Constitutive Model of Stress Path and Primary Anisotropy in Unloading Conditions

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Abstract
It is of importance to study and analyse the safety of foundation pit engineering when excavated by considering soil deformation under the unloading condition. There are many factors affecting the strength in unloading test, such as stress-paths, stress-history, and anisotropy, etc. In this paper, the classical elastic plastic model, HS model, is modified. The shear yield surface equation of considering stress path and anisotropy has been given under the unloading conditions. At the same time, the clay is taken as the research object, and the three triaxial tests that consider the stress path and anisotropy have been operated under the unloading conditions. The stress strain curves of the two directions are obtained and the constitutive model is validated. The test results and the model computed curves display the stress path and anisotropy effects of stress strain relationship can well be described by the new constitutive model; the degree of the constitutive correlation model and experimental curves is very high.

Key words: Anisotropy, Stress Path, Unloading Condition, Stress-strains, Constitutive Model.

1. INTRODUCTION

It is a hot academic research that study on the anisotropic characteristics of the soil. The anisotropy is divided into primary and induced anisotropy by Casagrande and Carillo (Xavier Cipriano and Jordi Carbonell, 2009). There is an important effect of anisotropy on the deformation and strength of the soil, through the anisotropic consolidation is studied by the experiments. The scholars have conducted the related research, such as laboratory test, constitutive model, numerical analysis and etc. Equamul Hoqueconsiders initial anisotropy is described by particulate material testing elastic deformation parameters (Kenzo Yonezawa and Fumio Yamada, 2010). The anisotropic shear modulus of two directions are obtained by a lot of natural Gao Lin clay stress-path tests, the results show that the Gao Lin clay has a highly anisotropic (I-Hai Lin and Broberg, 2012). Dudumu Ksyo study on the soil under the anisotropic consolidation condition and undrained stress-strain relationship, especially for the three axis Toyoura sand in undrained tests (Colak and Demirbas, 2008). When the P is equal in the test. Sushi K.Chaudhary observed the anisotropic characteristics and shear modulus changes that the deposition direction on the 0°, 22.5°, 45° plane shear, and the results are affected by loaded sample method and shear direction (Dang and Harley, 2012).

Based on the model in Cambridge, the S-CLAY1 model is put forward by Wheeler (Li and Calis, Gulben, 2012). The model considers the consolidation of anisotropic stress state. Yuan introduced an anisotropic elastoplastic model in FEM based on Biot consolidation theory method, and the deep excavation is analyzed and simulated that is considering anisotropy (Katsunori Sato and Masaki Samejima, 2012). In view of the influence of anisotropy of soils on the excavation deformation characters, Wei et al. studied from different angles. The above conclusions of the anisotropic study are generally conducted in loading conditions (Wu and Wang, 2007; Tang and Li, 2008; Yu and Zhao, 2011). However, it is also needed to be further studied to explore the influence of unloading state of soil anisotropic deformation properties.

Although, the stress-strain relationship characteristics have been affected by the stress-path and anisotropy under loading conditions, this conclusion has been reached. The study of the stress-strain relationship is relatively less which is considering stress-path and anisotropy. These conclusions are true under unloading condition. However, the degree of the stress path and anisotropic deformation properties is not clear. The influence is not specified due to no enough research achievements related to the unloading point of stress and unloading rate, no unified standards and complicated changes of unloading stress path during the excavation. Therefore, it is essential to study and conclude effects of these conditions on the stress-strain relationship of soil.

In order to avoid the negative influence of too many factors considered on test results. In this paper, it mainly focused on deformation characters of soil in consideration of stress path and anisotropy. Based on the previous research, an elastic-plastic modified model is provided in this paper. The purpose is to established a constitutive model which considers the stress path and anisotropic under unloading condition. The deformation and variation characteristics are studied by this model. Then, three triaxial tests that consider stress path and
anisotropic are described in this paper. It is compared to tests and calculation results; it shows the degree of two curves is high.

2. CONSTITUTIVE MODEL

2.1. Stress Path Parameter

Rule breaking by Mohr-Coulomb:

\[
\sin \varphi = \frac{\frac{\sigma_1 - \sigma_3}{2}}{\frac{\sigma_1 + \sigma_3}{2} + \cot \varphi} \quad \text{(1)}
\]

Hypothesis: \( \lambda = \frac{\sigma_1}{\sigma_3} \)

2.2. Original Anisotropic Parameters

Assuming that the soil is transversely isotropic, it only considers the two direction variation law of stress-strain. And the two directions of \( \sigma_1 \) and \( \sigma_3 \) are to uninstall, then:

\[
E_{11} = \frac{\sigma_1 - \sigma_3}{\varepsilon_1} \quad \text{(3)}
\]

\[
E_{13} = \frac{\sigma_3 - \sigma_1}{\varepsilon_3} \quad \text{(4)}
\]

Hypothesis:

\[
\tau = \frac{E_{11}}{E_{13}} \quad \text{(5)}
\]

Type: \( \tau \) - the anisotropic parameter

2.3. Modified Model

The basic hypothesis

In order to establish a modified model, it should be to require the assumptions of HS model and make appropriate adjustments.

1. The modified model should meet the Hooke's law and superposition principle. It is allowed that Poisson's ratio and elastic modulus can be different, when the main stress is loading in a single direction.

2. It is assumed that the deformation of soil is independent with stress path by an application of a level load increment. The total incremental load is composed of multilevel load increment; the changes of the total load should be satisfied superposition principle

The relationship between the tangent modulus and secant modulus

According to the generalized Hooke's law and three triaxial tests:

\[
\varepsilon_2 = \frac{1}{E_t} \left[ \sigma_2 + \nu \left( \sigma_0 + \sigma_0 \right) \right] \quad \text{(6)}
\]

\[
\varepsilon_3 = \frac{1}{E_t} \left[ \sigma_3 - \nu \left( \sigma_0 + \sigma_0 \right) \right]
\]

Type: \( E_t \) - Tangent modulus of elasticity; \( \nu \) - Poisson's ratio; \( \varepsilon_2, \varepsilon_3 \) - Lateral strain, and the vertical strain; \( \varepsilon_2, \varepsilon_3 \) - Maximum and minimum main strain; \( \sigma_1, \sigma_3 \) - Maximum and minimum main stress; the triaxial tests: \( \varepsilon_2 = \varepsilon_3 \); \( \sigma_2 = \sigma_3 \); \( \sigma_1 = \sigma_1 \); \( \sigma_2 = \sigma_2 \); \( \sigma_3 = \sigma_3 \);

Then:

\[
E_t = \frac{(\sigma_1 + 2\sigma_3)(\sigma_1 - \sigma_2)}{\sigma_3(\varepsilon_1 - 2\varepsilon_3) + \varepsilon_1\sigma_1} \quad \text{(7)}
\]

\[
\frac{\varepsilon_1}{\varepsilon_3} = \frac{2\nu}{\nu - 1} \quad \text{(8)}
\]

According to the definition of the secant modulus of the soil:
Simultaneous (7) to (9) type:

\[ E_i = \frac{v_i(1+2)}{1+v_i^2} E_s; \]  \hspace{1cm} (10)

HS model and modified

A basic idea for formulation of the Hardening-Soil model is the hyperbolic relationship between the vertical strains \( \varepsilon_i \), and the deviator stress \( q \), in primary triaxial loading. When it subjected to primary deviator loading, the soil shows a decreasing stiffness and simultaneously irreversible plastic strains develop. Standard drained triaxial tests tend to yield curves that can be described by:

\[ E_i = \frac{1}{2} E_s \left( \frac{q}{q_s} - 1 \right); \]  \hspace{1cm} (11)

\[ E_s = \frac{\sigma_s - \sigma_i}{\varepsilon_i}; \]  \hspace{1cm} (9)

The parameter \( E_i \) is the initial modulus. The parameter \( E_{so} \) is the confining stress dependent stiffness modulus for primary loading. The parameter \( q_f \) is derived from the MC failure criterion, which involves the strength parameters \( c \) and \( \phi \). The secant modulus \( E_{so} \) is determined from a triaxial stress-strain curve for a mobilization of 50% of the maximum shear strength \( q_f \).

The initial model of HS was modified by using experimental and theoretical methods (11). It is to change (11):

\[ \frac{q}{E_{so}} = 2 \frac{q}{q_s}; \]  \hspace{1cm} (16)

\[ \frac{E_i}{E_{so}} = 2 - 2R_f \frac{q}{q_f}; \]  \hspace{1cm} (17)

\[ \frac{E_s}{E_{so}} = a_i - b_i \frac{q}{q_f}; \]  \hspace{1cm} (18)

The parameter \( a_i = 2C_{\sigma_s} \) and the parameter \( b_i = 2R_f C_{\sigma_s} \):

\[ C_{\sigma_s} = \left( \frac{p_{so} + c \cot \phi}{\sigma_s + c \cot \phi} \right)^m; \]  \hspace{1cm} (19)

The conclusion is in literature (11):

\[ dq \cdot d\varepsilon_i \geq 0 : \frac{E_i}{E_{so}} = a_i - b_i \left( \frac{q}{q_f} \right)^{\varepsilon_i}; \]  \hspace{1cm} (20)

\[ q \cdot \varepsilon_i < 0 : \frac{E_i}{E_{so}} = C_{\sigma_s} \cdot a_s - C_{\sigma_s} \cdot b_s \left( \frac{q}{q_f} \right)^{\varepsilon_i}; \]  \hspace{1cm} (21)

The parameters \( g_1 \) and \( g_2 \) are related with the loading rate and initial static deviator stress.

Considering stress path and anisotropy modified model.

Assuming the soil is transversely isotropic and according to Hooke’s law, the stress-strain is considering the anisotropic:
$$\begin{align*}
\varepsilon_1 &= \frac{\sigma_1}{E_1} - \nu_{12} \frac{\sigma_2}{E_2} - \nu_{13} \frac{\sigma_3}{E_3}, \\
\varepsilon_2 &= -\nu_{21} \frac{\sigma_1}{E_1} + \frac{\sigma_2}{E_2} - \nu_{23} \frac{\sigma_3}{E_3}; \\
\varepsilon_3 &= -\nu_{31} \frac{\sigma_1}{E_1} - \nu_{32} \frac{\sigma_2}{E_2} + \frac{\sigma_3}{E_3}; \\
\varepsilon_i &= \frac{1}{E_i} \begin{bmatrix}
-\nu_{12} \\
-\nu_{21} \\
-\nu_{31}
\end{bmatrix}
\begin{bmatrix}
\sigma_2 \\
\sigma_3 \\
\sigma_3
\end{bmatrix}; \\
\varepsilon_2 &= \frac{1}{E_2} \begin{bmatrix}
1 \\
-\nu_{21} \\
-\nu_{32}
\end{bmatrix}
\begin{bmatrix}
\sigma_1 \\
\sigma_3 \\
\sigma_3
\end{bmatrix}; \\
\varepsilon_3 &= \frac{1}{E_3} \begin{bmatrix}
-\nu_{31} \\
1 \\
-\nu_{32}
\end{bmatrix}
\begin{bmatrix}
\sigma_1 \\
\sigma_2 \\
\sigma_3
\end{bmatrix}.
\end{align*}$$

(22)
(23)

It is only to consider variation law of stress-strain under the unloading conditions in two directions.

$$\begin{align*}
v_{12} &= v_{13} = -\frac{\partial \varepsilon_1}{\partial \varepsilon_1}, \\
v_{21} &= v_{11} = -\frac{\partial \varepsilon_1}{\partial \varepsilon_2}; \\
v_{23} &= v_{32} = -1.
\end{align*}$$

(24)

Simultaneous (22) to (24) type:

$$\begin{align*}
\varepsilon_1 &= \frac{1}{E_1} \begin{bmatrix}
-2v_{11} \\
-v_{11} \\
-\nu_{31}
\end{bmatrix}
\begin{bmatrix}
\sigma_1 \\
\sigma_3 \\
\sigma_3
\end{bmatrix}; \\
\varepsilon_3 &= \frac{1}{E_3} \begin{bmatrix}
v_{11}(1+\lambda v_{11}) \\
-v_{11}^2(1+\lambda v_{11}) \\
v_{11}^2(1+\lambda v_{11})
\end{bmatrix}
\begin{bmatrix}
\sigma_1 \\
\sigma_3 \\
\sigma_3
\end{bmatrix}.
\end{align*}$$

(25)

Then:

$$E_{13} = \frac{v_{13}}{v_{13}} \left(1 + \lambda v_{13} \right) E_{11};$$

(26)

The conclusion is when the $\sigma_1$ is the main stress, the stress-strain is described:

$$\begin{align*}
\varepsilon_1 &= \frac{1}{E_1} \begin{bmatrix}
-2v_{11}^2 \\
-v_{11}^2 \\
-\nu_{31}^2
\end{bmatrix}
\begin{bmatrix}
\sigma_1 \\
\sigma_3 \\
\sigma_3
\end{bmatrix}; \\
\varepsilon_3 &= \frac{1}{E_3} \begin{bmatrix}
v_{11}^2(1+\lambda v_{11}) \\
-v_{11}^2(1+\lambda v_{11}) \\
v_{11}^2(1+\lambda v_{11})
\end{bmatrix}
\begin{bmatrix}
\sigma_1 \\
\sigma_3 \\
\sigma_3
\end{bmatrix}.
\end{align*}$$

(27)

When the $\sigma_3$ is the main stress, the stress-strain is described:

$$\begin{align*}
\varepsilon_1 &= \frac{1}{E_1} \begin{bmatrix}
v_{11}(1+\lambda v_{11}) \\
-v_{11}(1+\lambda v_{11}) \\
-\nu_{31}(1+\lambda v_{11})
\end{bmatrix}
\begin{bmatrix}
\sigma_1 \\
\sigma_3 \\
\sigma_3
\end{bmatrix}; \\
\varepsilon_3 &= \frac{1}{E_3} \begin{bmatrix}
v_{11}^2(1+\lambda v_{11}) \\
-v_{11}^2(1+\lambda v_{11}) \\
\nu_{31}^2(1+\lambda v_{11})
\end{bmatrix}
\begin{bmatrix}
\sigma_1 \\
\sigma_3 \\
\sigma_3
\end{bmatrix}.
\end{align*}$$

(28)

The modified model of considering stress-path and anisotropic is described:

When the $\sigma_1$ is the main stress, the stress-strain is described:

$$\begin{align*}
\varepsilon_1 &= \frac{1}{E_1} \begin{bmatrix}
1 \\
-v_{11}^2(1+\lambda v_{11}) \\
-\nu_{31}^2(1+\lambda v_{11})
\end{bmatrix}
\begin{bmatrix}
\sigma_1 \\
\sigma_3 \\
\sigma_3
\end{bmatrix}; \\
\varepsilon_3 &= \frac{1}{E_3} \begin{bmatrix}
1 -2v_{11}^2 \\
-v_{11}^2(1+\lambda v_{11}) \\
v_{11}^2(1+\lambda v_{11})
\end{bmatrix}
\begin{bmatrix}
\sigma_1 \\
\sigma_3 \\
\sigma_3
\end{bmatrix}.
\end{align*}$$

(29)
when the $\sigma_j$ is the main stress, the stress-strain is described:

$$
\begin{align*}
\begin{bmatrix}
\varepsilon_i \\
\varepsilon_j
\end{bmatrix} = \frac{1}{E_{ij}} \begin{bmatrix}
\frac{v_{ij}(1+\lambda v_{ij})}{v_{ij}(1+\lambda v_{ij})} - 2v_{ij} \\
-2v_{ij}
\end{bmatrix} \begin{bmatrix}
\sigma_i \\
\sigma_j
\end{bmatrix} = \frac{1}{E_{w}^o} \begin{bmatrix}
\sigma_{o} + c \cot \phi \\
p_{o}^o + c \cot \phi
\end{bmatrix} \begin{bmatrix}
\frac{v_{ij}(1+\lambda v_{ij})}{v_{ij}(1+\lambda v_{ij})} - 2v_{ij} \\
-2v_{ij}
\end{bmatrix} \begin{bmatrix}
\sigma_i \\
\sigma_j
\end{bmatrix}; \quad (30)
\end{align*}
$$

The shear yield surface equation of HS model is given by Schanz(12):

$$
f' = \bar{f} - \gamma^p = \frac{1}{E_{o}^o} \frac{q}{1-q/q_o} - 2q E_{w}^o - \gamma^p;
$$

$$E_{w} = E_{w}^o \left( \frac{\sigma_{o} + c \cot \phi}{p^o + c \cot \phi} \right)^m;
$$

The parameter $E_{w}^o$ is unloading and reloading stiffness modulus; The parameter $E_{w}^o$ is the reference Young’s modulus for unloading and reloading, corresponding to the reference pressure $p^o$; The parameter $\gamma^p$ is the plastic shear strain.

The shear yield surface equation of modified HS model is given:

When the $\sigma_j$ is the main stress, the stress-strain is described:

$$
f' = \bar{f} - \gamma^p = \frac{1}{E_{o}^o} \frac{1}{v_{ij}(\lambda + 2)(R_f - 2)A} - 2q E_{w}^o - \gamma^p;
$$

$$f' = \bar{f} - \gamma^p = -\frac{1}{E_{o}^o} \frac{1}{v_{ij}(\lambda + 2)(R_f - 2)A} - 2q E_{w}^o - \gamma^p;
$$

when the $\sigma_j$ is the main stress, the stress-strain is described

$$
\begin{align*}
f'_j &= \bar{f} - \gamma^p \\
&= \frac{1}{E_{o}^o} \frac{1}{(R_f - 2)^2} \left( \frac{\sigma_{o} + c \cot \phi}{p^o + c \cot \phi} \right)^m \frac{v_{ij}(1+\lambda v_{ij})}{v_{ij}(1+\lambda v_{ij})} q \left( \frac{\sigma_{j}^o + c \cot \phi}{p^o + c \cot \phi} \right)^m \frac{2q}{E_{w}^o} - \gamma^p; \\
&= -\frac{1}{E_{o}^o} \frac{1}{(R_f - 2)^2} \left( \frac{\sigma_{o} + c \cot \phi}{p^o + c \cot \phi} \right)^m \frac{v_{ij}(1+\lambda v_{ij})}{v_{ij}(1+\lambda v_{ij})} q \left( \frac{\sigma_{j}^o + c \cot \phi}{p^o + c \cot \phi} \right)^m \frac{2q}{E_{w}^o} - \gamma^p.
\end{align*}
$$

3. MODEL VALIDATION

3.1. Testing Program

Samples were taken underneath the earth’s surface, approximately 10 meters, 20 meters and 30 meters deep. They were $Q_3$ recently deposited soil with the classical homogeneous silty clay of the northern area and little mezzanine impurities. In order to fully compare different test results, undrained conventional triaxial compression tests for undisturbed soil sample of each layer were conducted respectively. The main physical
parameters of undisturbed soil samples are concluded in Table 1, in which each parameter was the average value of 171 samples.

Unloading scheme

According to different depth, 3 groups of undisturbed soil sample were selected to the consolidated drained test. The consolidation pressure is 100kPa, 200kPa and 300 kPa. There are some conditions that indicate the termination of tests: the unloading is unstalling to 50kPa in some direction or the strain exceeds 15%. All experiments were conducted in the same axial radial stress and the strain of independent controlled type triaxial shear apparatus (13) with stress control as the control mode. Based on the range of control stress rate which can be obtained from the control shear strain loading rate of clay and silty clay in the consolidation, the shear rate strain is 0.003%-0.012% per minute. If the elastic modulus of clay is 0.25MPa, the axial loading rate is 0.00075kPa-0.03kPa per minute. So the unloading rate of the test is 0.25kPa/min. Table 2 demonstrates the detailed test program.

Test data processing method

The εr strain cannot be directly obtained because of the test instruments without this function. The formula εr = ε1 + 2ε3 is applied to the data processing. AL1100 and AA1100 in Table 2 are as examples to explain the data processing method. AL1100 and AA1100 come from the same undisturbed soil. After consolidation AL1100 is tested that axial stress is invariant and lateral stress is decreased by 0.2kPa/min. After the experiment is finished which reached the end condition, the εr strain is obtained. And then AA1100 is tested that axial stress is decreased by 0.2kPa/min and lateral stress is invariant. When it reached the end condition, the εr strain is also obtained. According to the formula εr = ε1 + 2ε3, the ε3 strain is calculated. In the same amount of unloading, the strain of two opposite directions has been given.

Table 1. The main physical parameters of undisturbed soil

| Property          | Depth/m | Density ρ (g/cm³) | Moisture content ω (%) | Specific weight Gs | Void ratio e | Plastic limit wP (%) | Liquid limit wL (%) | Cohesion c | Friction angle φ° |
|-------------------|---------|-------------------|------------------------|-------------------|-------------|----------------------|---------------------|-------------|-----------------|
| Undisturbed soil  | 10      | 1.88              | 30.8                   | 2.72              | 0.891       | 13.3                 | 37.7                | 8.6         | 5.1             |
|                   | 20      | 1.95              | 29.3                   | 2.71              | 0.769       | 13.6                 | 36.2                | 19.7        | 9.5             |
|                   | 30      | 1.98              | 27.4                   | 2.74              | 0.779       | 13.5                 | 34.6                | 20.1        | 6.9             |

Table 2. Test program

| Trial number      | Consolidation pressure /kPa | Rates of loading and unloading (kPa/min) | Stress paths of loading and unloading |
|-------------------|-----------------------------|------------------------------------------|-------------------------------------|
| AL100/AL200/AL300 | 100                         | 0.2                                      | σ1 constant, σ3 decreases to 50kPa |
| AL1200/AL2200/AL3200 | 200                          |                                           |                                      |
| AL1300/AL2300/AL3300 | 300                         |                                           |                                      |
| AA1100/AA2100/AA3100 | 100                         | 0.2                                      | σ1 constant, σ3 decreases to 50kPa |
| AA1200/AA2200/AA3200 | 200                         |                                           |                                      |
| AA1300/AA2300/AA3300 | 300                         |                                           |                                      |

Note: AL: Anisotropic Lateral unloading; AA: Anisotropic Axial unloading.

3.2. Testing Results and Analysis

The consolidation pressure is respectively 100kPa, 200kPa and 300kPa, the curves of the tests and calculated model are depicted in Figure 1, 2 and 3. The Figure 1 and Figure 2 show that the soil samples have not been damaged, when the experiments were finished. And the Figure 3 shows that the large deformation has been happened in this soil sample, when it has reached the end of test conditions. From the Figures 1-3, the fitting degree of computational and experimental is high.
Figure 1. The 100 kPa confining pressure, the stress-strain of tests and calculated model

Figure 2. The 200 kPa confining pressure, the stress-strain of tests and calculated model

Figure 3. The 300 kPa confining pressure, the stress-strain of tests and calculated model

The Strain of test curve values are substituted into the constitutive model of the formula, the basic calculation parameters as shown in table 3, the parameter $\lambda$ has been determined. Because it is unloaded and stress controlled, when the unloading amount reaches 50kPa/15, test parameters $a_i$ are given in Table 3. And the basic calculation parameter values of the constitutive model are described in table 4. It has showed that the relationship of the stress-strain is strain hardening in figure 1 and figure 3. So the parameters $a_i$ and $b_i$ have been only discussed.
### Table 3. The tests parameters

| Consolidation pressure /kPa | The parameter $\lambda$ |
|-----------------------------|--------------------------|
| 100                         | 2.37 2.55 - - - -        |
| 200                         | 2.64 2.72 2.87 3.35 - - |
| 300                         | 2.42 2.44 2.5 2.57 2.72 | 3.18 |

### Table 4. The basic calculation parameters

| Consolidation pressure /kPa | $\tau$  | $V_{13}$ | $V_{33}$ | $E_{50}^{ref}$ (kPa) |
|-----------------------------|---------|----------|----------|----------------------|
| 100                         | 0.25    | 0.25     | 0.31     | 73.4                 |
| 200                         | 0.33    | 0.29     | 0.35     | 98.03                |
| 300                         | 0.38    | 0.22     | 0.38     | 105.26               |

3.3. Relationship of $E_s/E_{50}^{ref}$ and $q/q_f$

The relationship of $E_s/E_{50}^{ref}$ and $q/q_f$ has been showed in Figure 4, Figure 5 and Figure 6. The test results have been described by solid and hollow points. According to the formula (11), the model calculation results have been described by solid and dashed lines.

![Figure 4. The 100 kPa confining pressure, the curves of $E_s/E_{50}^{ref}$ and $q/q_f$.](image)

![Figure 5. The 200 kPa confining pressure, the curves of $E_s/E_{50}^{ref}$ and $q/q_f$.](image)
Figure 6. The 300 kPa confining pressure, the curves of $E_s/E_{so}$ and $q/q_f$.

Table 5. The parameters of the tests and model

| pressure /kPa | Direction | Parameters | The tests | The model |
|---------------|-----------|------------|-----------|-----------|
|               |           | $a_1$      | $b_1$     | $a_2$     | $b_2$     |
| 100           | $\varepsilon_1$ | 0.967     | 0.793     | 0.936     | 0.974     |
|               | $\varepsilon_3$ | 0.932     | 0.758     | 0.897     | 0.854     |
| 200           | $\varepsilon_1$ | 0.921     | 0.863     | 0.989     | 0.963     |
|               | $\varepsilon_3$ | 0.914     | 0.896     | 0.914     | 0.937     |
| 300           | $\varepsilon_1$ | 0.875     | 0.783     | 0.894     | 0.923     |
|               | $\varepsilon_3$ | 0.835     | 0.887     | 0.925     | 0.927     |

The relationship of $E_s/E_{so}$ and $q/q_f$ has been showed a nonlinear relationship. The parameters of the tests and model have been listed in Table 5. It is shows that the model and test date have described a high correlation, which is considering the stress path and anisotropy. In unloading conditions, this model of considering the stress path and anisotropic can describe the deformation of the soil in excavation.

4. CONCLUSIONS

(1) The classical elastic plastic HS model is modified. In unloading conditions, the shear yield equation of considering the stress path and anisotropic has been given in this paper.

(2) The soil deformation of considering stress path and anisotropy effects has been studied in unloading tests. And the two directions of the stress strain relation is obtained. According to the experimental curve, the relevant parameters are calculated.

(3) The test curve is fitted by the new model. It is founded that the degree of the model and test curves is high. The calculation parameters are similar as the test parameters.

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REFERENCES

Casarande A, Carillo N. (1944) “Shear failure of circular excavations in soft clay”, Journal of the Boston Society of Civil Engineers, 31(4), pp.74-87.

Dudumu Kysyo, Kmrj Idhihsts Snf Ikuo Towbata (2001) “Undrained shear characteristics of saturated sand under anisotropic consolidations”, Soils and Foundations, 41(1), pp.1-11.

Eqramul Hoque , Fumio Tatsuoka (1998) “Anisotropy in elastic deformation of granular materials”, Soils and Foundations, 38(1), pp.163-179.

Lings, M.L, Pennington D.S, Nash (2012) “Anisotropic stiffness parameters and their measurement in a stiff natural clay”, Geotechnique, 50(2), pp.109-125.
Liao Zhanjun (2007) “Study on the influence of stress anisotropy and the excavation of deep foundation pit in soft soil”, Tongji University.
Liu Li, Zhang H R. (2012) “Studying on unloading testing technology”, Proc. of International Conference on New Technologies, pp.515-518.
Qiu Yumin (2011) Study on post-construction settlement of road under traffic load. Zhejiang University (Thesis).
Schanz, T., Vermeer, P.A. (1998) “Special issue on Pre-failure deformation behaviour of geomaterials”, Geotechnique, 48, pp.383-387.
Sushil K. Chaudhary, Jiro Kuwano (2002) “Effects of anisotropic elasticity on the yielding on cyclic deformation characteristics of sands”, Soil and foundations, 42(1), pp.147-157.
Wheeler S J, Naatanen A, Karstunen M et.al. (2003) “An anisotropic elastoplastic model for soft clays”, Canadian Geotechnical Journal, 40, pp.403-418.
Wei Xing (2005) “Research on constitutive model of anisotropic and structure of the natural clay”, Tongji University (Thesis).
Yuan Juyun, Ye Chaohan, Zhao Xihong (2006) “Finite element analysis on deep pit with isotropic elasto”, Chinese Journal of underground space and Engineering, 2(3), pp.407-410.
Zhang Kenyong (2004) “Study and Application on soil’s constitutive model with the consideration of stress-induced anisotropy”, Hohai University(Thesis).