Constitutive modelling hydro-mechanical behavior of unsaturated loess with a loss of structure

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ABSTRACT

This work develops a constitutive model for unsaturated nature loess considering the loss of structure upon wetting, the proposed model is an unsaturated kinematic hardening structure model (UKHSM) in the framework of Bishop’s effective stress approach. The UKHSM is characterized by isotropic and kinematic hardening law in the bounding surface formulation, as earlier proposed by Muir et al. The UKHSM incorporates measuring structure degradation caused by moisture variation and describes the essential phenomena of pre-failure behavior of natural loess, including load&wetting induced collapse behaviors. CU, CD and CW triaxial tests on Henan loess have been modeled to validate good capability of the UKHSM.

Keywords: structural loess, unsaturated kinematic hardening structure model, constant water content, Bishop’s effective stress

1 INTRODUCTION

Loess is a typical representative of collapsible soil with a honeycomb-type meta-stable structure, which is susceptible to collapse upon wetting and induces considerable reduction of plastic volume strain. The collapse triggering mechanism for various collapsible soils has been reported in the literature (e.g., Fredlund et al. 1995). For collapsible loess, the damage of its structure caused by increasing moisture content is essentially apart from those for sampling, laboratory testing, or geotechnical loading. From a perspective of micromechanics, as the water content increases beyond some critical value, loess structure fails accompanied by fundamental particle rearrangement, transforming an initial stable open structure into a remolded close structure. (Dijkstra et al., 1995).

The natural structure is significant in analyzing mechanical behavior of loess, especially peak softening when the loess is nearly saturated or the suction abruptly changes. There are the constitutive models predicting the behavior of the destructuration on the natural structural soil including clay and other clayey soils. Those models are based on the saturated remoulded material and thus the extension of the Cam-clay model via introducing new parameters to describe the structure or within a frame work of boundary models(Rouainia et al 2000; Asaoka et al.2000; Callisto et al.2002; Liu et al. 2002; Taiebat et al. 2010; Morvan et al. 2010). Although those models well predicts mechanical behavior and collapsibility of the compacted reconstituted soil or saturated natural structural soil, seldom did they combine the unsaturated state with structural loss. Many established models predicting the collapsibility of soils upon wetting were extended from the Barcelona Basic Model (Alonso et al.1990) and the behavior in peak and post–peak states were poorly modeled(Wheeler et al 1995; Cui and Delage, 1996; Chen, 1999; Chen et al., 1999; Rampino et al., 2000; Sun et al., 2004, 2007a,b). Besides, those models were validated based on suction-constant compression tests in CD conditions but the suction depends the permeability and drainage conditions. Under particular circumstances where the suction is variable but the moisture content stays constant, a shear failure of a relatively short time will occur(Maleki et al 2012;Ng et al 2001;Rahardjo et al 2004). Plus, a cost of complexity of those models is needed to ensure that the current stress points are located on the inner yield surface at every integration step (Zhao et al 2005), so it is necessary to simplify the calculation through implementation into finite element analysis.

In this study, the structural loss of loess consists of two coupled part, the strength degradation upon wetting and loading, the UKHSM is an advanced version of the kinematic hardening structure model (KHSM) to unsaturated states in the framework of Bishop’s effective stress. The UKHSM was enabled to predict the mechanic behaviors under the wide-ranging conditions including the saturated undisturbed loess under CU condition and the unsaturated undisturbed
loess under CD and CW conditions. Experimental data from CD and CW triaxial tests on natural loess are presented to support the proposed model.

2 CONSTITUTIVE FRAMEWORK AND FORMULATIONS

As noted in the introduction, the effective degree of saturation \( S'_{r} \) and Bishop’s effective stress \( \sigma_{y}' \) are selected as the basic constitutive variables since is not 1 at zero suction in some special cases, as mentioned in Zhou et al, which can be written as

\[
S'_{r} = \frac{S_{r} - S_{r}^{0}}{S_{r}^{e} - S_{r}^{0}}
\]  
(1)

\[
\sigma_{y}' = \sigma_{y} + S'_{r} s \delta_{ij}
\]  
(2)

where \( S_{r}^{0} \) is the degree of saturation at zero suction, \( \sigma_{y} \) is the net stress \( \sigma_{y}^{total} - u^{a} \), \( \sigma_{y}^{total} \) is the total stress, \( u^{a} \) is the pore air pressure, \( s \) is the suction \( u_{w} - u_{a} \); and \( \delta_{ij} \) is the Kronecker delta. When the suction vanishes, the Bishop’s effective stress \( \sigma_{y}' \) is the same as Terzaghi’s effective stress \( \sigma_{y}^{'} \). Note that in most engineering practice, the pore-air pressure \( u^{a} \) does not vary substantially and can be considered as zero for convenient analysis, thus the assumption that \( s = -u_{w} \) and \( \sigma_{y}' = \sigma_{y} \) will be used throughout the rest of the paper. The isotropic state of unsaturated soil has been researched into about the mechanical and hydraulic behavior based on the adoption of Bishop’s effective stress as a variable exclusively. One advantage of selecting the Bishop’s effective stress as the unique variant is that the expression, similar to the effective stress for saturated soil, could realize the transition from the unsaturated stress state to the saturated. Another benefit is that it renders the convergence of the UMAT subroutine more compatible with the definition of SWCC, while most of existing models were established but not applied into related finite element.

3 THE KHSM AND ITS ADAPTATION FOR UNSATURATED SOILS

In this study, the advanced version of kinematic hardening structure model described in detail by Rouainia(2000) selected as the basic model. By introducing a fictitious outer structure surface and a bubble enclosing the elastic domain at the current stress state, the structure model is primarily established according to a kinematic hardening rule. The elastoplastic behavior is described within the framework of plasticity theory, comprising of a yield surface constraining the accessible elastic stress states, a flow rule defining the mechanism of plastic deformation and hardening functions defining the magnitude of plastic strains. As the moisture content of soil varies, the elastoplastic strains evolve continuously inside and on the boundary surface as a result of change in suction, thereby demonstrating the degradation of structure with strain upon wetting and avoiding an abrupt change in classical elastoplasticity when the stress state reaches the outer yield surface.

3.1 Elastic constitutive relationship

With this Bishop’s effective stress variables defined above, all the equations described apply both to unsaturated and saturated states. The UKHSM is developed in effective stress space. For convenience, we will adopt the cylindrical symmetry corresponding to the conventional triaxial test. Thus in the p-q plane of triaxial tests, the mean Bishop effective stress \( p' \) and the deviatoric stress \( q \) are calculated as follows:

\[
p' = \frac{\sigma_{1}' + 2\sigma_{3}'}{3}
\]  
(3)

\[
q = \sigma_{1}' - \sigma_{3}'
\]  
(4)

where \( \sigma_{1}' \) and \( \sigma_{3}' \) are the effective axial stress and effective confining pressure, respectively.

Generally the additive decomposition of the strain rate into elastic and plastic parts respectively:

\[
d\varepsilon = d\varepsilon^{e} + d\varepsilon^{p}
\]  
(5)

The response associated with the elastic is expressed in terms of the bulk and shear moduli, \( K \) and \( G \), which are assumed to depend linearly on the pressure \( p' \):

\[
K = \frac{p'}{\kappa}
\]  
(6)

\[
G = \frac{3(1-2v)}{2(1+2v)}K
\]  
(7)

where \( \kappa \) is the slope of the swelling line in a volumetric strain logarithmic mean stress compression plane rather than in a specific volume logarithmic mean stress compression plane and \( v \) is a constant Poisson’s ratio. It has been verified that \( \kappa \) is a constant despite of variation of the moisture content or suction. Therefore a hypoelastic formulation is considered for the elastic part of the model as:

\[
\sigma' = D':\dot{\varepsilon}^{e}
\]  
(8)
3.2 Unsaturated mechanical behavior and hydraulic responses

Recent research (Zhou et al 2012) has shown that the hydraulic behavior is controlled by the suction as well as the volumetric strain due to net stresses \((\sigma_{ij})\). The change in \(S'_v\) can be expressed conceptually as:

\[
dS'_v = \frac{\partial S'_v}{\partial S} \, dS + \frac{\partial S'_v}{\partial \epsilon_{v}\sigma} \, d\epsilon_{v}\sigma
\]  

(9)

where \(\frac{\partial S'_v}{\partial \epsilon_{v}\sigma}\) is a general function that defines the effect of volumetric strain on the effective degree of saturation that is caused by a net stress change \((\epsilon_{v}\sigma)\).

In order to calibrate the first term on the right hand side of the equation, a laboratory water retention test should be conducted to attain the SWCC of studied soils, which could directly reflect volumetric strains resulted from the change in the suction. Furthermore, the sorption and exsorption curves defining relationship between the negative pore pressure and the saturation of the porous element are consistent with the SWCC.

As for the second term on the right hand of the equation, there are several attempts to simplify and attain the good consistency, in this paper this term is suggested as follows:

\[
\frac{\partial S'_v}{\partial \epsilon_{v}\sigma} \, d\epsilon_{v}\sigma = \lambda_v \, d\epsilon_{v}\sigma
\]  

(10)

where \(\lambda_v\) is the slope between the effective saturation and the volumetric strain caused by the net stress change. According to the isotropic compression tests under different constant suction, the relationship between the saturation and void ratio, namely the volumetric strain tends to be linearity, which proves the validity to formulate the second term of the Eq. (9).

Besides, it is generally accepted that the volumetric strain can be calculated depending on the movement of the normal compression surface as the saturation or suction of soil varies. Based on the isotropic compression tests under different suction including the driest state \((S'_v = 0)\) and the saturated state \((S'_v = 1)\), the normal compression surface can be discretized into a series of normal compression lines in the \(p' - V\) plane as long as the suction or the saturation remains constant. The isotropic compression lines are formulated in the \(\ln p' - \ln V\) plane with a constant suction in accordance with the based kinematic hardening model, which is given by:

\[
\ln V = N - \lambda^*(s) \ln p'
\]  

(11)

\[
\lambda^*(s) = \lambda^*(0) \cdot \left[1 - \left(1 - \frac{p'}{p_n'}\right)^{-1}\right]
\]  

(12)

where \(p'\) is the mean Bishop effective stress, \(N\) is the intercept of the normal compression lines with the \(\ln V\) axis. The compression index \(\lambda^*(s)\) is the slope of normal compression line in \(\ln V - \ln p'\) compression plane, which is assumed to be a function of the suction and can be fitted through the results of a series of isotropic compression tests under constant suction. As the suction remains constant during the isotropic compression, it is testified that a series of compression lines intersect at an approximate point under a relatively higher net stress in \(\ln V - \ln p'\) compression plane. With the decrease of suction, the compression index \(\lambda^*(s)\) ranges and the variation rules of \(\lambda^*(s)\) differ in soils. Generally the influence of suction on compression index depends on the soil properties, an initial increase of \(\lambda^*(s)\) is noticed at low suction for soils of low plasticity such as Kaolin and silt(Geiser et al 2006). However, the initial yield stress of different soils all conform to a principle that the higher the suction, the greater the initial yield stress.

Therefore, the concept of initial yield curve is proposed to be a set of initial yield stress while the soil of the same initial state is compressed desiccating or moistening. The specific equation is similar with the loading collapse curve and is defined as follows:

\[
\frac{p'_y}{p'_n} = \left(\frac{p'_y}{p'_n}\right)^{(\lambda^*(0) - \lambda^*(1)) / (\lambda^*(1) - \lambda^*(0))}
\]  

(13)

In this paper the \(p'_n\) can be obtained by the approximate intersection point of a series of normal compression lines with constant suction, which represents the isotropic stress state with no addition of volumetric strains as the suction changes, in general \(p'_n\) is suggested to a small value of pressure.

4 PLASTIC CONSTITUTIVE RELATIONSHIPS BASED ON BOUNDING SURFACE THEORY

The UKHSM is capable to predict the mechanical behavior of unsaturated collapsible soil represented by loess. Therefore the significance of modelling framework consists in the irreversible strains occurring in the wetting and loading stage, which refers to the structural loss induced by the variation of moisture and applied loads. The bounding surface theory including the yield surface equation, flow rule, hardening law and plastic modulus has been illustrated in this paper. A complete set of equations of an elastoplastic model for unsaturated structured soils upon wetting and loading is now formulated.
4.1 Yield surface equation

The bubble, reference and structural surface plotted in Fig. 1 have the similar elliptical shape. The reference surface encloses the region to describe the intrinsic behavior of the reconstituted soils, which corresponds to the final state as the soil barely retains structural strength. Here the formulation proposed by Rouainia (2000) is used. For simplicity of application, the proposed model in this paper is established without regard to the initial anisotropic stress state.

The reference surface equation is as followed:

\[ f = \frac{3}{2M_{\theta}} q^2 + p - p'_s \]  \hfill (14)

here the \( p' \) is the mean Bishop effective stress and \( q \) is the deviatoric stress. \( p'_s \) is scalar variable defining the size of reference surface. The dimensionless scaling function \( M_{\theta} \) affects the shape of reference surface, the bubble and the structural surface in the deviatoric space, adopting the suggested equation in Sheng(2000):

\[ M_{\theta} = M_c \left[ \frac{2m^4}{(1 + m^2) - (1 - m^2) \sin 3\theta} \right]^{1/4} \]  \hfill (15)

where \( M_c \) is the critical state stress ratio for axisymmetric compression, \( \theta \) is the Lode’s Angle. The material parameter \( m \) is the ratio between the radii of the sections through the surface for axisymmetric extension and compression in the deviatoric plane, which can be calculated by:

\[ m = \frac{M_c}{M_e} \]  \hfill (16)

In this paper, the modification based on the previous KHSM is to quantify \( p'_i \) with respect to the initial yield stress in unsaturated states, the value of \( p'_s \) can be determined:

\[ p'_s = \frac{1}{2} p'_r \]  \hfill (17)

Based on the Eq. (13) and Eq. (17), \( p'_s \) translates from \( p'_i \) to \( \frac{1}{2} p'_r \) while the soil desiccates with the suction increasing from zero, the suction uniquely controls the compression index \( \lambda'(s) \) and initial yield stress \( p'_s \) despite of initial void ratio or initial stress state.

The mathematical equation of the structure surface is given by

\[ f = \frac{3}{2M_{\theta}} q^2 + (p - r p'_s)^2 - (r p'_s)^2 \]  \hfill (18)

where \( r \) is the ratio of the sizes of the structure surface and the reference surface. The translation of the parameter \( r \) can realize the process of destructuration by varying the size and the center of the outer surface, theoretically, the relative location with respect to the center of the reference and structure surface in Fig.1 reflects the initial anisotropic stress state, nevertheless, the proposed model depreciates the anisotropic index \( \eta \) for simplicity thus both the reference and structural surface intersect at the coordinate origin.

4.2 Hardening law

The hardening rule consists of the isotropic hardening and the kinematic hardening. A volumetric hardening rule is adopted where the scalar \( p'_s \) is controlled by plastic volumetric strain rate for both saturated and unsaturated soils:

\[ dp'_s = \frac{p'_i}{\lambda'(s) - k} d\varepsilon_p^v \]  \hfill (20)

As for the kinematic hardening, the center of bubble is translating with the current stress on or inside of the bubble and simultaneously to avoid the intersection of the bubble and structural surface. The equation obtained by geometric requirement and consistency is given by

\[ \vec{a} = \vec{\dot{a}} + (\vec{a} - \vec{z}) \left[ \frac{\dot{\varepsilon}_p}{\varepsilon'_p} \right] \vec{r} + \vec{\dot{a}} \left[ \frac{\dot{\varepsilon}_p}{\varepsilon'_p} \right] + \vec{\dot{a}} \left[ \frac{\dot{\varepsilon}_p}{\varepsilon'_p} \right] \vec{r} - \vec{\dot{a}} \left[ \frac{\dot{\varepsilon}_p}{\varepsilon'_p} \right] \vec{r} \]  \hfill (21)

where \( \vec{a} \) is the Lode’s Angle. The ratio of the sizes of the structure surface to the reference surface is the critical state stress ratio for axisymmetric compression, \( \theta \) is the Lode’s Angle. The material parameter \( m \) is the ratio between the radii of the sections through the surface for axisymmetric extension and compression in the deviatoric plane, which can be calculated by:

\[ m = \frac{M_c}{M_e} \]  \hfill (16)
In this equation, $\tilde{\sigma}$ and $\tilde{\sigma}$ denote the stress tensor of the center of bubble and structural surface, respectively and $\tilde{\sigma} = \sigma - \tilde{\sigma}, \tilde{\sigma} = \sigma - \tilde{\sigma} ; n$ is the normalized stress gradient on the bubble at the current stress state, $\sigma$, is the conjugate point on the structure surface of any stress point $\sigma$. A flow rule associated with the yield function of bubble surface is assumed.

The plastic modulus is calculated by interpolation in accord with the distance between the current stress state $\sigma$ and the conjugate point $\sigma_c$ on the structural surface, of which the equation is consistent with Rouainia(2000). Nevertheless the wetting path which is similar with the unloading path leads to a decrease in the suction within a framework of Bishop’s effective stress, the smooth variation of the stiffness is ensured during the translation of the bubble after the adaptation of the original formula given as follow:

$$H = H_e + \frac{B \rho_s}{\left( \tilde{\lambda} (s) - \kappa^* \right) R \left( b_{\max} \right) \left( b_0 \right) \left( b_0 - b \right)}$$

where $H_e$ is the plastic modulus at the conjugate stress point $\sigma_c$, $B, \psi_1, \psi_2$ are constant material parameters; the constant parameter $R$ represents the ratio of the sizes of the elastic area and the reference surface. $b$ denotes the distance between the $\sigma$ and $\sigma_c$ which can be obtained by $b = n : (\sigma - \sigma_c)$. $b_{\max}$ and $b_0$ are respectively the allowable maximum and the initial value of $b$ when unloading. The introduction of $b_0$ legitimizes the variation of the plastic modulus.

4.3 Model calibration

Most of the parameters of the proposed model can be obtained directly from laboratory experiments including two experiments of isotropic compression tests under constant suction to determine the relationship between $\tilde{\lambda} (s)$ and suction, one set of undrained triaxial tests of saturated soil provide data to calibrate the $M, m, A, B, \psi_1, \psi_2, k$, as suggested by Rouainia (2000) and one-dimensional compression tests (loading and unloading) of saturated soil to calibrate the $\tilde{\lambda} (0), \kappa^*$, $p_r$ and $r$ at a first estimate. In engineering practice, most of the parameters related to the critical state and bounding surface like $M, m$, and $\nu$ can be deduced by the effective friction angle $\varphi$.

5 VALIDATION RESULTS ON CD AND CW PATH ON NATURAL LOESS

In order to investigate the influence of the intrinsic structure loss upon wetting based on the advanced kinematic hardening model apart from the compacted reconstituted soil, additional triaxial tests with respect to the same condition as the CD and CW path were carried out on the natural loess corresponding to different initial suction controlled.

5.1 Specimen preparation

The tested soil in this study was natural loess taken from a site in Nanyang area, Henan, China.

The specimens were 76 mm high and 38 mm in diameter and had a bulk density of 1.70kg/m3 and a dry density of 1.35 kg/m3. Special care was taken in sample preparation to ensure the reproducibility of the initial state. The water content were made approximately the plastic limit $\omega_p$ via trickling water upon surfaces of sample hermetically sealed, conserved for two weeks to ensure the uniformity of water distribution. In order to remove air bubbles accumulated as the base of the porous ceramic disk during unsaturated soil testing and saturate the ceramic disk, a back pressure of 25kPa was applied prior to the installation of samples to observe a thin layer of water film on the top of ceramic disk. Then the specimen was placed on the ceramic disk with a high-air-entry of 5bar. This means that the maximum matric suction applied should not exceed 500kPa. In terms of different test conditions as CW, additional particular filter paper, which is waterproof but allows the air through, was attached on the top of the sample to prevent the entrance into the pore-air pressure control system of the water from the inner sample and simultaneously maintain the constant water content. After preparation, a confining pressure of 10 kPa was applied to stand the specimen. Based on the results of the oedometer tests on undisturbed samples, the consolidation yield stress $\sigma'_0$ was determined to be 43.6 kPa, determining the initial yield stress $p'_0$. The initial structural parameter, $r_0 = 2$, is determined approximately from $\sigma'_0/\sigma'_w$ (Huang et al 2011).

Table 3 Index properties of Henan loess.

| Index properties | Value |
|------------------|-------|
| Water content, $\omega$ (%) | 26 |
| Plastic limit, $\omega_p$ (%) | 21 |
| Plasticity index, $I_p$ | 10 |
| Specific gravity, $G_s$ | 2.69 |
| Sand(%) | 2 |
| Clay(%) | 5 |
| Silt(%) | 93 |
| Initial void ratio, $\varepsilon_0$ | 0.95 |
| Over consolidation ratio, OCR | 1 |
| Coefficient of lateral pressure at rest, $K_0$ | 0.52 |
| Residual saturation, $S_{res}$ | 0.24 |
5.2 Test program and apparatus

Table 2 illustrates the test scheme of triaxial tests using triaxial apparatus manufactured by GDS Corporation. Different testing procedures before shearing were conducted on the samples for the CD and CW tests. The CD tests including the stabilization of suction, isotropic consolidation under constant suction and shearing tests under constant confining pressure and suction, the samples were firstly subjected to the suction control by increasing the pore air pressure slowly enough while maintaining the pore water pressure of 25kPa and then isotropically consolidated to certain confining pressure under constant suction. As for the CW tests including the measurement of suction, isotropic compression under constant water content and undrained shearing tests, after the saturation of ceramic disk, the pore air pressure was constant till the end of the whole tests, the pore water pressure slowly increased as a result of axis-translation under the mean confining pressure controlled at 5kPa. Therefore the difference between the pore air pressure and the pore water pressure were measured as the initial suction of the samples of certain water content. Subsequently, the samples were isotropically compressed. Note that the valve for the air pressure line was opened thus allow a drainage and the valve for the water pressure line was closed during the whole tests. When an equilibrium condition had been achieved under the applied pressures (i.e., $\sigma_3$, $u_w$, and $u_a$) for both CD and CW tests after consolidation, the soil specimen was sheared under constant suction or undrained conditions at the strain rate of 0.009mm/min. The axial load was applied by an axial actuator and measured by a load transducer mounted on the top of the cell. The cell-pressure and back-pressure controllers are used to independently control the cell pressure and back pressure using de-aired water. The pore air pressure controllers are applied based on the modified pressure seek algorithm for large volume change of air with pressure.

Fig. 2-3 presents the undrained compression behavior of saturated Henan natural loess with two different isotropic pressures of 50 and 100kPa, of which the purpose is to validate the UKHSM when applied to the saturated condition and calibrate the material parameters.

Table 4. Test conditions for undrained triaxial tests on Henan natural loess.

| Test number | Initial water content/% | Suction before consolidation/kPa | Mean Confining Pressure/kPa |
|-------------|-------------------------|---------------------------------|---------------------------|
| CU-1        | 36.5                    | 0                               | 50                        |
| CU-2        | 36.5                    | 0                               | 100                       |
| CD-1        | 12                      | 50                              | 200                       |
| CD-2        | 12                      | 200                             | 200                       |
| CW-1        | 14.2                    | 140                             | 200                       |
| CW-2        | 9.7                     | 320                             | 200                       |

5.3 Test results and model simulation

Based on the test results of the isotropic compression under constant suction and one-dimensional compression test, the relationship between the initial yield stress, the compression index and the suction are determined. The calibration of material parameters was based on the result of the triaxial shearing tests on the saturated loess. Table 4 shows the values of model parameters for natural loess. The other series of tests designed in the Table 2 have been simulated using previous material parameters. These values $e_0 = 0.95, r_i = 1.9$, for the initial stress state have been used for all simulations, while the the values $p'_i$ and compression index $\lambda^*(s)$ are dependent on the initial suction before shearing.

| Parameters | Value |
|------------|-------|
| Slope of swelling line, $k^*$ | 0.018 |
| Poisson’s ratio, $\nu$ | 0.342 |
| Slope of normal compression line, $\lambda(0)^*$ | 0.18 |
| Critical state stress ratio, $M$ | 1.066 |
| Ratio of extension and compression strengths, $m$ | 0.738 |
| Ratio of size of bubble and reference surface, $R$ | 0.1 |
| Stiffness interpolation exponent $\psi_1$, $\psi_2$ | 1.1 |
| Stiffness interpolation parameter, $B$ | 0.5 |
| Destruction parameter, $A$ | 0.5 |
| Destructuration parameter, $k$ | 1.5 |
| Initial degree of structure, $r_i$ | 2 |
| Initial yield stress of saturated state, $p'_0$ (kPa) | 43.6 |
| Compressibility parameter, $\xi$ | 0.733 |
| Compressibility parameter, $\tau$ | 0.0395 |
| Reference pressure, $p'_R$ (kPa) | 1 |

Fig. 2 Simulations of the CU triaxial tests. Axial strain versus deviator stress.
In Fig 2-3, the response of the model are compared with the experimental results, the UKHSM proposed in this study is capable of accurately simulating the stress-strain relationship, especially in the peak strength and the strength decay with the structural loss. The abrupt change in the stress path due to the sharp increase in pore pressure is reasonably simulated in the proposed model, of which the particular constitutive response in structural soils is superior to the conventional hardening modified Cam-Clay models.

Fig. 3 Simulations of the CU triaxial tests. Stress paths

Fig. 4 Simulations of the CD triaxial tests. Axial strain versus deviator stress

Fig. 5 Simulations of the CD triaxial tests. Axial strain versus volume strain

Fig. 6 Simulations of the CW triaxial tests. Axial strain versus deviator stress

Fig. 7 Simulations of the CW triaxial tests. Axial strain versus volume strain

Fig. 6-7 shows the undrained compression behavior under different initial suction before shearing. There was a change in deviator stress development from non-distinct peak deviator stress during shearing to a post-peak softening in the pair of specimens CW-1 and CW-2. The CW-2 exhibited dilation after the initial...
compression and a failure mode as commonly observed in over-consolidated soils. This is because when shearing the reduction of effective mean stress due to the abrupt change of the suction may exceed the increase of effective mean stress due to the deviatoric stress.

6 CONCLUSIONS

In the present study, based on the observations and experimentations existing in the literature and a series of triaxial tests on natural loess on different loading paths with various suction conditions (CD and CW), a constitutive model was developed for developed for describing the stress-strain behavior of unsaturated structural soils considering the structural strength loss upon wetting and loading. The proposed model is an advanced version of the kinematic hardening structure model (KHSM) for adaptation to unsaturated states achieved in the framework of Bishop’s effective stress approach. The following conclusions could be drawn:

(a) General performance in describing the behavior of saturated structural soils in CU conditions especially in the peak strength and the strength decay with the structural loss validates the proposed model, of which the particular constitutive response in structural soils is superior to the conventional hardening modified Cam-Clay models.

(b) Through introducing the model into unsaturated state as well as the compressibility in accord with the matric suction, a good conformity is verified for both the transitional behavior and the volumetric response considering different loading paths and various suction conditions (CD and CW).

(c) The sub-stepping integration scheme used in this study is an explicit Euler method with automatic error control which is capable to realize relatively complex constitutive model, and the advanced model simplifies the material parameters with a minimal number of 10. Besides, most of the parameters are easily determined by the effective friction angle or limited in a certain range.

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