Modeling Confined Compression on Structured Soils

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Abstract: In confined compression on structured soil, due to the exertion and decay of soil structure, the ratio of horizontal to vertical stresses is low in the earlier compression stage and tends to be the same as normally consolidated soil finally. In order to describe this behavior, the structured unified hardening model (structured UH model) was extended in the following 2 aspects: (1) properly selecting Poisson's ratio to reflect the stress ratio in earlier compression stage; (2) adding a shape parameter in dilatancy equation to control the stress ratio in final confined compression stage. According to the comparisons between model simulations and test data, the extended structured UH model is qualified in reasonably simulating the stress states of structured soil in confined compression.

1. Introduction

For a soil sample in confined compression ($K_0$ compression), the vertical deformation develops but the horizontal deformation is limited to 0. The ratio of horizontal stress $\sigma_h$ to vertical stress $\sigma_v$ is $K_0$. Understanding the evolution of $K_0$ in loading is of great significance to the design and construction of actual projects. In 1944, Jaky expressed $K_0$ as a function of internal friction angle $\phi$[1][2]. Since then, based on the confined compression test data, different scholars expressed $K_0$ as different functions of $\phi$[3][4]. This means that as long as parameter $\phi$ is determined, the $K_0$ value is constant. For the normally consolidated soils, only at low consolidation stress level, $K_0$ value can be approximately considered as a constant[5]. When the consolidation stress is high enough, even for normally consolidated soil, $K_0$ value will increase with consolidated stress[6]. However, the normally consolidated soil does not exist in nature. For the widely distributed natural soil, $K_0$ value is surely not constant due to many factors such as soil structure[7], water content[8], internal cracks[9] and admixtures[10]. By experimental study, Chen et al.[8] concluded that $K_0$ value increases with increasing water content, and $K_0$ value may be greater than 1 in unloading stage. Huang et al.[7] also pointed out that the $K_0$ value of undisturbed soil will be low in the early compression stage due to soil structure, and gradually tend to the $K_0$ value of the normally consolidated soil in the later compression stage. Literature [11] points out that, at the initial stage of $K_0$ compression on the structured soil, soil structure restricts the radial deformation tendency of the sample, which makes $K_0$ value low. At the later stage of loading, soil structure is almost exhausted, and the $K_0$ value restores to the same as that of the normally consolidated soil. Obviously, the constant $K_0$ equation obviously cannot describe the ever-changing $K_0$ value of structured soil in confined compression. From another point of view, confined compression is one kind of strain boundary controlled loading, thus the above $K_0$ value feedback of structured soil should be reflected by structured constitutive model.

On the basis of Camclay model[12][13] proposed for normally consolidated soil, and by formulating a unified hardening parameter$^{14}H$, a unified hardening model$^{15}$ (UH model) is established to simulate over-consolidated soil. Thereafter, by proposing a moving normally consolidated line$^{16}$, an
one-dimensional constitutive model simulating isotropic compression of structured soil\cite{17} is presented, which is then extended to a structured UH model\cite{18}\cite{19} and bonding structured UH model\cite{20}. And then the bonding structured UH model is applied in three-dimensional stress space\cite{21}. Though the established structured soil UH model\cite{17}-\cite{21} is qualified in simulating the primary behaviors of structured soil, they are still not good enough in reproducing stress state of structured soil in confined compression.

Therefore, this paper focus on extending the structured UH model to reflect stress state evolution of structured soil in confined compression. In the following discussion: Axial (vertical) stress, radial (horizontal) stress, axial strain and radial strain are respectively expressed as \( \sigma_y, \sigma_r, \varepsilon_a \) and \( \varepsilon_r \). The corresponding average principal stress \( p \), generalized deviatoric stress \( q \), volumetric strain \( \varepsilon_v \), generalized deviatoric strain \( \varepsilon_d \) and stress ratio \( \eta \) are expressed as \( p = (\sigma_a + 2\sigma_r)/3, \ q = (\sigma_a - \sigma_r), \ \varepsilon_v = \varepsilon_a + 2\varepsilon_r, \ \varepsilon_d = 2(\varepsilon_a - \varepsilon_r)/3, \) and \( \eta = q/p \).

2. Stress state description of confined compression

As illustrated in Fig. 1, stress state of soil element in semi-infinite space is confined compression\((K_0)\) state. The famous \( K_0 \) value expression for normally consolidated soil is given by Jaky\cite{1} whose simplified form\cite{2} is written as:

\[
K_0 = 1 - \sin \varphi
\]

Thereafter, different scholars proposed similar \( K_0 \) expressions from different perspectives. For example, the Simpson\cite{3} and Luo et al.\cite{4} respectively describe the \( K_0 \) value as equation (2) and (3):

\[
K_0 = \frac{\sqrt{2} - \sin \varphi}{\sqrt{2} + \sin \varphi}
\]

\[
K_0 = \frac{9 - \sin \varphi}{9 + 5\sin \varphi}
\]

Obviously, the above 2 equations describe the \( K_0 \) value to be constant. However test data of natural soil suggests otherwise. According to the test data of confined compression on structured soil and corresponding normally consolidated soil\cite{22} in Fig. 2, stress ratio \( (\sigma_0/\sigma_y) \) of normally consolidated soil decreases monotonously and finally tends to a constant value, while \( (\sigma_0/\sigma_y) \) of structured soil firstly decreases, then increases and finally tends to the same stress ratio as the corresponding normally consolidated soil.

In the elastoplastic constitutive model, both the volumetric strain increment \( d\varepsilon_v \) and the deviatoric strain increment \( d\varepsilon_d \) are composed of their respective elastic and plastic parts \( d\varepsilon_v^e, \ v_d^e, \ d\varepsilon_d^e \). From the definition of confined compression, \( d\varepsilon_a > 0, d\varepsilon_r = 0 \), it can be concluded that:

\[
\frac{d\varepsilon_v^e + d\varepsilon_d^p}{d\varepsilon_v + d\varepsilon_d} = \frac{d\varepsilon_v}{2(d\varepsilon_a - d\varepsilon_r)/3} = 1.5
\]

(4)

The elastic strain increments \( d\varepsilon_v^e \) and \( d\varepsilon_d^e \) are described by the generalized Hooke's law:

\[
\begin{align*}
\frac{d\varepsilon_v^e}{\frac{k}{1+\varepsilon_0}} &= \frac{d\sigma_v}{p} \\
\frac{d\varepsilon_d^e}{9(1+\varepsilon_0)(1+2\nu)} &= \frac{d\sigma_d}{p}
\end{align*}
\]

(5)

\[
\sigma_y \quad \sigma_h
\]

Figure 1. Stress state of soil in confined compression
In equation(5): $e_0$ is the initial value of sample void ratio $e$; $\nu$ is the Poisson's ratio; $\kappa$ is the absolute slope of isotropic swelling line of the reconstituted soil in $e$-$\ln p$ coordinates. The plastic strain increment $d\varepsilon_p^v$ and $d\varepsilon_d^p$ are described by the constitutive model through yield function, hardening law and plastic potential function (dilatancy equation). In the following, the description of normally consolidated soil and structured soil in confined compression by constitutive model will be introduced separately.

\begin{align*}
\text{(a) Relationship of } \sigma_v \sim (\sigma_h/\sigma_v) \\
\text{(b) Relationship of } \sigma_v \sim \eta
\end{align*}

Figure 2. Stress states of normally consolidated and structured soils in confined compression

2.1 Description of confined compression on normally consolidated soil

The famous Camclay model is appropriate to reproduce the behaviors of normally consolidated soil. The yield function $f$ and hardening law are:

$$f = c_p \ln \left[p_0 \left(1 + \frac{\eta^2}{M^2}\right)^\lambda\right] - e_0^p = 0$$  \hspace{1cm} (6)

Applying the associated flow rule, plastic potential function $g = f$, thus the dilatancy equation is:

$$\frac{d\varepsilon_d^p}{d\varepsilon_d^v} = \frac{\partial f/\partial q}{\partial f/\partial p} = \frac{M^2-\eta^2}{2\eta}$$  \hspace{1cm} (7)

In equation (6) and (7): $c_p = (\lambda - \kappa)/(1 + e_0)$; $\lambda$ is the absolute slope of isotropic compression line of the normally consolidated soil in $e$-$\ln p$ coordinates; $p_0$ is the initial value of $p$; $M$ is the critical state stress ratio. Therefore, the plastic strain increments by Camclay model are:

$$\begin{cases}
\varepsilon_d^p = c_p \frac{(M^2-\eta^2)dq + 2\eta dq}{p(M^2+\eta^2)} \\
\varepsilon_d^d = c_p \frac{(M^2-\eta^2)dq + 2\eta dq}{p(M^2+\eta^2)} \frac{2\eta}{(M^2-\eta^2)}
\end{cases}$$  \hspace{1cm} (8)

Combined equation (5) and (8), strain increments ratio $(d\varepsilon_v/d\varepsilon_d)$ by Camclay model is:

$$\frac{d\varepsilon_v}{d\varepsilon_d} = \frac{\kappa + (\lambda - \kappa)\frac{M^2 - \eta^2 + 2\eta dq}{M^2 + \eta^2}}{\kappa + (\lambda - \kappa)\frac{M^2 - \eta^2 + 2\eta dq}{M^2 + \eta^2}}$$  \hspace{1cm} (9)

If soil sample is compressed in constant stress ratio $\eta$ with initial stress state $(p, q) = (0 \text{kPa}, 0 \text{kPa})$, then $dq/dp = q/p = \eta$. Substituting $dq/dp = \eta$ into equation (9), strain increment ratio $(d\varepsilon_v/d\varepsilon_d)$ keeps constant. The strains also develop from $(\varepsilon_v, \varepsilon_d) = (0, 0)$, thus $(d\varepsilon_v/d\varepsilon_d) = (\varepsilon_v/\varepsilon_d)$. Therefore, according to equation (9) by Camclay model, constant strain ratio compression is also constant stress ratio compression. Constant strain ratio of confined compression is $d\varepsilon_v/d\varepsilon_d = 1.5$, the corresponding constant stress ratio $\eta_{0\text{CC}}$ can be derived by equation (9) as:

$$1.5 = \frac{\kappa + (\lambda - \kappa)\frac{M^2 - \eta^2 + 2\eta dq}{M^2 + \eta^2}}{\kappa + (\lambda - \kappa)\frac{M^2 - \eta^2 + 2\eta dq}{M^2 + \eta^2}}$$  \hspace{1cm} (10)

According to equation (10), stress ratio $\eta_{0\text{CC}}$ in confined compression by Camclay model is...
affected by $\kappa/(\lambda - \kappa)$, $\nu$ and $M$ (i.e. $\varphi$). Recalling equation (1), (2) and (3), the reproduced confined compression stress ratios are only affected by parameter $\varphi$. Setting $\kappa/(\lambda - \kappa) = 1/4$, $\nu = 1/3$, confined compression stress ratios $\eta$ by respectively Camclay model, Jaky equation, Simpson equation and Luo equation are illustrated in Fig. 3 in $\varphi - \eta$ coordinates.

From Fig. 3, descriptions by Jaky equation, Simpson equation and Luo equation are analogical, however description by Camclay model is dramatically different from the other three. This inspires the authors to modify the Camclay model to be more reasonable in reproducing confined compression stress state of normally consolidated soil.

As shown in equation (10) by Camclay model, both the numerator and denominator are composed of elastic $\kappa$ item and plastic $(\lambda - \kappa)$ item. If only elastic $\kappa$ items in equation (10) are considered, the stress ratio $\eta$ described degrades to:

$$\eta = \frac{3(1 - 2\nu)}{(1 + \nu)} \quad (11)$$

If only plastic $(\lambda - \kappa)$ items in equation (10) are considered, the stress ratio $\eta$ described degrades to:

$$\eta = \frac{\sqrt{9 + 4M^2} - 3}{2} \quad (12)$$

Actually, elastic and plastic strains occur simultaneously. Thus the confined compression stress ratio described by Camclay model falls in between equation (11) and (12). Setting $\kappa/(\lambda - \kappa) = 1/4$, $\nu = 1/3$, stress ratios by equation (10), (11) and (12) in $\varphi - \eta$ coordinates are shown in Fig. 4:

![Figure 3. Different descriptions of confined compression stress ratio](image1)

![Figure 4. Confined compression stress ratio described by Camclay model](image2)

Since the weight of plastic $(\lambda - \kappa)$ items is obviously larger than that of elastic $\kappa$ items, $\eta_{\text{occ}}$ is
close to the pure plastic description equation (12). Obviously, confined compression stress ratio \( \eta_{CC} \) mainly depends on the plastic \((\lambda - \kappa)\) items in equation (10). Thus, in order to extend the Camclay model to reproduce confined compression stress ratio of different normally consolidated soils, equation (12) is suitable to be modified. Inspired by literature [23], a more generalized Camclay model dilatancy equation[18] can be written as follows:

\[
\frac{\partial e_p^0}{\partial \sigma_0^v} = \frac{M^{n-\eta^n}}{\eta n^{n-1}}
\]  

(13)

In equation (13), parameter \( n \) is used to adjust the plastic flow direction. When \( n = 2 \), equation (13) degrades to Camclay model dilatancy equation (7). Applying equation (13) instead of equation (7), the confined compression stress ratio \( \eta_0 \) of normally consolidated soil is described as:

\[
1.5 = \frac{\kappa + (\lambda - \kappa)}{\nu} \frac{1 + \eta_0}{\nu} + (\lambda - \kappa) \frac{\eta_0^{n-1}}{n^{n-1} M^{n-1}}
\]  

(14)

If only plastic \((\lambda - \kappa)\) items in equation (14) are considered, the stress ratio \( \eta \) described degrades to:

\[
1.5 = \frac{(M^n - \eta^n)}{(n^p \eta^{n-1})}
\]  

(15)

Applying equation (15) to replace equation (12), confined compression stress ratio \( \eta_0 \) of normally consolidated soil falls in between the equation (11) and (15). The weight ratio of elastic and plastic items is expressed by the ratio \( \kappa/(\lambda - \kappa) \).

Setting \( \lambda = 0.2 \), \( \kappa = 0.04 \), \( M = 1.2 \), \( N = 2.0 \), \( \nu = 1/3 \), initial stresses \( \sigma_0 = \sigma_v = 1 \text{kPa} \), and initial void ratio \( e_0 = N \). The influences of parameter \( n \) on confined compression lines and confined compression stress ratio \( \eta_0 \) of normally consolidated soil are shown in Fig. 5.

![Figure 5](image_url)

(a) Relationship of \( \sigma_v - e \)  
(b) Relationship of \( n - \eta_0 \)

In Fig. 5(a), little difference among the confined compression lines with different \( n \) is observed in \( e \sim \ln \sigma_v \) coordinates. However, confined compression stress ratio \( \eta_0 \) illustrated in Fig. 5(b) increases with parameter \( n \). Accordingly, after measuring \( \eta_0 \) from test data, the appropriate \( n \) value can be speculated from equation (14). The following equations are applied to explain why there is little difference in Fig. 5(a). According to the yield function (6), the equation of the confined compression line in the \( e \sim \ln p \) coordinates is written as:

\[
e = N - \lambda \ln p - (\lambda - \kappa) \ln \left(1 + \frac{\eta_0^2}{M^2}\right)
\]  

(16)

The relationship between mean stress \( p \) and vertical stress \( \sigma_v \) is:

\[
p = 3 \sigma_v / (3 + 2 \eta_0)
\]  

(17)

Substituting equation (17) into equation (16), the following equation is deduced:

\[
e = N - \lambda \ln \sigma_v + \Delta
\]  

(18)

In equation (18), \( \Delta = -\ln\left\{3(3 + 2 \eta_0)\right\}(1 + \eta_0^2/M^2)^{\lambda - \kappa} \). Setting general parameter values: \( \lambda = 0.2 \), \( \kappa = 0.04 \), \( M = 1.2 \), \( N = 2.0 \), \( \nu = 1/3 \). When \( n \) is 2, 4, 6 and 8, \( \Delta \) is respectively 0.031, 0.030, 0.027 and 0.025, which is too little to influence the void ratio in equation (18).

So far, a modification of Camclay dilatancy equation by parameter \( n \) is implemented for more
reasonably reproducing the constant stress ratio $\eta_0$ of normally consolidated soils in confined compression. Nevertheless, this constant $\eta_0$ is still insufficient for reflecting the dynamic stress state of structured soils in confined compression.

During confined compression on structured soil, soil structure gradually decays and eventually disappears completely. Correspondingly, stress ratio $\eta$ will firstly reaches a larger value, then decreases, and finally approaches $\eta_0$, as shown in Figure 2 (b). And the structured constitutive model is expected to be able to reflect this phenomenon.

2.2 Description of confined compression on structured soil

The yield function and hardening law of the structured UH model\(^\text{[20]}\) are expressed as:

$$f = c_p \ln \left\{ \frac{p}{p_0} \left[ 1 + \frac{q^2}{M^2(p + s)} \right] \right\} - \frac{1}{\Omega_s} d\varepsilon^p = 0 \quad (19)$$

In equation (19), $\Omega_s = R(M_0 - \eta) (-\Delta e/\Delta e_0) (M^4 - \eta^4) / (M_1^4 - \eta_1^4)$; Stress ratio $\eta_s = q/(p + s)$; The cementation stress $s$ is the real-time intercept of yield surface on negative $p$-axis; Internal variable $R$ reflects the influence of structure or over-consolidation on modulus of soil. $R \rightarrow 0$ at the initial loading stage and $R \rightarrow 1$ finally as both of structure and over-consolidation decay completely; $\Delta e$ reflects soil structure level. Initially $\Delta e = \Delta e_0$ and $s = s_0$, both of $\Delta e$ and $s$ decay in equal proportion with loading. $M_1 = 6\left\{ \sqrt{\chi / R} \left[ 1 + \chi / R \right] - \chi / R \right\}$ and $\chi = M^2 / [12(3 - M)]$. $M_1$ decreases with the increasing $R$, $M_1 \rightarrow 3$ when $R \rightarrow 0$, $M_1 \rightarrow M$ when $R \rightarrow 1$. Applying the non-associate flow rule, the dilatancy equation (13) is modified as:

$$\frac{d\varepsilon^p}{d\varepsilon^d} = \Omega_s \cdot c_p \left\{ (M^2 - \eta_0^2) dp + 2\eta_0 dq + \Omega_s \eta_0^2 \right\} \quad (21)$$

In equation (21), $\Omega_b = -\zeta Rs$, is used to describe the decay of cementation stress $s$:

$$d\varepsilon^d = \Omega_b d \left\{ c_p \ln \left[ \frac{p + q^2}{M^2(p + s)} \right] \right\} \quad (22)$$

Combined equations (4), (5) and (21), confined compression state of structured soil is described as:

$$1.5 = \frac{\kappa \cdot \frac{dp}{\Delta e_0} + \Omega_s \cdot \frac{\Delta e_0}{\Delta e_0} (M^2 - \eta_0^2) dp + 2\eta_0 dq + \Omega_s \eta_0^2}{\kappa \cdot \frac{1 + v}{1 - 2v} \frac{dp}{dy} + \Omega_s \cdot \frac{1 + v}{1 - 2v} (M^2 - \eta_0^2) dp + 2\eta_0 dq + \Omega_s \eta_0^2} \frac{\eta_0^{n-1}}{M^2 - \eta_0^2} \quad (23)$$

Considering that the stress ratio $(q/p)$ of structured soil all long varies in confined compression, "$dq/dp = \eta$" is no longer valid, so increments $dp$ and $dq$ are retained in equation (23). Being identical to equations (10) and (14), the numerator and denominator of equation (23) are also composed of the elastic $\kappa$ items and the plastic $(\lambda - \kappa)$ items. If only elastic $\kappa$ items are considered, the stress state $(dq/dp)$ is expressed as:

$$dq/dp = 3(1 - 2v)/(1 + v) \quad (24)$$

If only plastic $(\lambda - \kappa)$ items are considered, the stress ratio $\eta_s$ degrades to:

$$1.5 = (M^n - \eta_0^n) / (\eta_0^{n-1}) \quad (25)$$

Actually, elastic and plastic strains occur simultaneously, stress state by equation (23) falls in between the equation (24) and (25). The weight ratio of elastic and plastic items is expressed by the ratio \{$\kappa / [\Omega_s(\lambda - \kappa)]$\}. Comparing with equation (14), an internal variable $\Omega_s$ is added into equation (23). $\Omega_s$ evolves with soil structure and influences the real-time elastic-plastic weight ratio.

In order to illustrate the function of $\Omega_s$, a confined compression by structured UH model is simulated in Fig. 6, setting the parameters $\lambda = 0.2$, $\kappa = 0.04$, $M = 1.2$, $N = 2.0$, $\nu = 1/3$, $n = 2.0$, $\Delta e_0 = 0.8$, $s_0 = 60$ kPa, $\zeta = 20$. The initial states are $c_0 = \sigma_c = 1$ kPa and $e_0 = 1.3$. 

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As illustrated in Fig. 6(a), confined compression line for normally consolidated soil is a straight line in $e \sim \ln \sigma_v$ coordinates. Referring this straight line, the confined compression line for structured soil firstly crosses over it, then turns back, and finally approaches it. In Fig. 6(b), stress ratio $\left(\sigma_h/\sigma_v\right)$ for structured soil firstly decrease to a lower value than that of normally consolidated soil; with confined compression proceeding and soil structure decays, $\left(\sigma_h/\sigma_v\right)$ increases, and finally approaches that for the normally consolidated soil. This stress state evolution of structured soil in confined compression is in agreement with the experimental data illustrated in Fig. 2(a). The key point for the reasonable reproducing stress state evolution of structured soil is attributed to the internal variable $\Omega_s$. As illustrated in Fig. 6(c), dynamically evolved $\Omega_s$ currently modifies the elastic-plastic weight ratio $\{\kappa/[\Omega_s(\lambda - \kappa)]\}$. In the initial loading stage, $\Omega_s \to 0^+$ make elastic $\kappa$ items in equation (23) are predominant, while in the final loading stage, $\Omega_s \to 1^+$ make plastic $(\lambda - \kappa)$ items in equation (23) are predominant. In other words, about stress state of confined compression on structured soil, equation (24) mainly determines the initial stress state and equation (25) mainly determines the final stress state.

Influence of parameter $n$ in equation (25) is discussed in the last part, this part demonstrates the influence of Poisson's ratio $\nu$ in equation (24) on confined compression stress state simulation. Different $\nu$ values ($\nu = 0.1, 0.2$, and $0.3$, respectively) are applied in simulations. The other model parameters are: $\lambda = 0.2$, $\kappa = 0.04$, $M = 1.2$, $N = 2.0$, $n = 3.0$, $\Delta e_0 = 0.8$, $s_0 = 60kPa$, $\zeta = 20$, the initial stress states and initial void ratio are $\sigma_s = \sigma_f = 1kPa$, $e_0 = 1.3$. As illustrated in Fig. 7, with the parameter $\nu$ decreasing, stress ratio $\left(\sigma_h/\sigma_v\right)$ at the early confined compression stage decreases accordingly. With confined compression proceeding, stress ratio $\left(\sigma_h/\sigma_v\right)$ finally tends to be consistent with that of normally consolidated soil. Herein the confined compressive stress ratio $\eta_0$ of normal consolidated soil is indeed slightly affected by parameter $\nu$ according to equation (14), considering the elastic-plastic weight ratio $\{\kappa/(\lambda - \kappa)\}$, the influence of $\nu$ on $\eta_0$ can be neglected.
Figure 7. Influence of parameter $\nu$ on confined compression simulations of structured soil
After the above discussion, the structured UH model is modified in the following 2 points for simulating structured soil in confined compression: (a) Properly selected parameter $\nu$ in equation (24) is applied to simulate the initial stress state of confined compression; (b) Properly selected parameter $n$ in equation (25) is applied to simulate the final stress state in confined compression.

3. Model verification
In order to verify the bonding structured UH model in confined compression simulation, experimental data of three kinds of structured soil are compared with the simulations of the model. In the figures, the points represent test data and the solid lines represent simulations. According to literature [24] and [25], the model parameters are listed in Table 1. Since the compression data of remolded soils are not provided in the literature, parameter $N$ applied in model calculation is estimated. Considering that the data of shear test are not provided in literatures, for simplicity, model parameter $s_0$ is set to be 0kPa.

Table 1. Model parameters applied in simulations

|                | $\lambda$ | $\kappa$ | $N$ | $M$ | $\nu$ | $n$ | $\Delta e_{0}$ | $\zeta$ | $s_0$ |
|----------------|------------|----------|-----|-----|-------|-----|----------------|---------|-------|
| Bothkennar clay| 0.145      | 0.014    | 2.2 | 1.20| 0.1   | 9.0 | 0.39           | 30.0    | 0kPa  |
| Louiseville clay| 0.225     | 0.005    | 2.6 | 1.32| 0.1   | 3.0 | 0.98           | 100.0   | 0kPa  |
| Yamashita clay  | 0.142      | 0.013    | 2.8 | 1.786| 0.05  | 2.0 | 0.68           | 30.0    | 0kPa  |

Bothkennar clay is a marine from Scotland. It was deposited 6000 to 8500 years ago, whose soil layers were very uniform. The soil samples were taken 8m underground and its plastic index was about 40. Louiseville clay is a part of Champlain Sea clay. It was deposited 8500 to 12500 years ago along the Saint Laurence River in Canada. The soil samples were taken 20m underground and its plastic index was between 47 to 55. Yamashita clay is a marine clay in Yokohama Port, Japan. The soil samples were taken from the layer 20m underground, which was deposited 7700 years ago. And the samples' plastic index was between 47 to 55. Comparisons between test data and model calculations of these three natural clays under confined compression are illustrated in Fig. 8~10.

According to the test data, stress ratio $\left(\sigma_h/\sigma_v\right)$ for all the three structured soils firstly come to be lower in early loading stage, and finally tends to a constant value $\left(\sigma_h/\sigma_v\right)_{f}$ in the later loading stage. From the $\sigma_v$~$e$ test data, Bothkennar clay exerts weaker structure decay compared with Louiseville clay and Yamashita clay, so the difference between $\left(\sigma_h/\sigma_v\right)_{\text{min}}$ and $\left(\sigma_h/\sigma_v\right)_{f}$ of Bothkennar clay is smaller than that of the other two structured soils. About the stress ratio comparisons, calculated stress ratio $\left(\sigma_h/\sigma_v\right)$ for all the three structured soil tends to be constant earlier than the corresponding test data. That means the simulated lateral stress is greater than the practical lateral stress, which is fortunately conservative. Comparisons between test data and model simulations indicate that, by properly selecting parameters $\nu$ and $n$, the extended bonding structured UH model is qualified in simulating confined compression on structured soil.
Figure 8. Comparison of confined compression test data and model simulation of Bothkennar clay

Figure 9. Comparison of confined compression test data and model simulation of Louiseville clay

Figure 10. Comparison of confined compression test data and model simulation of Yamashita clay
4. Conclusion
In order to reproduce confined compression behaviors of structured soil, the bonding structured UH model is extended by properly selecting Poisson's ratio \( \nu \) and introducing parameter \( n \). The proper selected Poisson's ratio \( \nu \) and parameter \( n \) are respectively applied to reproduce the stress state in early and later stage of confined compression on structured soils.

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