IOP Conference Series: Earth and Environmental Science



# Soil Parameters’ Influence on the Mechanical Behaviour considering Boundary Condition

IOP Conf. Series: Earth and Environmental Science **295** (2019) 042061 doi:10.1088/1755-1315/295/4/042061

# Soil Parameters' Influence on the Mechanical Behaviour considering Boundary Condition

**Binbin Xu[1,2,3,4,a] and Naishou Zhang[2]**

[1]Tianjin Port Engineering Institute Ltd. of CCCC, Tianjin, China

[2]CCCC First Harbour Engineering Co. Ltd., Tianjin, China

[3]Key Lab. of Geotechnical Engineering of Tianjin, Tianjin, China

[4]Key Lab. of Geotechnical Engineering, Ministry of Communication, Tianjin, China

[a]Corresponding author: Border@sohu.com

**Abstract.** In this paper, how to determine the soil parameters and initial values employed in the numerical calculation using experimental results is talked over in detail. The results of the triaxial compression tests and one-dimensional tests for undisturbed and disturbed samples are utilized step by step to modify the numerical results. It is found that there are several steps to model the whole procedure of sampling from the ground to the experiment equipment. Therefore, the step of taking the soil samples out from underground, which is assumed to be under the undrained condition accompanying with unloading process until the deviator stress becomes zero, is always ignored in the research.

## 1. Background

With the development of computational technology, the numerical analyses play more and more roles in geotechnical engineering. Usually, in order to acquire the reliable results one of the fundamental factors is the accuracy of the parameters employed in the calculation. Indoor experiments including triaxial compression test, one-dimensional test, and permeability test etc. are carried out to help get the de-tailed soil parameters. However, how to apply these experimental results into the numerical calculation is still debatable and there should be a criterion or bridge to connect two groups of parameters.

The usefulness of the calculation results is influenced significantly by the initial values of underground soil parameters and the precision of the material parameters. In order to decide the above values, a group of numerical calculation and back-analysis should be carried out considering the soil disturbance. For the constitutive model, totally about twelve soil parameters should be determined and those parameters can be divided into two groups. One group includes five parameters corresponding to the elastoplastic material, which is called the swelling index, intercept of NCL when the mean effective stress is 98.1kPa, compression index, critical state constant and Poisson's ratio. For the original Cam-clay model there are exactly five same parameters and the are derived from the triaxial compression tests and the one-dimensional tests. Another group contains seven parameters corresponding to the evolution of rules which are called the degradation parameters of structure, the development of stress induced anisotropy, the degradation parameters of overconsolidation ratio and the limitation of rotation. It should be explained that the significant characteristics of the constitutive model is that the soil response under different initial state can be represented by one single group of material constant for the given soil.

## 2. Determination of soil parameters

### *2.1 Triaxial test*

For the group of elasto-plastic soil parameters, the determination of the material constant can be obtained by the triaxial compression experiments and the one-dimension tests of the fully remolded samples. When the group of the soil parameters is determined, the group of the evolutional parameters are determined by reproducing the mechanical response of the soil samples and the initial values are derived by the similar way. Meanwhile, the triaxial compression experiment and the one-dimensional tests, together with other known initial values, should be carried out. The try and error method is used to predict the possible initial values and the optimum parameters. The ground survey datum provided by the design are used to evaluate the in-situ ground properties. For the clayey ground, the unconfined compression strength and the specific volume are provided usually and for the sandy ground the N value by CPT test, the void ratio and the bulk density are provided.

For the computation of the undrained triaxial compression tests, the reproduce stage is shown in Fig. 1 and there are totally four steps. The black dot is used to mark the initial state of the soil specimen and it can be seen that it lies on the line of $K_0$ state under the state of $K_0$ consolidation. For the first stage mark circle 1 the soil sample is drilled out from the $K_0$ state and released until the isotropic consolidation state with the water content constant, namely the process is undrained condition. The deviator stress gets zero at this point and the geotome outside the soil sample is still kept. For the second step, the soil specimen is isotropically unloaded when the mean effective stress decreases to the atmosphere pressure. At this point, the soil specimen is taken out of the geotome. Then the mean effective stress of the specimen becomes larger until certain value at the third stage, which means that the soil specimen is reloaded and installed inside the triaxial compression equipment. The shape of the fourth stage is similar to the curve of the undrained triaxial compression test with decreasing mean effective stress and increasing deviator stress.

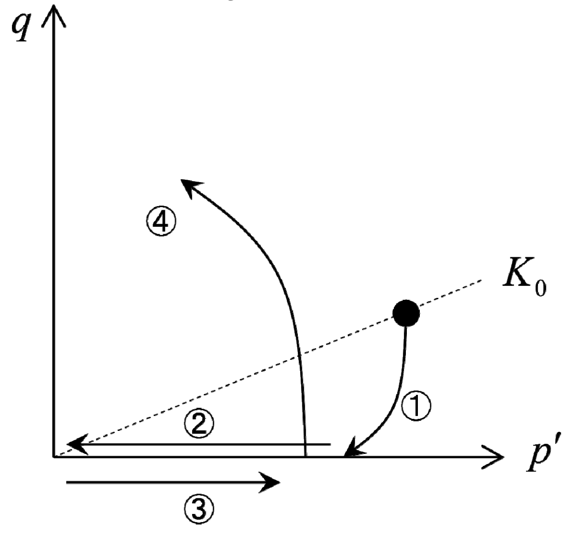


Figure 1. Reproducing process from soil drilling to trixial experiment

1) Determination of the initial vertical overburden soil pressure $\sigma_v'$. Here, the saturated state is assumed for the soil.

$$\sigma_v'(kPa = kN/m^2) = \gamma' \cdot z = \left(\frac{\rho_s - \rho_w}{1+\mathrm{e}}\right) \cdot g \cdot z \quad (1)$$

Here, $\gamma'$, $z$, $\rho_s$ and $\rho_w$ mean the buoyant bulk weight, the depth, the soil particle density and the unit weight of water. Initially the soil is the state of $K_0$ and then the deviator stress of the soil sample becomes zero that represents the soil is taken out from underground with the constant water content.

2) As to the preparation of the specimen, it can be regarded to be unloading process until the pressure equals the atmosphere pressure.

3) The soil sample is reloaded to a certain cell pressure.

4) The undrained triaxial compression experiment is carried out until a certain strain.

For the undisturbed soil, usually the above 4 stages can be used. For the disturbed soil specimen, the first stage can be neglected.

*2.2 Oedometer test*

The computation procedure of oedometer test of undisturbed specimen is shown in Fig. 2. As can be seen, it is composed of three stages and each stage will be explained in detail as follows:

1) Under the condition of constant volume the soil sample is taken out from underground and the deviator stress of the soil becomes zero, namely sampling. At the process, the vertical effective stress decreases in a slight magnitude.

2) After the soil specimen is sampling, one-dimensional compression test is carried out and the soil sample is situated to the compression ring as fast as possible.

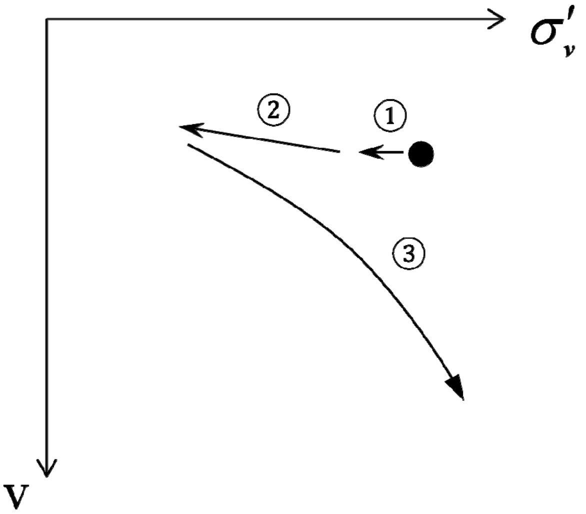


Figure 2. Reproducing process from soil drilling to one-dimensional experiment

## 3. Determination of initial values

Usually there will be some experiment results for clay. However, due to the difficult sampling there would be no experiment results of the dense sand. The in-situ ground survey datum including CPT value and bulk density becomes very useful to determine the sand parameters and the initial values.

The void ratio (Isai et al. 1986, Nakase 1972, Tanaka 1998) and the internal frictional angle (Iwasaki 1990) can be derived from the CPT value, and the maximum deviator stress can also be determined. The following stages is the same as the triaxial compression test as mentioned in the above section. The drained triaxial compression experiments are simulated under various cell pressures and different initial values are input to satisfy the maximum deviator stress and the specific volume. The detailed explanation can be seen in the following process.

*3.1. Void ratio*

There are three variable states involved in the determination of void ratio, which are the minimum void ratio, the maximum void ratio and the relative degree of density. According to the literature the equation is shown in Eq. 2.

$$\begin{cases} e_{\max} = 0.020F_c + 1.0 \\ e_{\min} = 0.008F_c + 0.6 \end{cases} \quad (2)$$

For the relative degree of density, the modified equation (Meyerhof 1957) can be used and the percentage of fine particle in soil is also used:

$$D_r = 21\sqrt{\frac{N}{\sigma'_v + 0.7} + \frac{\Delta N_f}{1.7}} \quad (3)$$

Here $N$, $\sigma'_v$, $\Delta N_f$ and $F_c$ are the CPT value, the vertical overburden soil pressure, the parameter related with $F_c$ and the percentage of fine particle.

Table 1. Determination of $\Delta N_f$ (after Tokimatsu and Yoshimi 1986).

| $F_c$ (%) | $\Delta N_f$ |
|---|---|
| 0~5 | 0 |
| 5~10 | 1.2($F_c$ -5) |
| 10~20 | 6+0.2($F_c$ -10) |
| 20~ | 8+0.1($F_c$ -20) |

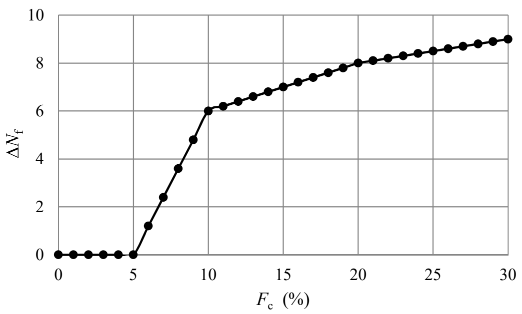


Figure 3. Relationship between $F_c$ and $\Delta N_f$

When the maximum void ratio, the minimum void ratio and the relative degree of density are derived, the next equations are used to derive the void ratio:

*3.2 Maximum deviator stress*

The deviator stress $q_f$ can be derived from the ground survey datum of CPT value (Tanaka et al. 2001) and it is related to the internal frictional angle $\Phi_d$:

$$q_f = \sigma'_a - \sigma'_r \quad (4)$$

where the major principal stress $\sigma'_a$ and minor principal stress $\sigma'_r$ are shown as follows:

$$\sigma'_a = \left(\frac{1+\sin\phi_d}{1-\sin\phi_d}\right)\sigma'_r \quad (5)$$

$$\sigma'_r = K_0\sigma'_v \quad (6)$$

Here $K_0$ is the coefficient of soil pressure and it is designated to be 0.6. And $\Phi_d$ is the internal frictional angle and determined by:

$$\phi_d = \begin{cases} \sqrt{20N_1 + 20} & (3.5 \le N_1 \le 20) \\ 40^\circ & (N_1 \ge 20) \end{cases} \quad (7)$$

The relationship between $\Phi_d$ and $N_1$ is shown in Fig. 4. Here $N_1 = N\sqrt{98/\sigma'_v}$. In addition, the slope of the critical state line (CSL) is calculated by Eq. 9. The relationship between the deviator stress $q_f$ and $N_1$ is shown in Fig. 5. As can be seen, there is a similar tendency compared with Fig. 4. The deviator stress firstly increases slowly while after $N_1$=20 a sudden increase can be seen in the deviator stress.

$$M = 6\sin\phi_d / (3 - \sin\phi_d) \quad (8)$$

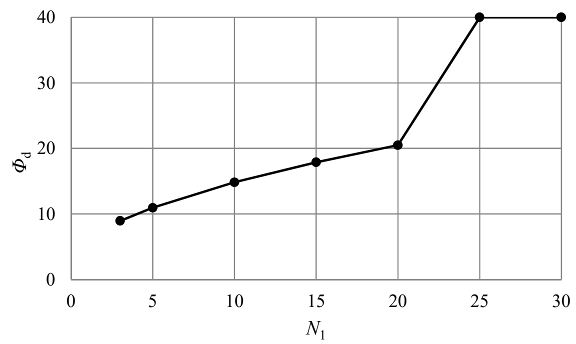


Figure 4. Relationship between $\Phi_d$ and $N_1$

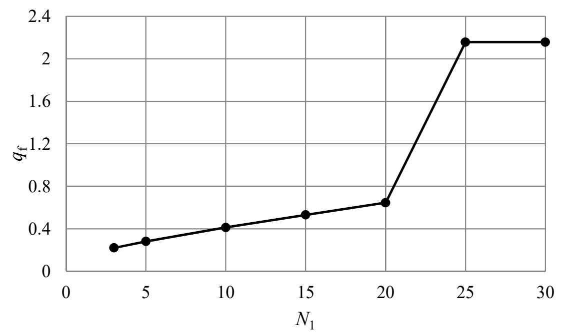


Figure 5. Relationship between $q_f$ and $N_1$

After the maximum deviator stress and the specific volume are derived from the ground survey datum, the try and error method should be used to predict the initial values and the constant parameters through numerical drained triaxial compression tests.

**4. Examples**

In Fig. 6, the triangle marks mean the displacement constraint condition in the designated direction. As can be seen, there are three groups of symmetrical planes including *3487* plane , *1243* plane and *1375* plane. The constant vertical stresses $\sigma_{xx}$ , $\sigma_{yy}$ and $\sigma_{zz}$ are acting correspondingly. The shear stress is reflected through adding a constant velocity on the *2486* plane. On the six side surface, the undrained boundary is applied. For the apparent behavior of the sample, at nodes *2*, *4*, *6* and *8* the equivalent nodal forces are measured to calculate the deviator stress; the apparent shear strain is derived through divide the ratio of horizontal displacement of node *2* by the initial height of the element.

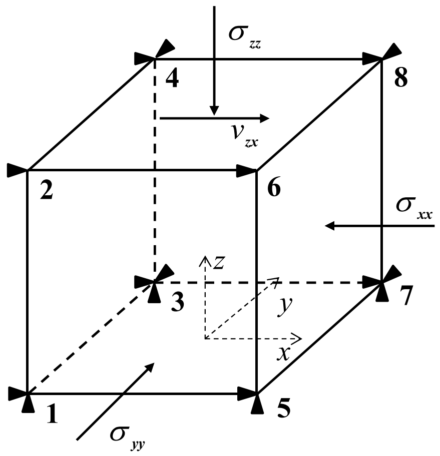


Figure 6. Mechanical and velocity boundary of three-dimensional element under uniform deformation field

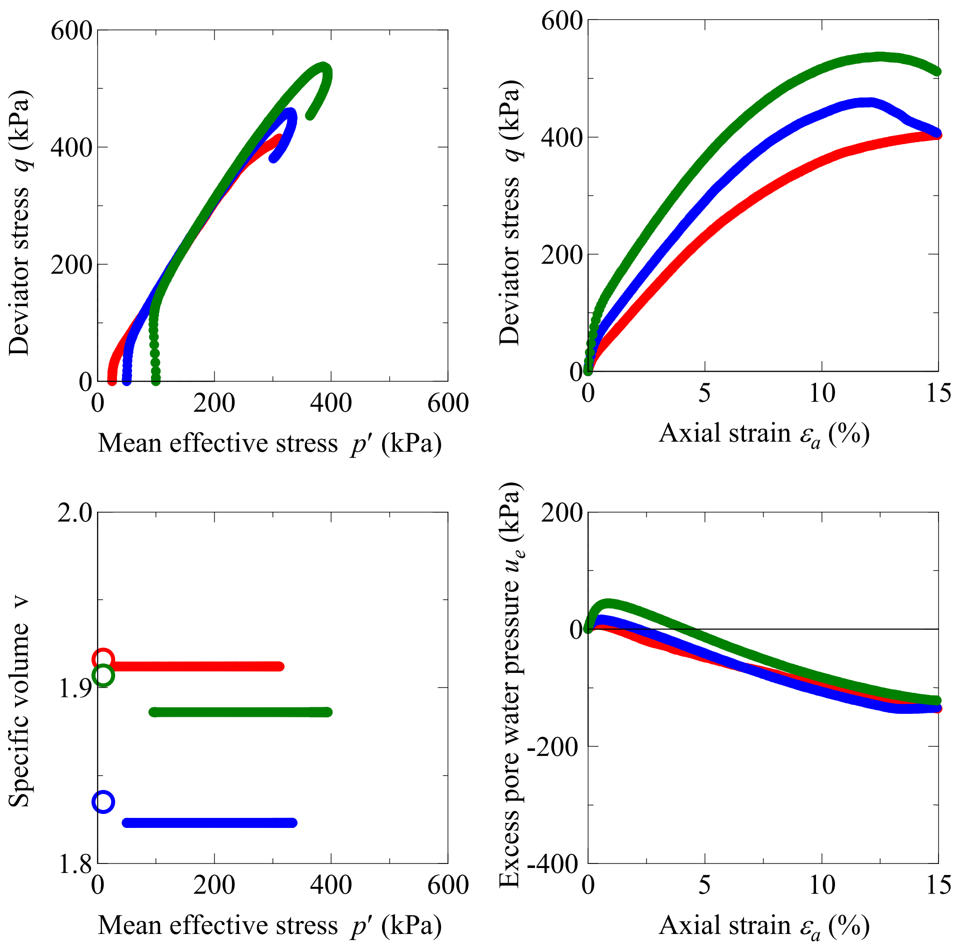


Figure 7. Experimental results of sand sample of undrained triaxial compression tests

The above steps can be calculated using the model shown in Fig. 7. And one of the typical results is shown in Fig. 6. As can be seen, the experiments of the undrained compression test are reproduced with the relationship of the deviator stress~the mean effective stress, the deviator stress~the axial strain, the specific volume~the mean effective stress, the excess pore pressure~the axial strain. Following the determination of undrained triaxial tests, the parameters can be obtained step by step.

## 5. Conclusions

In the numerical analysis, the precision of used parameters is the key whether the calculation results is reliable or not. Many researchers have reproduced the mechanical response of soils numerically under a uniform deformation field. However, the procedure of sampling and its influence on the mechanical properties are rarely considered, especially for the undisturbed soils. In the paper, the procedures of sampling for disturbed and undisturbed under triaxial tests and one-dimensional tests are decomposed step by step and discussed in details. The conclusions are as follows:

1) For triaxial tests, here are totally four steps to model the sampling procedure of undisturbed soil samples including the undrained unloading from $K_0$ state to isotropic state in the geotome, unloading isotropically from the geotome, reloading until designated pressure, shearing deformation, where the first step of undrained unloading is mostly easy to be forgotten. Using one-element FE analysis, the soil parameters can be obtained following the above steps.

2) There are three steps to model the sampling procedure of undisturbed sampling for oedometer tests, which is similar to the triaxial tests.

3) If there is lack of experimental results, it is feasible to predict the initial values and soil parameters using experiential equations. But at the boundary of the value range, it should be cautious when using these equations as there is usually sudden variation which may give a false result.

## References

[1] Isai, S. Kowzumi, K. and Tsuchida, H., Eur. Rept. of PHRI, **25**, 125-234 (1986)
[2] Nakase, A. Katsuno M. and Kobayashi, M. Rept. of PHRI,.**11**,140-147,(1972)
[3] Tanaka, H. Tanaka, M. and Tsuchida, T., *Journal of JSCE,* 589, 195-204, (1998)
[4] Iwasaki., *The Foundation Engineering and Equipment*,, 18, 40-48, (1990)
[5] Meyerhof, G.G., *Proc. of the 4th International Conference on Soil Mechanics and Foundation Engineering*, 3, 110, (1957).