabric of soil tends to further increase the discrepancy between the angles of friction from different testing methods. Design soil strength parameters should be obtained using a testing method that can reproduce the *in-situ* stress path as accurately as possible. Experimental friction angles (e.g. ϕ_{tc} and ϕ_{ds}) may not be applicable as design parameters directly.

RÉSUMÉ
Interprétation des essais en laboratoire sur les sols.

1. INTRODUCTION

Evaluation of soil strength parameters is no trivial task. The trace of steady accumulation of knowledge of soil properties can be found in a number of major overview papers (e.g. Ladd et al., 1977; Wroth, 1984; Jamiolkowski et al., 1985 and Tatsuoka, 2000) and in a major design manual on estimating soil properties (Kulhawy and Mayne, 1990). These documents have shown the increasing sophistication in the evaluation of soil properties and the interpretation of experimental data from various tests. The situation becomes more complex when the anisotropic behaviour of soil is considered. While sophisticated constitutive models tend to improve our ability to describe the behaviour of soils, some material parameters in sophisticated soil models may not have clear physical meaning. As such, engineers prefer to use simple parameters such as the cohesion and the angle of friction of soil for most engineering designs.

Various laboratory strength tests, such as triaxial compression (TC), triaxial extension (TE), direct shear (DS), simple shear (SS) and plane strain compression (PS), have been developed in the past to simulate the deformation and failure conditions met in field. All these tests can be used to estimate strength parameters, depending on *in-situ* conditions. However, owing to the differences in testing conditions, the results should be different. Furthermore, no one type of test usually addresses all actual field stress/strain conditions, as illustrated in Figure 1. Instead, a combination of different types of tests is required to obtained soil strength parameters for the various conditions. If the selected strength parameter for design does not match a specific field situation, the estimated factor of safety could be significantly different from the actually value. Since not all geotechnical laboratories have the capability of conducting various tests of soil strength, it is practically important to establish relations among soil strengths measured from different tests corresponding to various filed conditions.

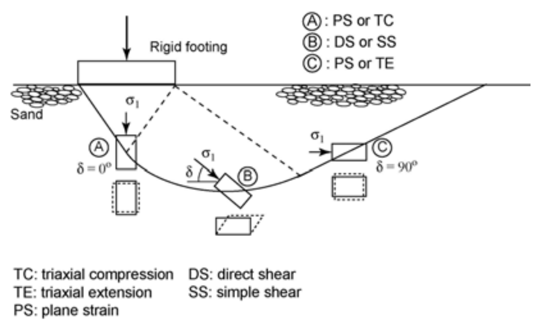

Figure 1. Relevance of laboratory strength tests to field conditions

The issues just mentioned have been a long-lasting topic with both desk and experimental studies conducted. Rowe (1969) was the first to establish the relation between the plane strain angle of friction ϕ_{ps} and the direct shear angle of friction ϕ_{ds} for dilatant soils, deriving the elegant relation

$$\tan\phi_{ds} = \tan\phi_{ps}\cos\phi_{cv} \qquad [1]$$

with ϕ_{cv} being the angle of friction at critical state. By assuming that the horizontal direction in the direct shear test is the direction of zero linear incremental strain and coaxility of σ_1 and $d\varepsilon_1$. Jewell and Worth (1987) suggested that ϕ_{ps} and ϕ_{ds} can be related via the angle of dilation and found that $\tan\phi_{ps}$ is about 20 to 25% greater than $\tan\phi_{ds}$. The effect of non-coaxility between σ_1 and $d\varepsilon_1$ has also been investigated (Tatsuoka, 1985; Jewell, 1987). Based on the analysis of extensive data of the strength and dilatancy of different sand under triaxial and plane strain conditions, Bolton (1986) proposed the following empirical relations to estimate the plane strain angle of friction ϕ_{ps} and the triaxial compression angle of friction ϕ_{tc}:

Plane strain: $\phi_{ps} - \phi_{cr} = 0.8\psi_{\max} = 5I_R$ [2]

Triaxial strain: $\phi_{tc} - \phi_{cr} = 3I_R$ [3]

$I_R = I_D(10 - \ln p') - 1$, $\psi_{\max}(^0) = 6.25 I_R$ [4]

with I_D being the relative density defined as $I_D = (e_{\max} - e)/(e_{\max} - e_{\min})$. Combining Equations [2] and [3] yields

$$\phi_{tr} = \frac{1}{5}\left(3\phi_{ps} + 2\phi_{cr}\right) \text{ or } \phi_{ps} = \frac{1}{3}\left(5\phi_{tr} - 2\phi_{cr}\right) \quad [5]$$

By applying an associated flow rule to Matsuoka-Nakai failure criterion (Matsuoka and Nakai, 1982), Worth (1984) suggested

$8\phi_{ps} = 9\phi_{tc}$ [6]

One shosuld also be aware that granular soils may exhibit significant strength anisotropy established during the deposition of the soil; see, for example, Tatsuoka et al. (1986) among others. Since Eqs. [1] through [6] are based on the assumption that the shearing behaviour of soil is isotropic, care must be exercised when implementing these equations to interpret laboratory test data.

The objective of this paper is to examine the impact of dilatancy and fabric (or strength anisotropy) on the correlations between the angles of friction measured in different tests. Within the framework of plasticity theory, the relation between ϕ_{ps} and ϕ_{tc} for isotropic granular soils is first re-appraised for taking the effect of dilation into account. The analytical results are then compared with experimental data. The strength anisotropy of granular soils under both direct shear and plane strain conditions is studied by introducing a second-order fabric tensor, and the relations between ϕ_{ps}, ϕ_{ds} and ϕ_{tc} for anisotropic soils are derived and the results are compared with experimental data.

The following symbols are used in this paper to denote the angle of friction in different tests: ϕ_{tc} = triaxial compression, ϕ_{te} = triaxial extension, ϕ_{ps} = plane strain; ϕ_{ds} = direct shear and ϕ_{ss} = simple shear. Furthermore, all stresses are referred to as effective stresses.

2. CORRELATIONS OF FRICTION ANGLES FOR ISOTROPIC SOILS

2.1 Triaxial compression and plane strain

Let us consider the Matsuoka-Nakai criterion for the failure condition of granular materials (Matsuoka and Nakai, 1982)

I_1I_2/I_3=constant [7]

where $I_1=\sigma_1+\sigma_2+\sigma_3$, $I_2=\sigma_1\sigma_2+\sigma_2\sigma_3+\sigma_1\sigma_3$ and $I_3=\sigma_1\sigma_2\sigma_3$. For the particular case of triaxial compression for which $\sigma_2=\sigma_3$, Eq. [7] can be expressed as

$$\frac{I_1I_2}{I_3} = \frac{(3-\sin\phi_{tc})(3+\sin\phi_{tc})}{(1-\sin\phi_{tc})(1+\sin\phi_{tc})} = 9 + 8\tan^2\phi_{tc} \quad [8]$$

Satake (1982) has shown that if an associated flow rule is applied to Matsuoka-Nakai criterion, then for plane strain conditions, ϕ_{ps} is the maximum value that $\tan\phi$ can have for all values of $b = (\sigma_2-\sigma_3)/(\sigma_1-\sigma_3)$. By finding the maximum value of the ratio σ_1/σ_3 (i.e. the maximum value of obliquity) for a fixed value of I_1, one obtains

$$\sec^2\phi_{ps} + \sec\phi_{ps} = 2\sec^2\phi_{tc} \quad [9]$$

which can be approximated for engineering purposes by the linear relation given in Eq. [6].

When a non-associated flow rule is assumed, the plastic potential function can be expressed as (Vermeer, 1982)

$$g = I_1^*I_2^* - KI_3^* = 0, \quad K = 9 + 8\tan^2\psi \quad [10]$$

where ψ is the angle of dilation, I_1^*, I_2^* and I_3^* are the invariants of a stress tensor defined as $\sigma_i^* = \sigma_i + a$ with a being determined from g = 0. The variation of $\phi_{ps} \sim \phi_{tc}$ relation is studied by considering an elasto-perfectly plastic material with elastic shear modulus G = 25 MPa and Poisson's ratio v = 0.25. Figure 2 presents the relation between ϕ_{tc} and ϕ_{ps} at different ψ. It is found that $\phi_{ps} = \phi_{tc}$ for ψ = 0 and ϕ_{ps}/ϕ_{tc} tends to increase with dilation angle ψ. The variation of ϕ_{ps}/ϕ_{tc} with the angle of dilation in Figure 2(a) can be approximated as

$$\phi_{ps}/\phi_{tc} = 1 + (0.10 \sim 0.15)\psi_{\max}/\phi_{tc} \quad [11]$$

which states that ϕ_{ps}/ϕ_{tc} increases with ψ/ϕ_{tc} and gradually approaches 1.1 when $\psi = \phi_{tc}$. This relation is consistent with Cornforth's experimental data (Cornforth, 1964), as shown in Figure 2(b).

A similar trend is obtained from the empirical relations in Eqs. [2] and [3]

$$\phi_{ps}/\phi_{tc} = 1 + 0.32\psi_{\max}/\phi_{tc} \quad [12]$$

which yields a more significant increase of ϕ_{ps}/ϕ_{tc} with the angle of dilation ψ. The inconsistency between Eqs. [11] and [12] might be due to the fact the Eq. [12] is based on experimental data in which the effect of fabric is embedded. Since both density and stress level affect the angle of dilation of granular soils, the difference between ϕ_{ps} and ϕ_{tc} will also vary with density and stress level, as shown in Figure 3. As such, one may conclude that Eq. [6] might overestimate the value of ϕ_{ps} for a given ϕ_{tc}, especially for the cases in which higher stress level is encountered or the soil is relatively loose.

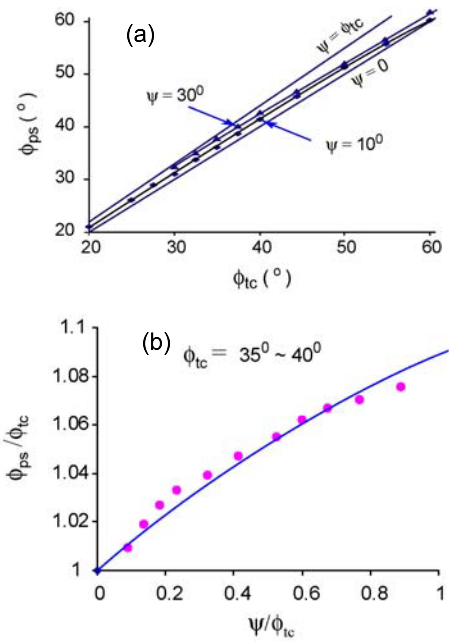

Figure 2. (a) ϕ_{ps} - ϕ_{tc} relation at different angles of dilation; and (b) Variation of ϕ_{ps} /ϕ_{tc} with the angle of dilation

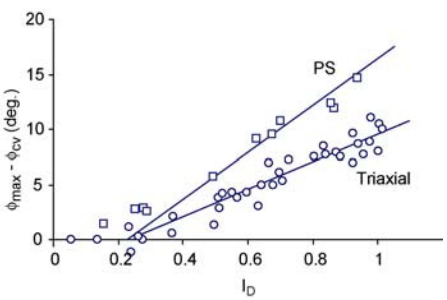

Figure 3 Data for sands at failure with p' in the range of 150-600 kPa (After Bolton, 1986)

2.2 Direct shear and triaxial compression

Jewell and Wroth (1987) suggested that the direct shear angle of friction $\phi_{ds} = \tan^{-1}(\tau_{yx}/\sigma_{yy})_{\max}$ that is the stress ratio at failure on the horizontal plane can be related to the plane strain angle of friction ϕ_{ps} by

$$\tan\phi_{ds} = \frac{\sin\phi_{ps}\cos\psi}{1-\sin\phi_{ps}\sin\psi} \quad [13]$$

The introduction of Rowe's stress-dilatancy formulation

$$\sin\psi = \frac{\sin\phi - \sin\phi_{cv}}{1+\sin\phi\sin\phi_{cv}} \quad [14]$$

yields Eq.[1]. On the other hand, if the associated flow rule is used, i.e. $\psi = \phi_{ps}$, Eq. [13] yields $\phi_{ds} = \phi_{ps}$.

Based on Eqs. [1] and [6], Kulhawy and Mayne (1990) recommended a correlation between ϕ_{ds} and ϕ_{tc} as

$$\tan\phi_{ds} = \tan(1.10\phi_{tc})\cos\phi_{cv} \quad [15]$$

For most granular soils, the typical value of friction angle at critical state is in the range of 30^0 to 35^0. When ϕ_{cv} = 32^0, ϕ_{ds} obtained from Eq. [15] is very close to ϕ_{tc}. Furthermore, Eq. [15] indicates that the correlation between ϕ_{ds} and ϕ_{tc} is unique for a given granular soil, since Eq. [15] only involves the critical angle of friction ϕ_{cv} that is usually considered as independent of void ratio and stress level. However, comparison of ϕ_{ds} calculated from Eq. [13] with ϕ_{tc} in [15] indicates that the direct shear angle of friction ϕ_{ds} may be larger or smaller than the triaxial friction angle ϕ_{tc}, depending on the value of ϕ_{tc} and the angle of dilation ψ (see Figure 4). Recalling that the angle of dilation ψ varies with void ratio and stress level, the ratio ϕ_{ds}/ϕ_{tc} is expected to vary with void ratio and stress level rather than following Eq. [15]. This conclusion is further confirmed by comparison of ϕ_{ds} from Eq. [15] with ϕ_{tc} from Bolton's empirical relation, Eq. [3].

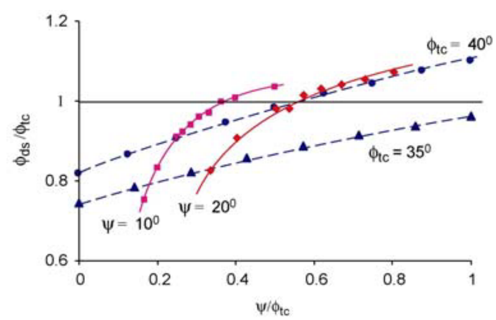

Figure 4: Comparison of ϕ_{ds} and ϕ_{tc} using Eq. [15] at ϕ_{cv} = 32°

Figure 5 further compares the relations between ϕ_{ds}, ϕ_{ps} and ϕ_{tc} by plotting Eq. [13] and the results of theoretical analysis shown in Figure 2. For a given angle of dilation, ϕ_{ps}/ϕ_{ds} tends to increase with ϕ_{ps}, while ϕ_{ps}/ϕ_{tc} shows an opposite trend. However, for most engineering soils with ϕ_{ps} = 30^0 – 40^0 and ψ = 10^0 – 20^0, ϕ_{ds} is very close to ϕ_{tc}. Both Figure 4 and Figure 5 clearly show that the difference between ϕ_{ds} and ϕ_{tc} varies with both the angle of friction and the angle of dilation ψ. Depending on the value of ψ, ϕ_{ds} may be smaller or larger that ϕ_{tc}. More specifically, the ϕ_{ds} tends to be larger than ϕ_{tc} for soils with large friction angle and/or large dilation angle.

According to Figure 4 and Figure 5, one may conclude that the difference between ϕ_{ds} and ϕ_{tc} depends on the stress level and the density of the soil. A series of triaxial and direct shear tests were carried out using Ottawa standard sand (C109) and a quartz sand. The shape of particles of Ottawa sand and the quartz sand are rounded and angular, respectively. All specimens were prepared using wet-tamping method to obtain the required density of the specimen. For drained triaxial compression tests, the effective confining pressure ranged from 50 kPa to 250 kPa, while the normal stress applied on the sample in the direct shear tests were in the range of 50 kPa to 105 kPa. From the results presented in Figure 6 (a), one observes that when the relative density I_D is less than 78%, ϕ_{ds} is smaller than ϕ_{tc}. However, for dense specimens with I_D higher than 78%, ϕ_{ds} is larger than ϕ_{tc}. The maximum difference between ϕ_{ds} and ϕ_{tc} reaches 4°, which can be significant for an engineering design. The test results of the angular, crashed quartz sand shown in Figure 6 (b) also show noticeable difference between ϕ_{ds} and ϕ_{tc}. Different from the results of Ottawa sand with mainly rounded particles, the direct shear friction angle ϕ_{ds} for the quartz sand is larger than ϕ_{tc} for all samples tested. It is likely that the fabric of soil specimens is responsible the difference between the results in Figure 6 (a) and Figure 6 (b). Rowe's experimental data (Rowe, 1969) show the same trends, as presented in Figure 6 (c).

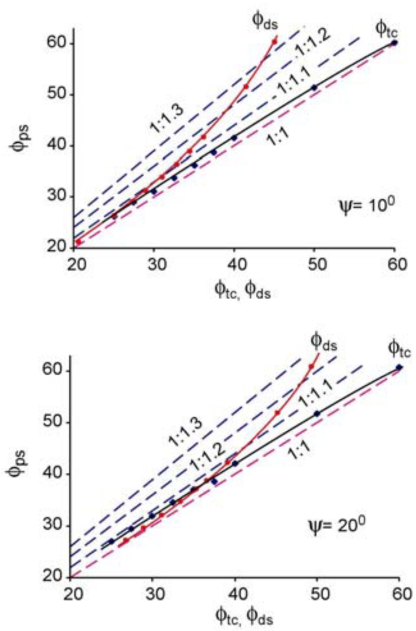

Figure 5: Effect of dilation angle

Owing to the differences of boundary conditions, the angle of friction in direct shear, ϕ_{ds}, is close to, but not exactly the same as, the angle of friction in simple shear ϕ_{ss}. It was reported that ϕ_{ss} is usually less than the angle of friction in triaxial compression, ϕ_{tc}, by about 15% when the bedding plane is horizontal (Tatsuoka et al, 1986 and Pradhan et al, 1988).

3. EFFECT OF FABRIC OF SOIL FRICTION ANGLES

Granular soil can exhibit significant strength anisotropy due to the deposition of the soil, and owing to the fact that soil particles may have some preference orientation. The soil properties (both the shear resistance and stiffness) thus depend on the direction with respect to the deposition, or the bedding planes, in which the soil is subsequently sheared. This section will discuss the friction angle for anisotropic granular materials.

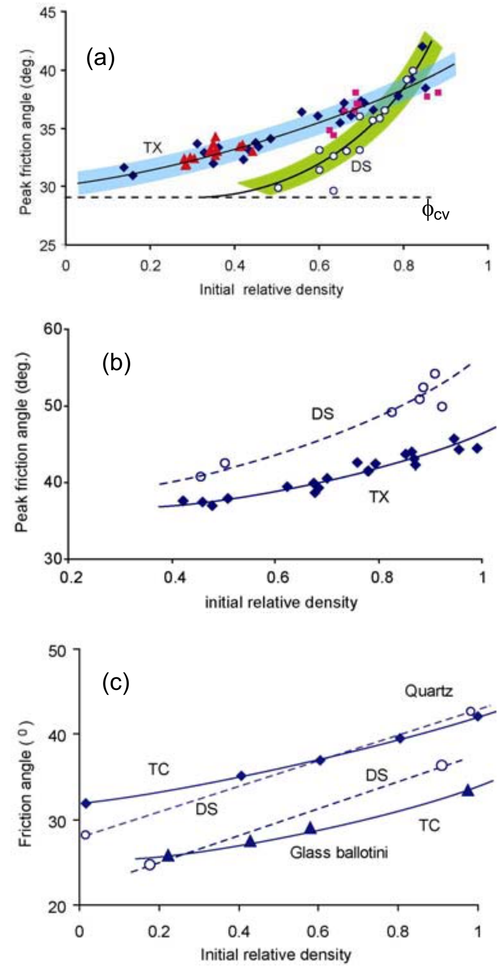

Figure 6 Comparison of measured ϕ_{ds} and ϕ_{tc}: (a) Ottawa standard sand (C109); (b) Quartz sand (angular); (c) Quartz and Glass ballotini

3.1 Direct shear friction angle for anisotropic soils

Theoretically, the directional dependency of the coefficient of soil friction m ($=tan\phi$) and the cohesion c can be described by a second tensor (Pietruszczak and Mroz, 2001). For granular material, a micromechanical analysis (Guo and Stolle, 2004) on the distribution of particle contact normals reveals that the direct shear friction angle of anisotropic soils can be related to fabric via

$$m_\theta = m^{iso}(1 - \omega \cos 2\theta) \quad [16]$$

where m^{iso} is the coefficient of friction for the "isotropic" material, θ is the angle between the major principal direction of contact normals to the shearing plane, ω is a factor describing the distribution of the contact normals. The coefficients of friction when shearing occurs along and perpendicular to the bedding plane are then expressed as

$$m_h = m^{iso} - \omega, \; m_v = m^{iso} + \omega \quad [17]$$

respectively, which further yields

$$m_\theta = m_h + (m_v - m_h)\sin^2\theta \qquad [18]$$

The anisotropic parameter ω is related to m_v and m_h via

$$\omega = (m_v - m_h)/(m_v + m_h), \;\; m_v/m_h = (1+\omega)/(1-\omega) \qquad [19]$$

Mahmood and Mitchell (1974) performed directed shear tests on crashed basalt by pouring through the side and the top of the shear box. In the case of samples poured from the top (in the conventional way), the sample was sheared along the bedding plane. When a sample was prepared by pouring from the side, shearing took place perpendicular to the bedding plane. For samples with the relative density I_d = 62% and 90%, the values of the anisotropic factor ω determined from the measurement of particle orientation were found to be 0.23 and 0.06, respectively. According to Equation [19], the resulting m_v/m_h ratio is 1.60 and 1.12 respectively. The comparison of the experimental data and the calculated m_h from m_v shows that Equation [16] or Equation [18] could be used to describe the directional dependency of direct shear friction angle.

Assadi (1975) carried out the same type of directed shear tests as Mahmood and Mitchell (1974) on dense samples of Leighton Buzzard sand (void ratio e = 0.52) under a vertical stress of 42 kPa. When samples were prepared by pouring from the side of the shear box, the angle of friction was 48°, which was about 3° higher than the case in which the sample was poured through the top of the shear box. The experimental is also consistent with Equation [19] when ω = 0.06 is assumed.

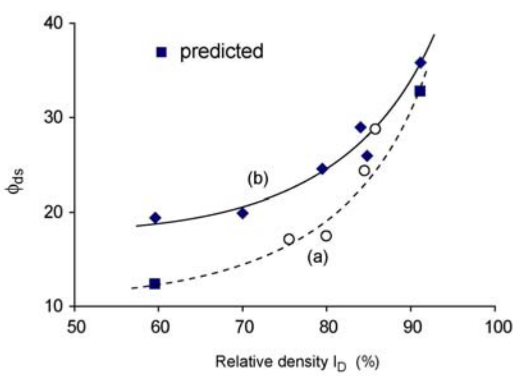

Figure 7 Direct shear tests on crushed basalt: (a) shear along bedding plane; (b) shear across bedding plane (Experimental data after Mahmood and Mitchell, 1974)

It is important to note that direct shear test on a sample deposited through the top of the shear box develops a horizontal bedding plane, and thus corresponds closely with the minimum plane strain shearing resistance (Tatsuoka et al, 1986). Consequently, one should expect that the shear friction angle obtained from a conventional direct shear test will yield a plane strain angle of friction close to the minimum value of the soil.

It should be mentioned that the direction dependence of friction described by Equation [18] is the same as that of the cohesion c

$$c_\theta = c_h + (c_v - c_h)\sin^2\theta \qquad [20]$$

proposed by Lo (1965).

3.2 Correlation between ϕ_{ps} and ϕ_{ds} for anisotropic soils

From micromechanics analysis in which the fabric is described by the distribution of contact normals, the plane strain friction angle ϕ_{ps} may be expressed as

$$\sin\phi_{ps} = \sin\phi_{ps}^{iso} + \frac{1}{2}K_\delta\omega\cos\phi_{ps}^{iso} \qquad [21]$$

with

$$K_\delta = \frac{\cos(\phi_{ps}+2\theta)}{1+\omega\sin(\phi_{ps}+2\theta)} \qquad [22]$$

where ϕ_{ps}^{iso} is the friction angle for isotropic soils, ω the second invariant of a fabric tensor describing anisotropy of granular soil, and θ is the bedding plane angle that is the same as the angle between the direction of major principal stress and the normal of the bedding plane, It should be noted that the normal of the bedding plane coincides with the direction of the major fabric component. Rearrangement of Equation [21] yields

$$\tan\phi_{ps}^{iso} = \frac{\sin\phi_{ps}}{\cos\phi_{ps}^{iso}} - \frac{1}{2}K_\delta\omega \qquad [23]$$

According to Equation [16], the coefficient of friction corresponding to the friction angle of the "isotropic" material can be expressed as

$$\tan\phi_{ds}^{iso} = \tan\phi_{ds}/(1-\omega\cos 2\theta) \qquad [24]$$

Inserting Equations [23] and [24] into Equation [1], after some algebraic manipulations taking Equation [17] into account, one obtains

$$\tan\phi_{ds} = \left(\frac{\sin\phi_{ps}}{\cos\phi_{ps}^{iso}} - \frac{1}{2}K_\delta\omega\right)(1-\omega\cos 2\theta)\cos\phi_{cv} \qquad [25]$$

The angle of friction for an "isotropic" soil sample can be approximately estimated from

$$\sin\bar{\phi}_{ps}^{\perp} \approx \sin\phi_{ps}^{iso} + \frac{\omega}{2}\frac{\cos\bar{\phi}_{ps}^{\perp}\cos\phi_{ps}^{iso}}{1-\left(\omega\sin\bar{\phi}_{ps}^{\perp}\right)^2} \qquad [26]$$

with $\sin\bar{\phi}_{ps}^{\perp}$ and $\cos\bar{\phi}_{ps}^{\perp}$ being the average of $\sin\phi_{ps}$ and $\cos\phi_{ps}$ corresponding to horizontal and vertical bedding planes, respectively. For the purpose of engineering application, ϕ_{ps}^{iso} could be replaced by $\bar{\phi}_{ps}^{\perp}$ without significant error. Equation [25] is consistent with the experimental data of Pradhan et al (1986) when the bedding plane is horizontal, as shown in Figure 8.

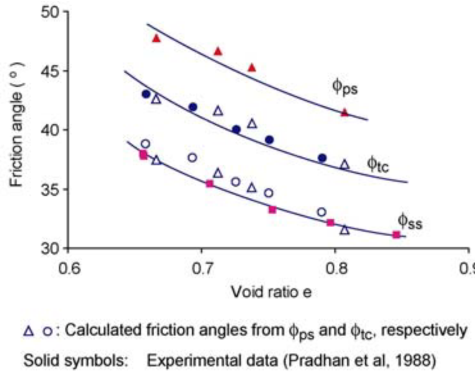

Figure 8: The angle of friction for anisotropic soil

4. PRACTICAL APPLICATIONS

As discussed in the previous sections, the angle of friction of soils highly depends on the testing method and the fabric of the material. It is equally important for any engineering design to interpret experimental data and then select design soil strength parameters correctly. This section gives some examples to demonstrate the selection of soil parameters depending on the feature of given problems.

The angle of friction plays a crucial role in determining the bearing capacity of footings. Various theoretical solutions are available, however, without clarifying how the friction angle is obtained. Figure 9 presents experimental results (Ingra, and Baecher, 1983) for rough footings with length-width ratios of L/B = 1 and 6, respectively. In the case of L/B =6, which essentially represents a strip footing, the bearing capacity factors calculated from different theoretical solutions using the angle of friction from triaxial compression tests systematically under-estimate the bearing capacity, even though both the experimental data and the theoretical solutions vary over a wide range. The discrepancy is significantly reduced if the plane friction angle ϕ_{ps}, which is approximately 10% larger than ϕ_{tc}, is used for theoretical solutions. On the other hand, for footings with L/B = 1, the use of the triaxial friction angle ϕ_{tc} yields reasonable theoretical predication. Consequently, the friction angle used for calculating the bearing capacity of footing should vary between ϕ_{tc} and ϕ_{ps}, depending on the length-width ratio L/B. Meyerhof (1963) suggested the following modification for the angle of friction

$$\phi = (1.1 - 0.1B/L)\,\phi_{tr} \qquad [27]$$

with the increase of L/B ratio, ϕ approaches $1.1\phi_{tr}$, which is close to ϕ_{ps}, the angle of friction under plane strain conditions.

When the soil mass has remarkable inherent anisotropy or when the ground has an obvious layered structure, care must exercised in the determination of soil parameters. Oda and Koishikawa (1979) investigated the effect of strength anisotropy on the bearing capacity of shallow footings on dense sand. In a series of small-scale model tests, a strip footing was placed on dense Toyoura sand with different orientations of bedding planes. As shown in Figure 10, even though the measured data are scattered, it is clearly observed that the average bearing capacity in the case of horizontal bedding plane is approximately 70% higher that for vertical bedding planes. According to Pradhan et al (1986), the variation of the bedding plane angle induce the angle of friction for Toyoura sand to change by approximately 10%, which causes the significant variation of bearing capacity with respect to the orientation of the bedding plane. As a particular example, the change of the friction angle from 38° to 42° corresponds to an increase in the bearing capacity factor N_q by approximately 75%, which matches the data shown in Figure 10.

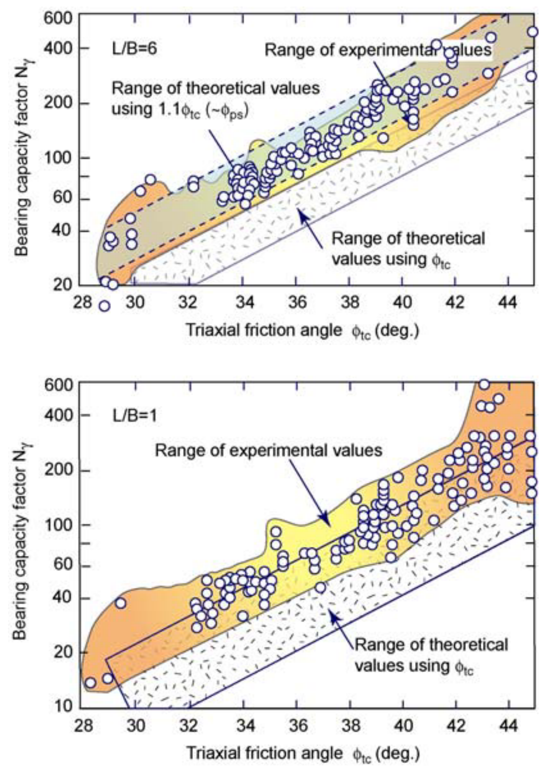

Figure 9: Bearing capacity factor: Experimental and theoretical results

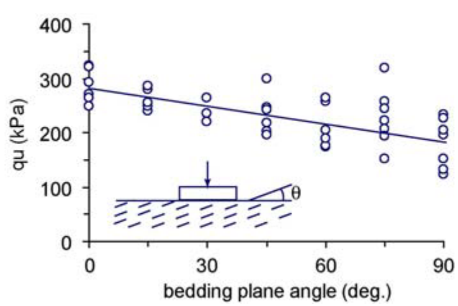

Figure 10: Influence of bedding plane angle on bearing capacity

5. CONCLUDING REMAKS

From both the theoretical and practical point of view, it is essential to appreciate the experimental characterisation of shear strength parameters and their appropriate use in the analysis of geotechnical engineering problems. The experimental shear resistance of granular soils varies with many factors, including soil properties (e.g., void ratio and fabric), deformation history and stress level, as well as the testing method. The following remarks will conclude the study presented in this paper:

(1) Theoretically, the angle of friction obtained from direct shear test might be questionable owing to the non-uniform distribution of stresses along the shear plane. Even though both analytical and experimental results show that direct shear friction angle ϕ_{ds} could be related to ϕ_{tc} and ϕ_{ps} via some elegant relations, care must be taken when these relations are used.
(2) Depending on the density and the fabric of the soil, the direct shear friction angle ϕ_{ds} may be larger or smaller than the angle of friction obtained from conventional triaxial compression tests, with the difference as high as 4°.
(3) Design soil strength parameters should be obtained using a testing method that can reproduce the *in-situ* stress path as accurately as possible. Experimental friction angles (e.g. ϕ_{tc} and ϕ_{ds}) may not be used as design parameters directly.
(4) Strength anisotropy should be taken into account when possible.

6. ACKNOWLEDGEMENTS

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