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Article

Influence of stress state on volumetric behaviour of soils

Influencia del estado de esfuerzos en el comportamiento volumétrico de los suelos

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Abstract

Soils are versatile yet difficult to predict because their strength and structure cannot be controlled unlike concrete or steel: their grains move freely, constantly modifying the microstructure and closing voids [which increase strength]. This behaviour is governed by hydro-mechanical coupling where compression reduces pore size, lowers hydraulic conductivity, and enhances shear strength. This research paper reproduces the nonlinear response of soils which evolve with each strain increment, strongly influenced by preconsolidation stress. Below this threshold, soils behave elastically and recover upon unloading; beyond it, particle displacement, friction, and energy dissipation occur. The transition from elastic to plastic behaviour is gradual, requiring constitutive models that capture both regimes. The model results are confronted with experimental data exhibiting good theoretical-experimental agreement.

Influence of stress state on volumetric behaviour of soils		
Objectives	**Methodology**	**Contribution**
• To use elastoplastic theory to address the behavior of soils subjected to isotropic loading. • To reproduce the nonlinear loading-unloading behavior of saturated soils using conventional laboratory parameters. • To compare the model predictions with reported results for soils subjected to hydromechanical stress paths.	A formulation based on elastoplastic theory is used for the hydromechanical stress path. An associated flow rule is used to reproduce the volumetric deformations. The formulation considers the connection between the elastic and elastoplastic states through a stiffness that evolves with the stress state, allowing the reproduction of the initial elastic states and subsequent elastoplastic deformations.	The model is confronted with experimental results for a residual gneiss, showing excellent agreement between theoretical and experimental data. The model allows for future research into its implementation in unsaturated soils undergoing wetting-drying cycles.

Isotropic consolidation, Saturated soil, Mechanical behaviour

Resumen

Los suelos son versátiles, pero difíciles de predecir ya que su resistencia y estructura no se pueden controlar a diferencia del hormigón o el acero: sus granos se mueven libremente, modificando constantemente la microestructura y cerrando huecos [lo que aumenta la resistencia]. Este comportamiento se rige por el acoplamiento hidromecánico, donde la compresión reduce el tamaño de los poros, disminuye la conductividad hidráulica y mejora la resistencia al corte. Este trabajo de investigación reproduce la respuesta no lineal de los suelos que evoluciona con cada incremento de la deformación fuertemente influenciada por la tensión de preconsolidación. Por debajo de este umbral, los suelos se comportan elásticamente y se recuperan al descargarse; por encima de él, se produce desplazamiento de partículas, fricción y disipación de energía. La transición del comportamiento elástico al plástico es gradual, lo que requiere modelos constitutivos que capturen ambos regímenes. Los resultados del modelo se comparan con datos experimentales que muestran una buena concordancia teórico-experimental.

Influencia del estado de esfuerzos en el comportamiento volumétrico de los suelos		
Objetivos	**Metodología**	**Contribución**
• Utilizar la teoría elastoplástica para abordar el comportamiento de suelos sujetos a carga isotrópica. • Reproducir el comportamiento no lineal de carga-descarga de los suelos saturados utilizando parámetros convencionales de laboratorio. • Comparar las predicciones del modelo con los resultados reportados de suelos sometidos a la trayectoria de tensión hidromecánica.	Se utiliza una formulación basada en la teoría elastoplástica para la trayectoria de la esfuerzos hidromecánica. Una regla de flujo asociada se utiliza para reproducir las deformaciones volumétricas. La formulación considera la conexión entre los estados elástico y elastoplástico mediante una rigidez que evoluciona con la evolución del estado de tensión, lo que permite reproducir los estados elásticos iniciales y posteriores deformaciones elastoplásticas.	El modelo se compara con resultados experimentales para un gneis residual, que muestran una excelente concordancia entre los datos teóricos y experimentales. El modelo permite considerar trabajos futuros para su implementación en suelos no saturados en ciclos de humectación-secado.

Consolidación isotrópica, Suelo saturado, Comportamiento mecánico

Area: Development of strategic leading-edge technologies and open innovation for social transformation

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Introduction

Soils are the most versatile building materials, but also the most difficult to predict. This is because, in their abundance, they exhibit a range of behaviours that are not easily foreseen [Rotta Loria et al., 2023]. The fundamental difference between soils and other building materials like concrete or steel is that their strength and internal structure cannot be controlled. In concrete, for example, we can modify its compressive strength by adjusting the proportions of its components. This procedure is not feasible for soils because it would be prohibitively expensive.

An additional consequence of the internal structure of soils is that their individual grains are in free movement. Therefore, when subjected to external stress, the particles must deform to balance and withstand this stress. This deformation, for example, is not possible in concrete because its individual grains are tightly bound together by the cement between them.

These differences lead to a constant modification of the soil's microstructure, a phenomenon that does not occur in other materials. This, in turn, results in the constant strength of these materials because their individual components close the empty spaces between them, thus increasing their strength due to the reduced space for volumetric deformations.

These ideas are framed within a concept that can be expressed as hydro-mechanical coupling of soils [Arroyo & Rojas, 2019; Della Vecchia & Romero, 2012] which means that the hydraulic properties of soils affect mechanical ones and vice versa. For instance, if a soil is compressed, the volume reduction affects the size of the pores that water can pass through, reducing its hydraulic conductivity. Moreover, this also has the consequence of increasing the shear strength because, due to the pore closure, individual particles are closer to each other having an overall increased strength [Arroyo et al., 2015; Zhang et al., 2020; Zolfaghari & Piri, 2017].

When the soil material is being stressed, the observed behaviour is nonlinear, which means that it cannot be predicted by simple linear rules. This is because of the aforementioned behaviour of particle moving thus producing a different material every strain suffered.

Shear resistance is not on the scope of this paper, however, it is very common to observe triaxial compression tests that receive large stresses producing a nonlinear behaviour up to failure [Weber et al., 2022; Wei et al., 2022].

When it comes to volumetric deformations, the friction between particles contribute to the strength that the bulk of the soil exhibit to external stresses. All soil materials appertain a certain mechanical feature called preconsolidation stress, which is actually the external stress that the internal consequences can sustain [Nuth & Laloui, 2008; Rojas, 2022]. When external stresses are smaller than the preconsolidation stress, the strain behaviour, when plotted in a cartesian axis, it recalls very much to a proportional relationship such as elastic materials [Amorosi et al., 2020; Della Vecchia & Romero, 2012]. When it reaches certain stress level, strains develop much faster and an elastic linear behaviour is no longer exhibited.

A specific feature of these ideas is that, if a soil material is unloaded before it reaches its preconsolidation stress, it recovers almost all its initial configuration. This is because, the particles have sustained themselves all the stress without needing any displacement of them relative to each other.

However, if preconsolidation stress is surpassed, the particles themselves cannot sustain the external stress and displacement of them is needed. Then, friction happens, and dissipation of energy is exhibited, sometimes even in the form of heat [Arroyo et al., 2015; Laloui, 2006].

This paper deals with this constitutive framework, dealing with the prediction of this initial linear and further nonlinear behaviour. However, it is very clear that this transition is not a sharp one. It is instead gradual and because of this, a connection with “later” plastic strains when elastic ones are exhibited for stresses smaller than the preconsolidation stress, is needed.

The constitutive model

The proposed constitutive model is elastoplastic in nature. The void ratios, e, will be used to express the volumetric changes.

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In an elastoplastic model, volumetric deformations can be partitioned into elastic and plastic:

$$e = e^e + e^p \qquad [1]$$

Elastoplastic theory allows us to relate volumetric plastic deformations $d\varepsilon_v^p = de/(1+e)$ to the yield surface f, where f generally takes the form of an ellipse [Dai et al., 2024; Mu et al., 2023; Muir Wood, 1990]. However, the present analysis is specifically restricted to deformations undergone in the isotropic domain and, therefore, is of exclusive interest what occurs under the application of isotropic stresses p.

In the case of triaxial compression, the value of the isotropic stress coincides with that of the confining stress applied to a triaxial compression chamber.In this sense, under the elastoplastic theory, it is possible to identify a proportionality between the volumetric deformations and the state of stress on the yield surface ∂f/∂p [Muir Wood, 1990], through a parameter L:

$$d\varepsilon_v^p = L\frac{\partial f}{\partial p} \qquad [2]$$

Replacing Equation [2] in Equation [1], the change in void ratios can be obtained:

$$de_v^p = d\varepsilon_v^p(1+e) = L\frac{\partial f}{\partial p}(1+e) \qquad [3]$$

If the consistency condition for plastic deformations is considered:

$$df = \frac{\partial f}{\partial p}dp + \frac{\partial f}{\partial q}dq + \frac{\partial f}{\partial p_0}dp_0 = 0 \qquad [4]$$

Where p_0 is the size of the yield function f, and p and q are the isotropic and deviator stress, respectively.

It is important to note that, classical elastopilastic models such as the Cam Clay [Roscoe & Burland, 1968], use also a plastic potential function g in order to define the magnitude of the plastic deformations related to the yield function f. In general, the plastic potential function is very much the same as the yield function in terms of its shape.

However, the aforementioned is expressed in terms of the plastic increments of strains [produced by deviator and isotropic strains]. However, if it is considered that the material will behave isotropically, it is enough to provide an associated mechanism of behavior between the yield function and the plastic potential function.

This is because they actually resemble each other when plotted in a cartesian space. However, this is not the case for anisotropic materials such as sedimentary deposits of natural soil materials nor the majority of the rock materials or highly consolidated soils.

However, this research paper works with the associated flow rule because the studied materials do work under that scheme. Because of this, the mathematical formulation is less complex and the implementation of the model equations is easier in any computational platform [Arroyo et al., 2024].

Hence, parameter L can be defined as:

$$L = \frac{\frac{\partial f}{\partial p}dp + \frac{\partial f}{\partial q}dq}{K} \qquad [5]$$

Where K establishes the stiffness of the material. Of course, K varies as the stress state progresses [Dai et al., 2024], and this parameter allows for a connection between the deformed and undeformed states of the soil.

Evolution of *K* with stress state

Figure 1 shows the initial stress state $\sigma = [p, q] = [p, 0]$ within the yielding surface f. In this research paper, Equation [6] is proposed for the model, which establishes a connection between future and present strain states through d_1 and d_2. It is in fact, a very common type of proposal [Jung & Yune, 2011; Russell & Khalili, 2004; Seidalinov & Taiebat, 2014; Wang et al., 2022]. Here, the idea is to produce a relationship for the plastic states and the elastic states, combining it to act in specific stress states.

Note that d_1 is the distance from the origin of coordinates, and d_2 is the distance from the same origin to a projected point at p_0. As the stress progresses and σ increases, the value of d_1 approaches d_2. Once $d_1 = d_2$, the stiffness value takes the usual value of the Modified Cam Clay Model [MCCM].

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Box 1

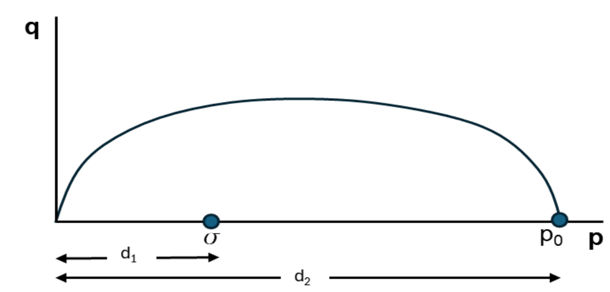

Figure 1
Schematic of yield function f for the model
Source: Own elaboration

$$K = -\left(\frac{\partial f}{\partial p_0}\right)^2 \frac{\partial p_0}{\partial \varepsilon_v^p}\left[1 + C\left(1 - \left(\frac{d_1}{d_2}\right)\right)\right] \quad [6]$$

Function $\frac{\partial p_0}{\partial \varepsilon_v^p}$ can be obtained from the proposal of Rojas et al. [2015], where:

$$\frac{dp_0}{d\varepsilon_v^p} = \frac{1+e}{e} p_0 \frac{1}{\lambda-\kappa} \quad [7]$$

A paramount feature of Equation [6], is that it restricts the volume deformation, inhibiting indefinite deformations thus providing a more real mechanical framework.

In Equation [7], λ and κ are soil compressibility parameters that can be easily obtained from a consolidation test in an oedometer or an isotropic consolidation test. Function $\frac{\partial f}{\partial p_0}$ may be obtained by elaborating a partial derivative for p_0. For this, the classic shape for f as that of the MCCM will be adopted.

Finally, it is needed to calibrate parameter C in Equation [5]. In Figure 2, the influence of C is depicted for a hypothetical soil material with an initial void ratio of 1.0.

Box 2

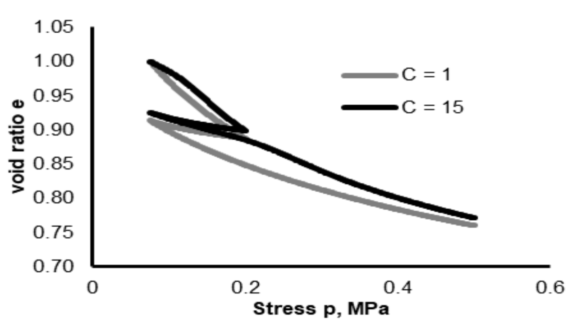

Figure 2
Influence of parameter C on soil stiffness
Source: Own elaboration

The material is stressed starting from 75 kPa to 200 kPa, then unloaded to the initial stress [75 kPa] and then re-loaded to 500 kPa. Both simulations show remarkable differences. For instance, it can be seen that the material that reaches the larger volume deformations is the one with $C = 1$, which is consistent since a smaller value produces a larger L, and thus larger volume strains.

It can be seen from Figure 2, that the larger differences are seen between 200 and 400 kPa, however, the model provides almost the same final void ratio at the end of the stresses path. This is in accordance with reported experimental data [Arroyo & Rojas, 2019], where it can be seen that the soil material slows down its deformation as it reaches larger stresses.

This is because, pore space will not indefinitely close unless individual particles break.

Theoretical-experimental result comparisons

The experimental results of a saturated soil subjected to isotropic compression will be compared with the predictions of the volumetric strain model.

This is a low-plasticity soil with a specific gravity of solids of 2.65. Its initial void ratio is 0.486 and its compressibility parameters are λ=0.167 and κ=0.03. The experimental programme is reported elsewhere [Gens, 1982].

The soil samples are fabricated departing from a slurry soil. It is placed within a triaxial cell to compress it isotropically. Afterwards, the materials excess pore water pressure is expelled by consolidating the material under isotropic stress. The goal for this previous experimental programme was to produce an isotropic material that would allow to discard the anisotropic conditions and furtherly evaluate the mechanical behaviour of them both [anisotropic and isotropic].

This research paper's goal is to analyse the isotropic behaviour material. Further research is oriented towards the mechanical behaviour of anisotropic granular material under unsaturated and saturated conditions.

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Box 3

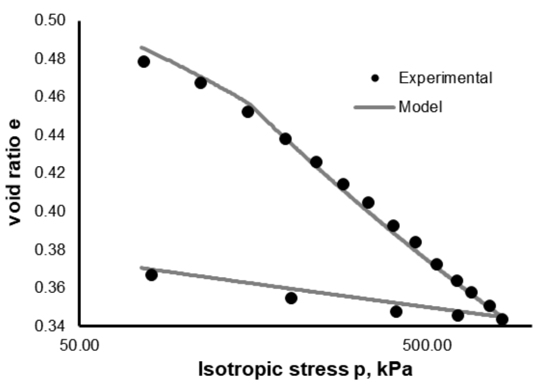

Figure 3

Numerical-experimental result comparisons for the volumetric behavior of a saturated soil subjected to isotropic compression.

Source: Own elaboration

As can be noted from Figure 3, the initial void ratio is 0.48 and the preconsolidation stress is 200 kPa. The simulations start stressing the material from the starting point of 75 kPa and goes further than the preconsolidation stress up to 823 kPa ad then unloaded up to 75 kPa again.

Because of the features of the model used, the initial elastic zone [from 75 to 200 kPa] shows different slope than that of the unloading ranghe [827 to 75 kPa], even though the same elastic slope parameter is being used [$\kappa = 0.03$]. This are the most distinctive features when compared to classic elastoplastic models.

Here, the reason for this behavior, is that the elastic inner zone, within the yielding surface f, is connected to the plastic zone through the distances d_1 and d_2, providing an elastoplastic behavior even at the stress range that would be identified as purely elastic [Chen et al., 2019].

Conclusions

A constitutive elastoplastic model has been proposed. The model is formulated under the elastoplastic theory providing a yielding function that works that uses a plastic flow rule, the consistency condition

The model distinctive feature is the formulation of the stiffness parameter K, allowing a connection between elasti and plastic zones. The connection is provided by a nonlinear relationship that is evaluated determining the distance between a projection center at the origin or the stress space, and the yielding surface f.

Because of this, a soft transition between elastic and plastic states is observed for the numerical purposes. A drawback of the present model is the need for calibration of parameter C. This is because this parameter can be variated between a narrow range but still has strong influence on the predicted stresses.

The model is confronted with experimental results reported elsewhere for a residual gneiss showing excellent agreement between the theoretical and the experimental data.

Future work is oriented towards the implementation of the present model for unsaturated soils allowing the prediction and inclusion of wetting-drying cycles on the overall strength and volume behavior of the soil materials.

Declarations

Conflict of interest

The authors declare no interest conflict. They have no known competing financial interests or personal relationships that could have appeared to influence the article reported in this article.

Author contribution

Arroyo, Hiram: Contributed to data analysis and interpretation, manuscript writing and revision.

Palos-Barba, Viviana: Contributed to the project idea, research method, technique and manuscript revision.

Chávez-Cárdenas, Xavier: Contributed to data analysis.

Availability of data and materials

The datasets used or analyzed during the current study are available from the corresponding author upon reasonable request.

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Abbreviations

MCCM	Modified Cam Clay Model

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