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Research Paper

A simple soil model for low frequency cyclic loading

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ABSTRACT

A three-dimensional elastoplastic soil constitutive model capable of capturing the response of granular soils under low-frequency cyclic loading is introduced and verified. The model is piecewise linear with a hyperbolic stress-strain relationship. The size of the hysteresis loop is controlled using different scaling factors with a shift in the backbone curve at load reversal. The model introduces a new algorithm to better capture the soil's response upon reloading for plane strain. Model verification with experimental results at different scales shows that the model has good capabilities in capturing the response of granular soils under low frequency cyclic loading.

1. Introduction

In classical formulations of geotechnical problems (such as bearing capacity), the displacements to reach plastic equilibrium are not considered, and the changes in stresses and stiffness with deformation are not generally included. As such, when displacements of a geotechnical structure are of concern or when the relative stiffness of the soil with respect to surrounding structures controls design, it becomes necessary to mathematically formulate the soil response to changes in stress or strain through a soil constitutive model.

The mathematical formulation of the soil response requires understanding of the behavior and coupled response of stress changes and volumetric/pore pressure changes. Due to the degradation of the soil modulus with increasing stress/strain, the nonlinear response of the soil usually exhibits a hyperbolic relationship in the stress-strain space. The first formulation of a hyperbolic stress-strain curve for soils was based on Kondner's argument that soil response to drained triaxial loading in compression can be approximated by a hyperbola [1]. The first implementations of a hyperbolic response of soils were based on either stress dependent stiffness [2] or strain dependent shear modulus [5,6]. The formulations of a hyperbolic relationship utilized the small strain modulus; subsequent modifications recognized the dependency of the small strain modulus on the mean effective confining stress [7].

Formulation of a soil constitutive model that is capable of capturing the soil's response under any loading combination, strain rate with various drainage conditions, and boundaries is a challenging task. The challenge arises from the need to model the multiphase nature of the soil structure (solids, voids with water and/or air) in a continuum formulation [10,1].

In general, soil constitutive models fall under three categories: The first category is simple elastic-perfectly plastic models. In elastic-perfectly plastic models, the initial loading curve follows Hooke's law up to the yield stress. At yield, the soil obeys a yield criterion, e.g. Mohr-Coulomb. The advantage of simple elastic-perfectly plastic models is the limited number of required parameters (E, υ, ϕ, c). In these models, however, the stress path dependency of the soil response cannot be captured; the associated volumetric deformation with changes of stress and the dependence of the stiffness on stress level are not considered [10].

At the other end of the spectrum of soil constitutive models (the third category) are advanced models that accurately capture the soil behavior regardless of the stress path and couple volumetric change with changes of stress level and with changes in soil stiffness. The advanced constitutive models either implement bounding surface plasticity such as MIT-S1 [12] or Multi Yield Surface Plasticity models [18]. The disadvantages of such advanced models are the large number of parameters that require specialized testing and the difficulty in implementing such models in a numerical framework by practitioners, limiting their use to research applications. Pestana

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et al. [13] suggested that seven (7) tests ranging from hydrostatic compression, to triaxial, to resonant column tests were required to obtain the thirteen (13) parameters needed to capture Toyoura sand behavior with the MIT-S1 model.

As such, the need arises for relatively simpler soil constitutive models that are capable of capturing the soil response to specific loading combinations and for specified drainage and boundary conditions (the second category): Such models, however, may be only applicable to the particular conditions for which they are developed.

The extension of constitutive models to capture cyclic behavior adds to model complexity as the response of the soil to cyclic loading exhibits hysteretic behavior. The well-known Massing rules developed in 1926 and subsequent modifications have been implemented in many constitutive models to capture the response of soils to irregular cyclic loading such as those generated by earthquakes [15,17,8].

In this study, a simple elastoplastic constitutive model is proposed to capture the soil response to cyclic loading from the expansion and contraction of Integral Abutment Bridges (IAB); that is, under small to moderate strains. The intent of the model is to reasonably capture the response of granular soils while maintaining the number of parameters to a minimum under these specific loading conditions.

The proposed constitutive model is a modified version of what was proposed by Jung [8] to study the response of retaining walls to seismic loading. The modifications of the constitutive model were implemented based on the observed behavior of backfill and foundation soils of integral abutment bridges [9,4]. The modifications made to the Jung [8] model are: extension of the two dimensional model to three dimensions; rotation of the backbone curve to account for the increase in soil pressure from abutment movement with number of cycles; and capture of the soil response upon reloading. The model has been implemented in Abaqus® standard (UMAT) and explicit (VUMAT). Verification of the proposed constitutive model is performed using test results from element and physical model tests and a large-scale test of a laterally loaded pile. Verification and calibration of the model from large-scale tests of an integral abutment bridge and from a full scale instrumented bridge will be presented in a future paper.

2. Development of a model for IAB structures

The proposed model is developed as a piecewise linear rate independent model in the general elastoplasticity framework. The proposed model is isotropic with dependency of the shear modulus and small strain shear modulus on the octahedral shear strain and the mean effective stress, respectively. The strain increments are decomposed into elastic and plastic increments. The Drucker-Prager (D-P) yield criteria with unassociated flow rule is adopted to identify the state of stresses at which plastic strains evolve.

The relationship between stresses and strains is given based on the following equation (Eq. (1)):

$$d\sigma_{ij} = C^e_{ijkl} d\varepsilon^e_{kl} \tag{1}$$

where

$d\sigma_{ij}$ = incremental stress tensor,
C^e_{ijkl} = elastic modulus tensor, and
$d\epsilon^e_{ij}$ = incremental elastic strain tensor.

The elastic constants are written as shown in Eq. (2):

$$C^e_{ijkl} = \frac{E}{2(1+\mu)}\left(\delta_{il}\delta_{jk} + \delta_{ik}\delta_{jl}\right) + \frac{E\mu}{(1+\mu)(1-2\mu)}\delta_{ij}\delta_{kl} \tag{2}$$

$$K = \frac{E}{3(1-2\mu)}, \quad G = \frac{E}{2(1+\mu)} \tag{3}$$

where

E = Young's modulus,
μ = Poisson's ratio,
K = bulk modulus,
G = shear modulus, and
δ_{ij} = Kronecker delta and stands for 0 when $i \neq j$ and for 1 when $i = j$.

During elastoplastic response, the total strain increment is equal to the summation of the elastic strain increment and the plastic strain increment, as shown in Eq. (4):

$$\dot{\epsilon}^{total}_{ij} = \dot{\epsilon}^e_{ij} + \dot{\epsilon}^p_{ij} \tag{4}$$

where

$d\dot{\epsilon}^{total}_{ij}$ = incremental total strain tensor,
$d\dot{\epsilon}^e_{ij}$ = elastic strain tensor increment, and
$d\dot{\epsilon}^p_{ij}$ = the plastic strain tensor increment.

Hardin and Drnevich [5] identified several factors that control the degradation of the modulus of a granular soil during loading, namely strain amplitude, mean stress, void ratio, and others such as cementation. In addition, Hardin and Drnevich [6] proposed a modified hyperbolic stress-strain relation to better capture soil response which was achieved by distorting the strain scale by defining a hyperbolic relation for the strain as shown in Eqs. (5) and (6).

$$G = \frac{d\tau}{d\gamma} = G_o \frac{1}{(1+\gamma_h)^2} \tag{5}$$

Defining γ_h as:

$$\gamma_h = \frac{\gamma}{\gamma_r}[1 + a\exp(-b(\gamma/\gamma_r))] \tag{6}$$

Defining γ_r as:

$$\gamma_r = \frac{\tau_f}{G_0} \tag{7}$$

where

G_0 = small strain shear modulus,
γ = shear strain,
γ_r = reference shear strain,
a and b = fitting parameters, and
τ_f = shear stress at failure.

The small strain shear modulus dependency on the mean effective stress is given as follows by Eq. (8):

$$G_o = G^{ref}_o \left(\frac{\sigma'_m}{\sigma'_{m,ref}}\right)^{\alpha} \tag{8}$$

where

σ'_m = mean effective stress,
G^{ref}_o = reference small strain shear modulus,
$\sigma'_{m,ref}$ = reference effective mean stress, and
α = constant (material-dependent).

The small strain shear modulus can be measured either directly by means of geophysical methods (e.g. cross hole), laboratory testing (resonant column), or estimated from empirical correlations. For example, Hardin [7] proposed the following relation for granular soils (PI = 0) with $0.4 < e < 1.2$:

$$G^{ref}_o = 625 \frac{1}{0.3 + 0.7e^2}\sqrt{P_a \sigma'_{m,ref}} \tag{9}$$