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Uncertainties in reference states in the determination and validation of soil parameters

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ABSTRACT The manifold soil behavior is usually simplified in geotechnical practice by considering constant parameters which dependent of the soil state. A check and evaluation of the parameters determined in the laboratory without regarding the soil state is hardly feasible, making an objective description of soil behavior impossible. In order to increase the reliability and validity of the parameter determination, a systematic procedure for the identification of soil parameters is proposed. The soil behavior is mainly evaluated on the basis of the "Critical State Soil Mechanics" (CSSM). Its parameters should be valid independently from the state of the soil. A reasonable verification can be achieved by plausibility check through a combined evaluation of shear strength and compressibility. For this purpose asymptotic conditions in the theory of critical states are considered. Uncertainties in the determination of the boundary conditions in laboratory tests are also emphasized and their influences are illustrated. Uncertainties may occur e.g. in the determination of the void ratio or the dip angle of the shear plain in a triaxial test and the resulting maximum shear stress. The advantage of the proposed approach is illustrated for chosen laboratory tests.

1 INTRODUCTION

The complex stress-strain and limit-stress behavior of soils is, in the geotechnical practice, usually described by parameters which values depend on factors including the state and history of the soil. These parameters may be measured in tests which correctly reproduce the important influence factors, but their evaluation and the valuation is hardly feasible and making an objective judgment of soil behavior sometimes impossible. Especially in geotechnics the number of laboratory investigations is limited due to a finite amount of soil samples and the evaluation of soil behavior is bases only on the results from a few tests.

In order to increase reliability and validity such material parameters should be used, which have a constant value for a particular soil and are independent of its state or history. The description of soil behavior should be related to predefined reference states, e.g. the asymptotic states of the "critical state soil mechanics". The question is, how exact can these reference states be determined and what is the influence of scatter of these parameters. In the following some examples for the reference states and their standard determination are described and some uncertainties are specified. Here, the focus is on the determination of void ratio and maximum shear stress in a triaxial test specimen. Also the influence on the resulting parameters is shown and discussed

2 EXAMPLES OF REFERENCE STATES

Reference states herein are referred to asymptotic boundary conditions. In the following some examples of reference states will be explained.

2.1 Normal compression line (NCL)

The first reference state is the compression line in normally consolidated state. In this state the soil is not prestressed and every stress increase is new for the soil. As described in Wood (1990) and Schofield (1968) this state can be defined by a line in a semi-logarithmic e-log stress diagram. For example for an oedometric compression by:

$$e = e_0 - C_c \cdot \log\left(\frac{\sigma'_v}{\sigma_{ref}}\right) \tag{1}$$

2.2 Critical State line (CSL)

The second reference state is the critical state Schofield (1968). The critical state is characterized by a constant shear stress without volume change during a continuous shear deformation. As shown in Figure 2 the deviatoric stress q over the mean effective pressure p' in the critical state can be described by a line

$$q = M \cdot p' \tag{2}$$

where M is the increase of the deviatoric stress over the mean effective pressure p'. The void ratio in the critical state is also characteristic and influenced by the mean effective stress. The void ratio in critical state has a similar trend as the normal compression line and can be defined by

$$e_c = e_{0c} - C_c \cdot \log\left(\frac{p'}{p_{ref}}\right). \tag{3}$$

3 UNCERTAINTIES IN THE DEFENITION

3.1 Uncertainties in void ratio

One point in the description of soil behavior in the reference state concept is a unique void ratio – stress relation. But in this case a unique void ratio has to be measured.

Fluctuations in the void ratio can originate from two sources. The first case is a measurement uncertainty in the lab. Here for two samples, with the same real void ratio e, two different void ratios e_1 and e_2 can be determined because of measurements errors in the laboratory. The second case is the natural fluctuation of the void ratio. Here different samples from the same soil and excavation depth have different void ratios e_a, e_b or e_c. If the void ratios are not determined for every sample, one value e_a, e_b or e_c would be assumed for all samples.

To quantify the fluctuations in the case of natural inhomogeneity several data from the literature can be found, e.g. Mitchel (1976), Gasparre (2007). Differences up to 15% were reported.

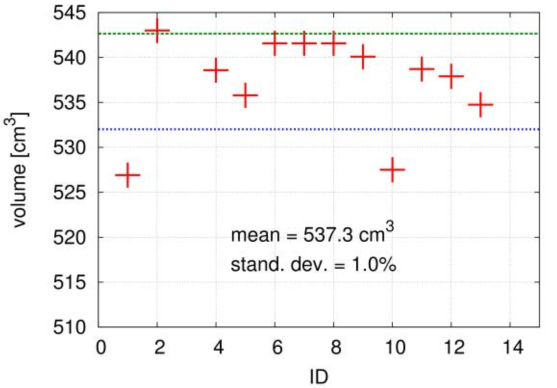

Figure 1. Volume of a test cylinder measured by different lab workers (ID).

For the uncertainties in the lab, the calculation of the void ratio e = (ρ_s/ρ_d)-1 can be checked. As shown in Vaid (1996) serious errors in assigning void ratio can occur due to the low resolution in measurements of the volume. To check that, a cylinder made of silicon rubber was used to demonstrate the scatter of density which follows from the measured volume and mass. As shown in Figure 1 the scatter in the measured volume was about 1.0%, which is much larger than the scatter of the mass being about 0.2%. This can be explained by a high accuracy of the scale. On the contrary, the volume was determined by measuring the diameter and the height with a caliber. Here the accuracy was approximately 0.05 mm. And because of the soft silicon material (which can represent a soft soil in case of volume determination with a caliber) the accuracy was mostly influenced by the sensitive handling of the lab worker.

The other component, which influences the determined void ratio, is the grain density. To evaluate the scatter, values for the grain density of one soil were brought together from different experimental series in the soil mechanical laboratory of the TU Dresden. Here the determined standard deviation was 0.7% (see Fig. 2).

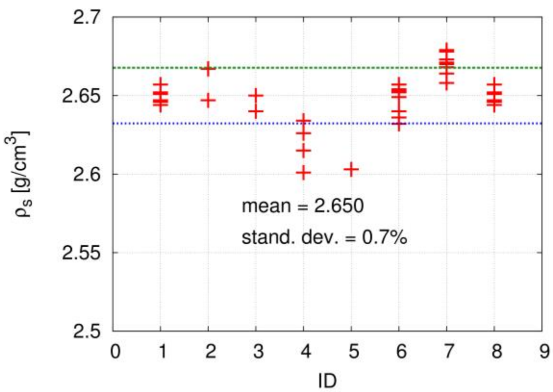

Figure 2. Grain densities of the tested soil.

The scatter of the volume, mass and the grain density results, in this described test series, in the void ratio scatter up to 3.0%. This means that it is not possible to determine a unique void ratio in the laboratory. The stress-void ratio relation is more a bandwidth than a single line. Figure 3 shows this for the case of the NCL from several oedometer tests which were conducted with the same reconstituted soil at different initial water contents. It is obvious that for all tests the material behavior tends to an asymptotic state. But for single tests different normal consolidated states could be defined. When considering the variation in the initial void ratio of 3% all results lie over or under the mean line in the marked bandwidth.

If the difference in void ratio results from natural inhomogeneity the tested specimens have different shear strength. In a thought experiment, as illustrated in Figure 4, the same void ratio would have been assigned for two soil specimens, although the real void ratio is different and accordingly the shear strength is different. If the critical state would be determined by the asymptotic behavior of these specimens, as described in Georgiannou (2001) or Burland (1990), different maximum deviatoric stresses q_1, q_2 would be measured.

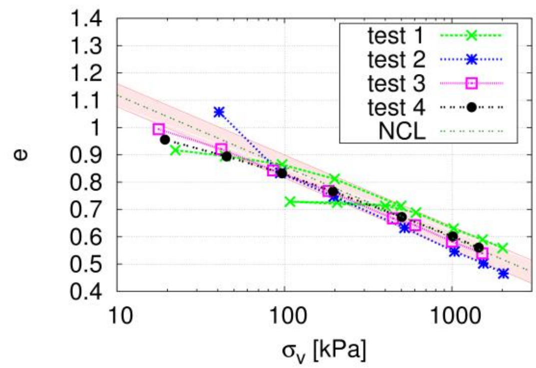

Figure 3. Bandwidth of NCL regarding the void ratio fluctuations.

The deviatoric stress q can be calculated from the critical state theory if some standard parameters (e_{0c} = 1.0, C_c = 0.2 and M = 1.0) are used:

$$q = M \cdot 10^{\left(\frac{e_{0c}-e}{C_c}\right)} \tag{4}$$

If a difference of void ratio *e* of 5% is assumed, a difference in *q* up to 30% can be calculated. The results of *q* are influenced by the chosen parameters e_{0c}, C_c and M. To get an idea of the influence of the chosen values for e_{0c}, C_c and M their values were increased by 20%. An increase of 20% in e_{0c} and M has only a minor influence on *q*, whereas an increase in C_c results in a smaller deviation of 25.7%.

If the critical friction angle is determined from the different deviatoric stresses q_1 and q_2 using:

$$\varphi_c = \arcsin\left(\frac{3 \cdot M}{6 + M}\right) \tag{5}$$

$$M = \frac{q}{p'} \tag{6}$$

A difference of 9° can be calculated. This variation in the friction angle shows how important it is to determine the void ratio for every single specimen and not only assign the same one from another specimen from the same soil type and excavation depth. If the shear strength of more than one specimen is investigated, which is the standard case, it is suspected that the influence of the natural inhomogeneity overlaps and the mean error in the friction angle is smaller and the resulting CSL may not run through the origin.

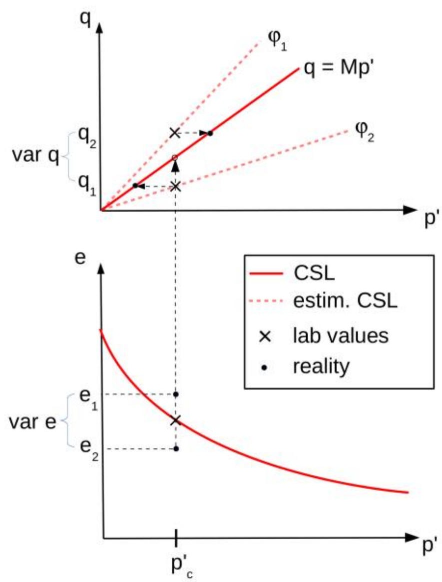

Figure 4. Sketch of resulting max. deviatoric stress for two specimens with different void ratios resulting from natural inhomogeneities.

3.2 *Uncertainties in inclination of slip surface*

A standard method to determine the maximum shear stress for soil specimen is the triaxial test. Diagrams of Mohr circles are a standard method to evaluate the Mohr-Coulomb limit stress condition (see Fig. 5):

$$\frac{\sigma_1 - \sigma_3}{2} = \frac{\sigma_1 + \sigma_3}{2} \cdot \sin(\varphi) + c \cdot \cos(\varphi) \quad (7)$$

Here a slip surface inclination of

$$\alpha_0 = 45° - \frac{\varphi}{2} \quad (8)$$

is assumed. As shown in Figure 5, the difference in the assumed inclination α_0 to the real inclination α can result in an over estimation of the shear stress τ. This influence is also described in Hvorslev (1960).

The following is mainly of interest, if a localized shear failure occurs in the specimen. This could be the case, when the critical state is assumed from over-consolidated specimens, which often show a localized shear failure, as described in Georgiannou 2001) or Burland (1990). Reconstituted soils normally don't show this behavior, but as described in Fearon (2000) also reconstituted soils can fail with a localized shear zone.

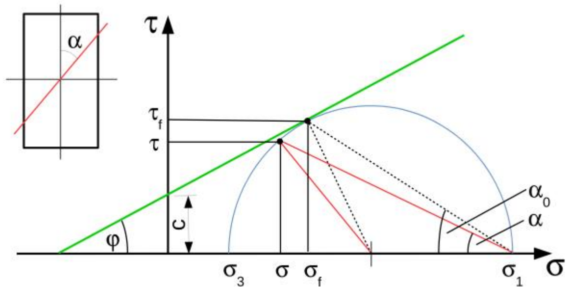

Figure 5. Sketch of definition of the stress envelope and the definition of τ_f/τ.

3.2.1 *Inclination of slip surface in laboratory tests*

To analyze the inclination of the slip surface, specimens from several undrained triaxial tests on two different soils have been photographed after the shearing phase. The tests were conducted on remolded specimen. From the pictures the dip angles of the failure plains were measured.

For soil 1, a high plasticity clay, a friction angle φ = 15° was determined. Consequently from the Coulomb-Theory (see Eq. 8) a dip angle α_0 = 37.5° should be expected. As shown in Figure 6, the dip angles α of the observed slip surfaces lie between 31° and 45°. The average difference to the expected value α_0 is 7°.

In the triaxial tests with soil 2, a low plasticity clay with high silt content, a friction angle φ = 35° was determined. Thus a dip angle α_0 = 27.5° is expected. In Figure 7 can be observed that the dip angles α lie between 42° and 47°. The average difference to the expected α_0 corresponds to 17°. These results show, that the theoretical value of α_0 (Eq. 8) does not correspond to the inclination of the shear bands in experiments.

3.2.2 *Influence of the dip angle on the maximum shear stress on the slip surface*

To evaluate the influence of the dip angle α on the maximum shear stress τ (Eq. 10) on the slip surface some calculation were carried out. For given principal stresses σ_1 and σ_3 the normal stress σ and the shear stress τ on the slip surface were calculated by:

$$\sigma = \frac{\sigma_1 + \sigma_3}{2} - \frac{\sigma_1 - \sigma_3}{2} \cdot \cos(2\alpha) \tag{9}$$

$$\tau = \frac{\sigma_1 - \sigma_3}{2} \cdot \sin(2\alpha) \tag{10}$$

This shear stress τ can be compared with the Mohr-Coulomb failure stress τ_f. From the Mohr circle (Fig. 5) follows:

$$\frac{\tau_f}{\tau} = \frac{1 - \sin(\varphi) \cdot \cos(2\alpha_0)}{\cos(\varphi) \cdot \sin(2\alpha_0)}. \tag{11}$$

As illustrated in Figure 8, the ratio from Eq. (11) depends on the difference in the dip angle

$$\Delta\alpha = \alpha_0 - \alpha \tag{12}$$

and on the determined friction angle φ of the soil. As shown in the previous section, the difference in the expected and measured dip angle $\Delta\alpha$ can be up to 17° in case of soil 2. With φ = 30° the difference from τ_f to τ can be up to 25 to 30%. To emphasize the difference in τ_f and τ the resulting ratio τ/σ was calculated. The results are shown in Figure 9.

From the above analysis follows that, the shear stresses in the specimen are overestimated in every case were $\Delta\alpha$ is not 0 (standard evaluation $\Delta\alpha$ = 0). For larger initial friction angles, e.g. φ=30°, the influence on τ/σ can be, if τ/σ is expressed in degree, up to 5°. This means, that the determined friction angle could be overestimated by 5°, if the inclination of the slip surface is not measured and taken into account.

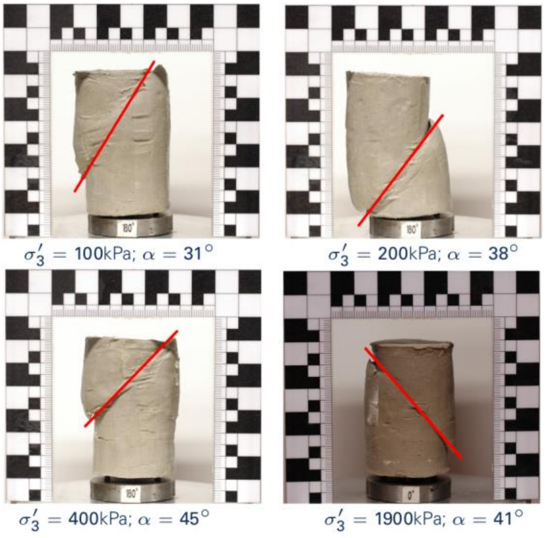

Figure 6. Slip surfaces after undrained triaxial shearing for soil 1 for different consolidation stresses

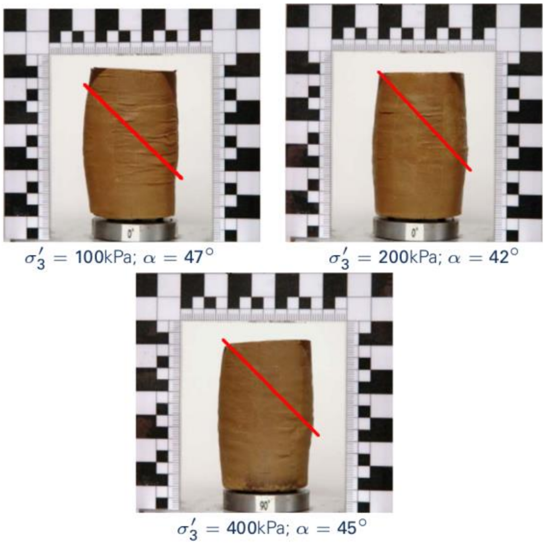

Figure 7. Slip surfaces after undrained triaxial shearing for soil 2 for different consolidation stresses

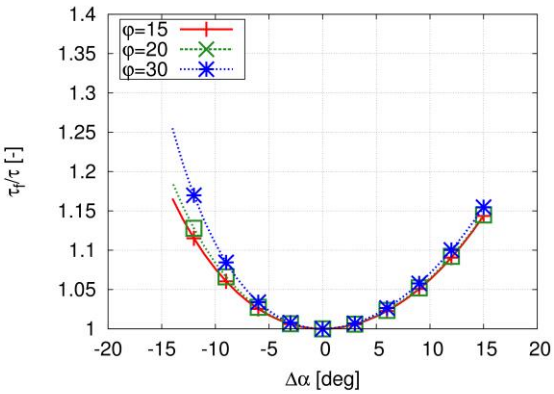

Figure 8. Mohr-Coulomb failure stress τ_f compared to the shear stress τ on the slip surface of a triaxial specimen for a given difference in the dip angle $\Delta\alpha$

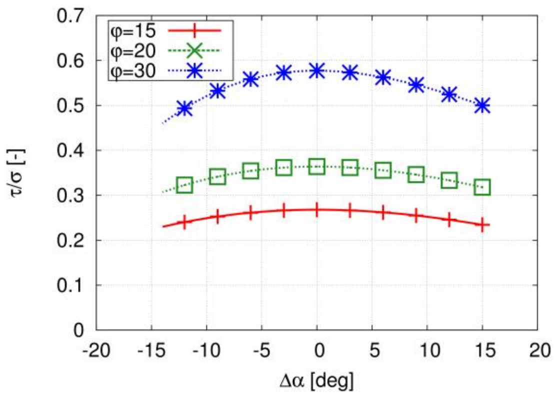

Figure 9. Stress ratio τ/σ depending on a given difference in the dip angle $\Delta\alpha$

4 CONCLUSION

It was described, that the evaluation of soil behavior can be done by the comparison to predefined reference states. Here examples of reference states were shown for the normally consolidated state and the critical state.

These introduced reference states assume a uniqueness which can not be achieved by measured laboratory data. It was pointed out that for the used specimen, e.g. a void ratio could only be determined with a variance of approximately 3% because of the low accuracy in the volume determination. This means, that it was not possible, under the described circumstances, to determine a measureable unique relationship between the effective stress and the void ratio. As a consequence of the natural scatter of void ratio, the determination of representative void ratio may be problematic. However, the value of void ratio has a pronounced influence on the maximum deviatoric stress. This lead to a misinterpretation of the friction angle.

And in the last part it was shown, that the inclination of the occurring slip plane in triaxial tests is not in every case the same as the expected from the Mohr-Coulomb theory. The difference between the theoretical shear stress and the calculated shear stress following from the inclination of the measured slip plain in a triaxial specimen can be up to 25%. Consequently, this can result in the overestimation of the value of the friction angle φ.

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