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On the constitutive modeling of partially saturated interfaces

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Generalization of soil–structure interface models from dry/saturated states to consider partially saturated states is studied in this paper. For this purpose, basic constitutive equations of a conventional elasto-plastic interface model are firstly presented. Then, consideration is given to the effect of partial saturation on definition of effective stress, location of the critical state line as well as the impact of interface state on plastic hardening modulus and dilatancy. For each concern, proper independent approaches together with associated constitutive equations are discussed to be included in the basic model as complementary ingredients. Among many different possibilities to combine complementary constitutive equations for effective stress, relocation of the critical state line with degree of saturation, and impact of the interface state on plastic hardening modulus and dilatancy, six essential cases are selected. Evaluations show that all six cases can realistically consider the impact of partial degree of saturation on the peak and residual shear strengths as well as the volume change behavior of unsaturated interfaces.

1. Introduction

Soil–structure interface is a particular class of soil–structure interaction in which, soil interacts with structure at the contact surface. The mechanical response of many soil–structure systems such as shallow and deep foundations, tunnel linings, landfill liners, retaining walls, buried pipes, soil nails and reinforced soil structures, as well as some in-situ tests like cone penetration test are strongly influenced by the stress–displacement–strength behavior of the soil–structure interfaces. Practically, interfaces are considered as weak zones since their shear strength is usually less than that of the surrounding soil. Research conducted in the past few decades has revealed that the mechanical behavior of soil–structure interfaces are influenced by several factors including soil gradation and mineralogy [7], structural material and roughness [7,13,23,50], soil density and normal stress level [7,11,12,48], stress path and normal stiffness condition [11–13,48]. However, recent experimental studies have indicated that the soil degree of saturation must be added to the above list since it can have a remarkable impact on the shear strength and volume change response of soil–structure interfaces (e.g., Miller and Hamid [43]; Hamid and Miller [20]; Khoury et al. [28]; Hossain and Yin [22]; Borana et al. [4]; Hatami and Esmaili [21]). In arid or semi-arid regions where the depth of unsaturated soil may be subjected to sizable seasonal changes, an in depth understanding of the mechanical behavior of unsaturated interfaces becomes practically vital.

For interfaces in dry condition, De Gennaro and Frank [6] and Mortara et al. [44] proposed constitutive models within the elasto-plasticity theory taking into account phase transformation, imposed normal stiffness, and residual strength. Hu and Pu [23] and Navayogarajah et al. [45] put forward interface models based on the disturbed state concept (*see* Desai [10] for review). De Jong et al. [8] published experimental evidence supporting that the mechanical behavior of soil–structure interfaces can be effectively explained within the critical state soil mechanics. Introducing critical state compatible interface models, Liu et al. [39], Liu and Ling [37], Lashkari [29,33], and Liu et al. [40] succeeded to simulate the mechanical behavior of dry interfaces within a wide range of densities and normal stresses using a unique set of parameters.

Constitutive modeling of unsaturated interfaces is essentially a young matter. Recently, Hamid and Miller [19] and Khoury et al. [28] modified the model of Navayogarajah et al. [45] and suggested the first constitutive model for partially saturated interfaces. Later, Lashkari [32] introduced a modification to the model of Lashkari [29,33] in order to simulate the behavior of unsaturated interfaces over a wide domain of density, net normal stress, and matric suction values using a single set of parameters. In accordance with the independent stress state variables concept of Fredlund and Morgenstren [15] and Fredlund and Rahardjo [16], constitutive equations in the models of Hamid and Miller [20], Khoury et al. [28],

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and Lashkari [32] have been established in terms of net normal stress, matric suction, and shear stress as the independent stress variables. More recently, Lashkari and Kadivar [34] introduced another extension to the model of Lashkari [33] for unsaturated interfaces by adopting the effective (skeleton) stress theory of Wheeler et al. [52] as well as the cementation parameter of Gallipoli et al. [17,18] into the model formulation.

In recent years, several modern approaches have been introduced in the literature for constitutive modeling of unsaturated soils (e.g., [17,18,24,27,38,46]). Herein and as a benchmark, the general formulation of the interface model of Lashkari [29,33] for dry and saturated interfaces is adopted. Subsequently, the limitations of the basic frame in modeling of the behavior of unsaturated interfaces are discussed. Then, healing strategies for each drawback are explained exhaustively. To this purpose, detailed constitutive equations for complementary ingredients are presented. The complementary ingredients can be combined in 18 different ways in order to enable the benchmark model to properly cover unsaturated interfaces. However, due to the limitation in number of pages, only six fundamental cases are selected. It is shown that all six cases can reasonably simulate the mechanical behavior of partially saturated interfaces.

2. Benchmark constitutive equations for soil–structure interface model

The incremental relationship linking the changes in effective (i.e., skeleton) stress and displacement vectors is (e.g., Mortara et al. [44]; Liu and Ling [37]; Liu et al. [39,40]; Lashkari [29,33]):

$$\{\dot{\boldsymbol{\sigma}}''\} = \frac{1}{t}[\mathbf{D}]^{ep}\{\dot{\boldsymbol{\Delta}}\} \tag{1}$$

where $\{\boldsymbol{\sigma}''\}$ is effective (skeleton) stress vector whose definition is presented in Section 4.1. $\{\boldsymbol{\Delta}\} = \left\{ \begin{matrix} v \\ u \end{matrix} \right\}$ is displacement vector in which u and v are, respectively, the horizontal and normal displacements. t represents the average interface thickness. Experimental studies using image analysis and particle image velocimetry as well as FEM simulations using advanced micropolar continuum theories have revealed that the average interface thickness usually varies from $2d_{50}$ to $12d_{50}$ depending on mineralogy, particles angularity, crushability and the interface state [7,23,33,39,40,44]. In Eq. (1), $[\mathbf{D}]^{ep}$ is the elastic–plastic stiffness matrix that is calculated by (e.g., Mortara et al. [44]; Liu and Ling [37]; Liu et al. [39,40]; Lashkari [29,33]):

$$[\mathbf{D}]^{ep} = [\mathbf{D}]^{e} - \frac{[\mathbf{D}]^{e}\{\mathbf{R}\}\{\mathbf{n}\}^{T}[\mathbf{D}]^{e}}{K_p + \{\mathbf{n}\}^{T}[\mathbf{D}]^{e}\{\mathbf{R}\}} \tag{2}$$

where a class of consistent constitutive equations for $[\mathbf{D}]^{e}$ (elastic stiffness matrix), $\{\mathbf{n}\}$ (yield direction vector), $\{\mathbf{R}\}$ (plastic strain rate direction vector), and K_p (plastic hardening modulus) are presented through Eqs. (3)–(8) in Table 1.

3. Definition of problem

In general, the benchmark constitutive model defined through Eqs. (1)–(8) is incomplete since it is not capable to distinguish partially saturated interfaces from dry and saturated ones. The current knowledge signifies that water menisci in partially saturated geomaterials affect the inter-particle forces and thus, definition of a generalized skeleton (effective) stress theory for such materials is necessary. The mentioned theory must be reduced into the well-known Terzaghi's effective stress theory and total stress, respectively, in fully saturated and dry conditions. Furthermore, bonding phenomenon associated with water menisci leads to the evolution of critical state void ratio. The latter issue is important in view of the fact that both dilation [Eq. (7) in Table 1], and plastic hardening modulus [Eq. (8) in Table 1] are direct functions of the distance between the current state from the critical state line (e.g., [5,32–34,36–40,42,51]). In the following sections, proper essential ingredients in order to complete the benchmark constitutive model are discussed. To this aim, proper concepts/elements for skeleton (effective) stress vector, relocation of critical state line, state parameters, state-dependent elements and soil–water characteristic curve are presented throughout Sections 4–8.

4. Effective (skeleton) stress concept

For partially saturated soils, Bishop [3] defined the effective (skeleton) stress tensor in the form:

$$\boldsymbol{\Sigma}'' = \boldsymbol{\Sigma}_{net} + \chi \times s\mathbf{1} \tag{9}$$

Table 1
Consistent equations for $[\mathbf{D}]^e$, $\{\mathbf{n}\}$, $\{\mathbf{R}\}$, and K_p in Eq. (2).

Description	Constitutive equation	Parameter(s) [a]	Equation
Elastic stiffness matrix	$[\mathbf{D}]^e = \begin{bmatrix} K_n^e & 0 \\ 0 & K_t^e \end{bmatrix} = \begin{bmatrix} K_{n0}^e\sqrt{\sigma_n''/p_{ref}} & 0 \\ 0 & K_{t0}^e\sqrt{\sigma_n''/p_{ref}} \end{bmatrix}$ K_n^e and K_t^e are, respectively, elastic normal and elastic shear moduli p_{ref} (=100 kPa) is a reference normalizing pressure	K_{n0}^e and K_{t0}^e	(3)
Yield function	$f = \tau - \eta\sigma_n'' = 0$ η, stress ratio, acts as a hardening variable τ is shear stress and σ_n'' is defined through Eq. (13)	–	(4)
Yield direction vector	$\{\mathbf{n}\} = \left\{ \begin{matrix} \partial f/\partial\sigma_n'' \\ \partial f/\partial\tau \end{matrix} \right\} = \left\{ \begin{matrix} -\eta \\ 1 \end{matrix} \right\}$	–	(5)
Plastic strain rate direction vector	$\{\mathbf{R}\} = \left\{ \begin{matrix} d \\ 1 \end{matrix} \right\}$	–	(6)
Dilation	$d = \left[A_0\sqrt{\frac{p_{ref}}{\sigma_n''}} + \frac{\eta}{M^b}\left(A_1 - A_0\sqrt{\frac{p_{ref}}{\sigma_n''}}\right)\right](M^d - \eta)$ M^b and M^d are, respectively, state-dependent peak and dilatancy stress ratios. M^b and M^d are mathematically defined in Section 7	A_0 and A_1	(7)
Plastic hardening modulus	$K_p = h_0 K_t^e\left(\frac{M^b}{\eta} - 1\right)$ where K_t^e is defined through Eq. (3)	h_0	(8)

[a] In the third column of Table 1, A_0 and A_1, and h_0 are dimensionless model parameters. K_{n0}^e and K_{t0}^e must be in kPa when p_{ref} = 100 kPa is used.