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Investigating the cyclic behaviour of clays using a kinematic hardening soil model

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ABSTRACT

The stability of geotechnical structures under repeated loading depends to a large extent on the induced cyclic shearing stresses. The design of these structures usually requires engineers to employ advanced soil models in their analyses. While a number of such models do exist, their validation against cyclic laboratory tests is still very limited. In particular, the influence of the initial structure of the clay and its subsequent degradation under cyclic loading appears to be insufficiently investigated from both experimental and constitutive modelling standpoint.

The work outlined in this paper adds a new contribution to the theoretical understanding of cyclic response of clayey materials, presenting the extensive validation of an advanced kinematic hardening model against laboratory data on a number of natural and compacted clays found in literature. In order to analyse in detail the evolution of shear and hysteretic soil behaviour over a wide strain range, further modelling accounting for the effects of overconsolidation ratio and structure degradation is undertaken. The modelling results of shear stiffness degradation, hysteretic dissipation and pore pressure accumulation are presented and compared with experimental data. The results show that the enhanced kinematic hardening model gives very satisfactory predictions of clay response during cyclic loading.

1. Introduction

Saturated clayey deposits can be subjected to undrained cyclic loads by earthquakes, pile driving, traffic, explosions and storm waves. Under such repeated and irregular loading, clay structure deteriorates, pore water pressure changes and shear stiffness and strength degrade accordingly. Therefore, the behaviour of geotechnical structures and infrastructures interacting with clayey deposits is strongly influenced by the predictive capabilities of the constitutive model adopted for their design.

In particular, the characteristic features of the mechanical response of clays under cyclic loading, such as state dependency, early irreversibility, non-linearity, build-up of excess pore water pressures, decrease of nominal stiffness and related increase of hysteretic damping with cyclic shear strain, have been identified through extensive laboratory investigations on reconstituted samples by Sangrey et al. [1], Castro and Christian [2], Vucetic and Dobry [3], Yasuhara et al. [4], Matasović and Vucetic [5], Lee and Sheu [6] and Gu et al. [7], among others. More recently, there have been considerable advances in the experimental and constitutive modelling of natural soils to account for structure and its subsequent degradation under monotonic loading (e.g. Burland [8], Leroueil and Vaughan [9], Gens and Nova [10], Burland et al. [11], Cotecchia and Chandler [12], Callisto and Calabresi [13], Callisto and Rampello [14], Amorosi and Rampello [15]). Structure degradation under static loading has been shown to be critically important in reproducing the response of geotechnical systems interacting with clayey deposits, such as shallow foundations (e.g. Lagioia and Potts [16], Nova et al. [17]), earth embankments (e.g. Karstunen et al. [18], Panayides et al. [19]) and tunnels (e.g. Gonzáles et al. [20]). In contrast, only few contributions can be found in literature where the damage to structure caused by cyclic loading is accounted for in the analysis (e.g. Elia and Rouainia [21]). It is well-known that natural clays exhibit higher small-strain stiffness (G_0) than the corresponding reconstituted materials (Rampello et al. [22,23], Viggiani and Atkinson [24], Rampello and Viggiani [25], Cafaro and Cotecchia [26]). However, it appears that little research has been directed towards understanding the evolution of microstructure (destructuration) during cyclic loading and its effect on the normalised shear modulus (G/G_0) degradation curve, excess pore water pressure (Δu) and damping (D) curves with cyclic shear strain (γ). Only few laboratory data presenting a direct and consistent comparison between the cyclic/dynamic behaviour of natural and reconstituted samples of the same material are available in the literature. d'Onofrio et al. [27,28]

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Nomenclature

A, m, n	non-dimensional factors in Eq. (1)
b	normalised distance between bubble and structure surface
$b_{\max}$	maximum value of b
c_u	undrained shear strength
D	damping ratio
F	structure yield surface
f_r	reference yield surface
f_b	bubble yield surface
G_0	small-strain shear modulus
G_{sec}	secant shear modulus
H	plastic modulus
H_c	plastic modulus at conjugate stress
$\boldsymbol{I}$	second rank identity tensor
K	bulk modulus
l	non-dimensional factor relating G_0 to structure in Equation (1)
N	number of cycles
$\bar{\mathbf{n}}$	normalised stress gradient on the bubble
p, p_0	mean effective stress
p_c	stress variable controlling size of the surfaces
q	scalar deviator stress
R_0	isotropic overconsolidation ratio
r	parameter describing ratio of sizes of structure and reference surfaces
r_u	normalised cyclic pore pressure
$\mathbf{s}$	tensorial deviator stress
u, Δu	pore and excess pore pressure
v	specific volume
$\bar{\boldsymbol{\alpha}}$	location of the centre of the bubble
$\hat{\boldsymbol{\alpha}}$	location of the centre of the structure surface
γ, γ_c	cyclic shear strain
ε_a	axial strain
ε_v^p	plastic volumetric strain
ε_q^p	plastic deviatoric strain
ε_d	damage strain
μ	positive scalar of proportionality
$\boldsymbol{\sigma}$	effective stress tensor
$\boldsymbol{\sigma_c}$	conjugate stress
τ_c^*, τ_h^*	normalised cyclic shear stress

reported the results of an extensive testing programme on two natural Italian stiff clays using a resonant column/torsional shear device. In particular, the normalised shear modulus reduction curves obtained by testing a reconstituted (4BSR04) and a natural sample (1BSI04) of Bisaccia clay at the same mean confining pressure of 50 kPa are shown in Fig. 1a. It can be observed that the normalised shear stiffness of the natural sample degrades faster than the corresponding reconstituted material and the authors highlighted that the linear threshold strain of the natural specimens is always smaller than that of the reconstituted samples in all their experiments. Rampello and Silvestri [29] also presented experimental G/G_0 degradation curves for natural and reconstituted samples of overconsolidated Vallericca clay obtained during resonant column (RC) and torsional shear (TS) tests (see Fig. 1b). Allman and Atkinson [30] and Atkinson et al. [31] reported the secant shear modulus (G_{sec}) normalised with respect to the initial mean effective stress (p_0) for intact and reconstituted samples of Bothkennar clay obtained from undrained triaxial (TRX) tests (Fig. 1c). These experimental results showed that natural clays, although characterised by initial higher small-strain stiffness, exhibit smaller normalised shear stiffness than reconstituted soils due to damage to structure caused by the increased loading throughout the test.

In the simpler constitutive relations, the shear modulus degradation is indirectly obtained from a hyperbolic function that describes the backbone curve (e.g. Ramberg and Osgood [32], Iwan [33], Duncan and Chang [34]). Unloading-reloading behaviour is modelled by sets of rules, such as those proposed by Masing [35] or Pyke [36], which control the shape of hysteresis loops and, therefore, the damping of the soil. One of the limitations of these basic models is that they tend to overestimate the damping in the medium to large strain range and to under-predict it for small shear strains (Puzrin and Shiran [37]). Attempts have been made in recent years to improve the predictive capabilities of these simple hyperbolic models, trying to fit both the experimental normalised shear modulus and the damping ratio curve over the entire range of cyclic shear strain amplitudes (e.g. Darendeli [38], Phillips and Hashash [39]). Nevertheless, the stress-strain response predicted by these enhanced models is still decoupled from the generation of excess pore pressures, as the formulation is developed in terms of total stress. Alternatively, more advanced constitutive laws have been developed to capture the mechanical behaviour of cohesive materials within the framework of the work-hardening elasto-plasticity theory (e.g. Mroz et al. [40], Prevost [41] and Dafalias and Herrmann [42]). The presence of soil structure and its subsequent degradation under increased loading has been recently included into a number of elasto-plastic constitutive models, such as those proposed by Asaoka et al. [43], Rouainia and Muir Wood [44], Kavvadas and Amorosi [45], Baudet and Stallebrass [46] and Seidalinov and Taiebat [47]. These effective-stress-based models are able to describe the response of natural clays under both monotonic and cyclic loading conditions, accounting for the accumulation of plastic strains and shear induced excess pore water pressures with increasing number of cycles. Nevertheless, the essential aspect of the influence of soil structure on small-strain stiffness has not been incorporated in these models.

The aim of the paper is to investigate the performance of one of these non-linear constitutive models during cyclic shear simulations representing advanced dynamic laboratory tests on clays (such as RC and TS tests). The first part of the work introduces a modification of the well-known expression proposed by Viggiani and Atkinson [24] for the variation of G_0 with current state, which allows to independently account for the influence of mean effective stress, overconsolidation ratio and soil structure on shear stiffness. The new elasticity formulation has been implemented into an existing multi-surface kinematic hardening model, the Rouainia and Muir Wood (*RMW*) model [44], to enhance its predictive capabilities under static and cyclic loading conditions. The influence of model parameters and state variables on the *RMW* predictions of G-γ, D-γ and Δu-γ curves has been then investigated through an extensive parametric study. Finally, the modified constitutive model has been validated using experimental data on natural, reconstituted and compacted clays found in literature.

2. Soil constitutive model

The constitutive model adopted in this work has been formulated for natural clays by Rouainia and Muir Wood [44] within the framework of kinematic hardening with elements of bounding surface plasticity. This model converges to the Modified Cam-Clay model for remoulded structureless soils. It is characterised by