Obtaining the Mechanical Parameters for the Hardening Soil Model of Tropical Soils in the City of Brasilia

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Abstract. In this article, the mechanical parameters of characteristic soils of the city of Brasilia are obtained, calibrated and validated for the Hardening Soil (HS) model, based on laboratory and field test results obtained in previous research studies conducted in the Experimental Field of the University of Brasilia (CEGUnB). The strength and compressibility parameters are obtained from triaxial CU tests (with isotropic and anisotropic consolidation) and one-dimensional consolidation tests, respectively. The obtained parameters are calibrated via explicit numerical modeling using the finite element method and the SoilTest module of Plaxis software. After the parameters are evaluated and calibrated, a geotechnical model characterizing the city of Brasilia for HS is proposed. Finally, this geotechnical model is validated through the numerical modeling of load testing on footings and piles conducted at the CEGUnB. It is concluded that the mechanical behavior of the Brasilia soils under natural moisture conditions can be modeled using the HS model.

Keywords: finite element method, Hardening Soil model, tropical soils, model validation and calibration, load testing.

1. Introduction

The tropical soils of the city of Brasilia have been extensively studied in the Postgraduate Program in Geotechnics of the University of Brasilia. A significant number of theses and dissertations have been developed on these soils, focusing on the investigation of their physical-chemical, mineralogical, structural, mechanical and hydraulic properties as well as the behavior of shallow and deep foundations. The use of numerical tools for research related to these soils is becoming increasingly common, including Plaxis software, among others. Plaxis is a more versatile analysis tool than other commercial programs for the analysis of practical problems and is increasingly employed by geotechnical companies all over the world. One of the most complete constitutive models of Plaxis is the Hardening Soil (HS) model (Schanz et al., 1999; Brinkgreve et al., 2014, 2015), which is capable of:

1) calculating the total strains using a stress-dependent stiffness that is different for loading and unloading/re-loading conditions;

2) modeling irreversible strains due to primary deviatoric loading (shear hardening); and modeling irreversible plastic strains due to primary compression under oedometric and isotropic loading (compression hardening).

Given this context, the objective of this study is to obtain, adjust and validate the mechanical parameters of characteristic soils of the city of Brasilia for the HS model, making use of laboratory and field test results obtained in previous research studies conducted in the Experimental Field of the Graduate Program in Geotechnics of the University of Brasilia, and additionally to present a validation methodology that can be applied to any other soil type.

The methodology proposed herein begins with the evaluation of the strength and compressibility parameters of triaxial CU tests (with isotropic and anisotropic consolidation) and one-dimensional consolidation tests, respectively (Guimarães, 2002). Then, the parameters obtained for the HS model are calibrated through the explicit numerical modeling of the tests using the finite element method (FEM) and the SoilTest module of Plaxis software. Based on the evaluation and calibration of these parameters, a geotechnical model profile of the Experimental Field of the University of Brasilia (CEGUnB) is proposed for the HS model. The profile is composed of characteristic soils of the city of Brasilia: a deeply weathered soil mantle composed of lateritic soil, followed by a thin layer of transitional soil that overlaps the poorly weathered residual saprolite soil.

Finally, this geotechnical model is validated through the numerical modeling of the load testing of footings and piles conducted in the CEGUnB (Sales, 2000; Guimarães, 2002).

2. General Description of the Problem

2.1. General characteristics of the Federal District subsoil

The Federal District (DF) is located in the Central Plateau in the Center-West region of Brazil and is home to the city of Brasilia, which is the federal capital of the country. The DF region is covered by a mantle of Tertiary-Quaternary detritus-lateritic soil composed mainly of red-yellow latosols, according to the Brazilian soil classifica-
The thickness of this cover is quite diverse depending on the topography, vegetation and original rock and can range from centimeters to tens of meters. High degrees of weathering and leaching were responsible for the formation of this soil, which led to the development of a very porous, metastable aggregate structure with a large proportion of voids and, consequently, low density, called "porous clay" by local geotechnicians. Due to its aggregate state and metastable structure, this clay has a low penetration resistance standard ($N_{\text{SPT}}$ from 1 to 6 strokes; stable meta structure) and high permeability (from $10^{-3}$ to $10^{-4}$ m/s; particles in an aggregate state), similar to that of fine granular soils, which, incidentally, is how its texture is presented in its natural state. Due to its high porosity and cementitious bond type, it has a highly unstable structure when subjected to increased moisture and/or changes in the stress state, almost always presenting a volume variation as high as the variation of these factors (referred to as a collapsible structure).

According to Ortigão & Macedo (1993), in the city of Brasília, along the pathway designed for the subway (Asa Sul neighborhood), it was found that the porous clay has a variable thickness ranging from 20 to 30 m, generally with a deep groundwater level, in some cases at 5.0 m depth, as occurs at the end of Asa Sul. The end of the porous clay layer is clearly identified in percussion drillings by the significant increase of $N_{\text{SPT}}$ at the transition, followed by contact with the underlying saprolite soil.

For this study, the stratigraphy of the Experimental Field of the Postgraduate Program in Geotechnics of the University of Brasília (CEGUnB) was considered representative of the city of Brasília. This Program has valuable geotechnical information obtained from surveys, field trials, laboratory tests and loading tests on superficial and deep foundations (Perez, 1997; Jardim, 1998; Sales, 2000; Guimarães, 2002; Mota, 2003; Coelho, 2013; Sales et al., 2015). According to this information and the tropical soil profiles proposed by Cruz (1987) and Cardoso (2002), it was possible to define the typical stratigraphic profile of the CEGUnB, as shown in Fig. 2.

### 2.2. Soil properties characteristic of the CEGUnB

The mechanical parameters for the HS model of the tropical soils of the CEGUnB were obtained through laboratory tests conducted by Guimarães (2002). Characterization, shear strength and compressibility tests were performed on undisturbed samples obtained at each meter depth in two open pit wells excavated up to eight and ten meters deep. Table 1 presents a summary with some of the
index properties of the most representative layers of the CEGUnB, and it should be noted that the particle size percentages were obtained by using the sodium hexametaphosphate deflocculant for the fraction passing through a no. 10 sieve.

One-dimensional consolidation tests were conducted to obtain the compressibility and soil collapse parameters of the CEGUnB. For each collected sample, the following were performed: a “conventional” consolidation test, according to the Brazilian standard, and a “simple” consolidation test, according to the procedure recommended by Jennings and Knight (1975). In the “conventional” test, the samples were saturated after the first loading (5 kPa) and loaded until reaching a stress of 800 kPa. In the “simple” test, the sample was loaded with a natural moisture content until it reached a stress of 200 kPa; the specimen was then saturated and loaded until reaching a stress of 800 kPa. Fig. 3 shows the compressibility curves for the undisturbed sample obtained at 2 m depth.

In addition to the compressibility tests, consolidated undrained (CU) triaxial tests were performed. The test specimens were consolidated both isotropically and anisotropically following the $K_0$ (lateral earth pressure at rest) path. Both tests were performed with natural moisture (unsaturated) and saturated samples. The results of these tests are presented in section 5.2, with the validation of the constitutive model.

In the case of this study, the soil properties were analyzed exclusively in natural moisture conditions; therefore, only “simple” consolidation tests and triaxial tests with unsaturated test specimens were used.

### 2.3. Load tests conducted at the CEGUnB

The stratigraphic model and the mechanical parameters obtained for the HS model were validated through the simulation of load tests on piles and footings performed at the CEGUnB by Guimarães (2002) and Sales (2000), respectively.

Guimarães (2002) conducted five load tests on piles mechanically excavated at the site (0.3 m in diameter and 7.25 to 7.85 m in length). Table 2 presents the characteristics of the piles and the results obtained for each test, and Fig. 4 presents the load vs. displacement curves.

Sales (2000) performed a load test on a single concrete footing at the CEGUnB for natural moisture and saturated conditions. The footing (1 $\times$ 1 m$^2$ concrete plate, 15 cm thick) was built at the bottom of a square pit 80 cm deep. Figure 5 shows the results of the load tests under natural moisture and porosity conditions.

### 3. Hardening Soil Model

Soil constitutive models have advanced significantly from basic models that idealize the soil as a linear elastic medium or a perfectly plastic linear elastic medium. The Hardening Soil (HS) model is implemented in Plaxis soft-

![Figure 3 - Compressibility curves obtained from conventional and simple consolidation tests at a depth of 2 m (Guimarães, 2002).](image)

#### Table 2 - Pile characteristics and load test results (Guimarães, 2002).

<table>
<thead>
<tr>
<th>Pile #</th>
<th>Date</th>
<th>Length (m)</th>
<th>Maximum applied load (kN)</th>
<th>Maximum displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>02/2000</td>
<td>7.65</td>
<td>270</td>
<td>16.10</td>
</tr>
<tr>
<td>2</td>
<td>08/2000</td>
<td>7.25</td>
<td>300</td>
<td>3.82</td>
</tr>
<tr>
<td>3</td>
<td>10/2000</td>
<td>7.80</td>
<td>240</td>
<td>8.71</td>
</tr>
<tr>
<td>4</td>
<td>03/2001</td>
<td>7.30</td>
<td>210</td>
<td>6.82</td>
</tr>
<tr>
<td>5</td>
<td>06/2000</td>
<td>7.85</td>
<td>270</td>
<td>9.42</td>
</tr>
</tbody>
</table>

#### Table 1 - Index properties of the characteristic layers of the CEGUnB.

<table>
<thead>
<tr>
<th>Layer</th>
<th>$G_s$ (kN/m$^3$)</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$\gamma_{sat}$ (kN/m$^3$)</th>
<th>$e$</th>
<th>$n$ (%)</th>
<th>$G$ (%)</th>
<th>$S$ (%)</th>
<th>$M$ (%)</th>
<th>$C$ (%)</th>
<th>$S_{sat}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2.65</td>
<td>14.2</td>
<td>16.9</td>
<td>1.4</td>
<td>58</td>
<td>0.7</td>
<td>38.0</td>
<td>26.5</td>
<td>44.3</td>
<td>34.8</td>
</tr>
<tr>
<td>B</td>
<td>2.63</td>
<td>15.9</td>
<td>18.0</td>
<td>1.0</td>
<td>51</td>
<td>3.3</td>
<td>27.4</td>
<td>25.0</td>
<td>44.3</td>
<td>51</td>
</tr>
<tr>
<td>C</td>
<td>2.74</td>
<td>17.7</td>
<td>18.6</td>
<td>1.0</td>
<td>50</td>
<td>0.3</td>
<td>6.8</td>
<td>86.8</td>
<td>6.1</td>
<td>82</td>
</tr>
</tbody>
</table>

A: porous sandy clay, B: residual lateritic soil, C: saprolitic soil, $G_s$: density of solids, $\gamma$: apparent specific weight of moist soil, $\gamma_{sat}$: saturated specific gravity, $e$: void ratio, $n$: porosity, $G$: percentage of gravel, $S$: percentage of sand $M$: percentage of silt, $C$: percentage of clay $S_{sat}$: degree of saturation under natural conditions.
ware and is based on the theory of plasticity. Its main characteristics are described as follows:

1) The total strains are calculated using a stress-dependent stiffness.
2) The stiffness is defined for both loading and unloading/reloading conditions.
3) Modeling of irreversible strains due to primary deviatoric loading (shear hardening).
4) Modeling of irreversible plastic strains due to primary compression under oedometric and isotropic loading (compression hardening).
5) A non-associated flow rule is assumed for shear hardening, and an associated flow rule is assumed for compression hardening.
6) The Mohr-Coulomb failure criterion is applied.

In the HS model, the stress-strain relationship \( (q - \varepsilon) \) due to the primary load is hyperbolic (Kondner, 1963; Duncan & Chang, 1970) for a drained triaxial test (Fig. 6).

We then have:

\[
\varepsilon_1 = \frac{1}{E_i} \frac{q}{1 - \frac{q}{q_a}}, \text{ for } q < q_f
\]  

where the initial stiffness \( E_i \) is related to \( E_{50} \) by:

\[
E_i = \frac{2E_{50}}{2 - R_f}
\]  

where \( \varepsilon_i \) is the axial strain, \( q \) is the deviatoric stress, and \( q_a \) is the asymptote of the shear strength:

\[
q_a = \frac{q_f}{R_f}
\]  

where \( R_f \) is the failure ratio (0.9 by default in the software) and \( q_f \) is the ultimate deviatoric stress, defined by the Mohr-Coulomb criterion:

\[
q_f = \frac{2 \sin \phi '}{1 - \sin \phi} (\sigma'_s + c' \cot \phi ')
\]  

where \( c' \) and \( \phi ' \) are the effective shear strength parameters, \( \sigma'_s \) is the confining stress in the triaxial test (in the software, \( \sigma'_s \) is negative in compression), and \( E_{50} \) is the confining stress-dependent stiffness modulus for the primary load, defined as follows:

\[
E_{50} = E_{50}^{ref} \left( \frac{c' \cos \phi ' + \sigma'_s \sin \phi '}{c' \cos \phi ' + p^{ref} \sin \phi '} \right)^m
\]  

where \( E_{50}^{ref} \) is the reference secant stiffness modulus for the drained triaxial test, \( p^{ref} \) is the reference isotropic stress (100 kPa by default in the software), and \( m \) is the exponent that defines the strain dependence value of the stress state. In natural soil, the exponent \( m \) varies between 0.3 and 1.0. As suggested by Brinkgreve et al. (2014), to simulate a log-logarithmic compression behavior, as observed in soft clays, \( m \) should be taken equal to 1.0. As noted by Obrzud & Truty (2018) and by Brinkgreve et al. (2014), Janbu (1963) reported values of \( m = 0.5 \) for Norwegian sands and silts, Kempfert (2006) provided values between 0.38 and 0.84 for soft lacustrine clays and von Soos (1990) reported various different values in the range \( 0.5 < m < 1.0 \).

The confining stress-dependent stiffness modulus for unloading and reloading conditions is defined as:

\[
E_{ur} = E_{ur}^{ref} \left( \frac{c' \cos \phi ' + \sigma'_s \sin \phi '}{c' \cos \phi ' + p^{ref} \sin \phi '} \right)^m
\]
where $E_{ur}^{ref}$ is the reference stiffness modulus for unloading and reloading conditions ($E_{ur}^{ref} = 3E_{50}^{ref}$ by default in the software).

In the HS model, the elastic region is limited by two yield functions (Fig. 7): the shear hardening yield function ($f_s$) and the cap compression hardening yield function ($f^c$). The first is defined as:

$$f_s = f - \gamma^p$$  \hspace{1cm} (7)

where

$$f = \frac{2}{E_{ur}} \left( \frac{q}{q_a} - \frac{2q}{E_{ur}} \right)$$  \hspace{1cm} (8)

The plastic shear strain ($\gamma^p$) is given by:

$$\gamma^p = 2\varepsilon_i^p - \varepsilon_i^p \approx 2\varepsilon_i^p$$  \hspace{1cm} (9)

where $\varepsilon_i^p$ is the plastic axial strain and $\varepsilon_i^p$ is the plastic volumetric strain.

The cap compression hardening yield function is given by Fig. 8.

$$f^c = \frac{\tilde{q}}{\alpha^2} + p^2 - p_p^2$$  \hspace{1cm} (10)

where $\alpha$ is an auxiliary parameter of the model related to $K_o^w$ ($K_o^w = 1 - \sin \phi'$, by default in the software), $p$ is the isotropic stress, $p_p$ is the preconsolidation isotropic stress, and $\tilde{q}$ is the special stress measurement for deviatoric stresses:

$$\tilde{q} = \sigma_z' + (\delta - 1)\sigma_2' - \sigma_3'$$  \hspace{1cm} (11)

where

$$\delta = \frac{3 + \sin \phi'}{3 - \sin \phi'}$$  \hspace{1cm} (12)

For triaxial compression ($\sigma_2' = \sigma_3'$), $\tilde{q} = \sigma_1' - \sigma_3'$, and for triaxial extension ($\sigma_1' = \sigma_3'$), $\tilde{q} = \delta(\sigma_1' - \sigma_3')$.

The volumetric plastic strains in isotropic compression ($\varepsilon_v^{pc}$) are obtained as follows:

$$\varepsilon_v^{pc} = \frac{\beta}{1 - m} \left( \frac{p_p}{p_p^{ref}} \right)^{1-m}$$  \hspace{1cm} (13)

where $\beta$ is an auxiliary parameter of the model related to the reference tangent stiffness modulus for oedometric loading $E_{oed}^{ref} (E_{oed}^{ref} = 1.25E_{50}^{ref})$ by default in the software). Similar to the triaxial moduli, the axial stress-dependent stiffness modulus ($E_{oed}$) for primary oedometric loading ($\sigma_1'$) is obtained as follows:

$$E_{oed} = E_{oed}^{ref} \left( \frac{c' \cos \phi' + \sigma_1' \sin \phi'}{c' \cos \phi' + p_p^{ref} \sin \phi'} \right)^m$$  \hspace{1cm} (14)

4. Evaluation of the Compressibility and Strength Parameters

4.1. Information obtained from consolidation tests

The information was obtained from a total of six “simple” one-dimensional consolidation tests (with natural moisture up to a pressure of 200 kPa under which they were saturated). Table 3 shows the calculated values of the reference oedometric moduli ($E_{oed}^{ref}, E_{ur, oed}^{ref}$) and the parameter that defines the dependency level of the strains on the stress state ($m$). As suggested by Surarak et al. (2012), for Bang-

![Figure 7 - Possible stress paths and yield and failure surfaces for the HS model.](image)

![Figure 8 - Compression hardening yield surface on the plane (modified from Schanz et al., 1999).](image)
The parameter $m$ and moduli $E_{oed}^{ref}$ and $E_{u, oed}^{ref}$ were obtained as follows:

1) The tangent stiffness moduli $E_{oed}$ and $E_{u, oed}$ for several vertical stress values $\sigma'$ were determined as indicated in Fig. 9;

2) As shown in Fig. 10, the $E_{oed}/p^{ref}$ and $E_{u, oed}/p^{ref}$ normalized moduli were plotted vs. the $\sigma'/p^{ref}$ normalized stress on a double logarithmic graph considering a $p^{ref}$ value of 100 kPa (any value can be used as a reference, but the authors decided to use the value proposed by the software manual);

3) Finally, the values of the $E_{oed}^{ref}$ and $E_{u, oed}^{ref}$ moduli were found for $\sigma'/p^{ref}$ (stiffness moduli for the reference isotropic stress value). Because the exponent $m$ (Eq. 14) represents the amount of stress dependency, to simulate the logarithmic compression behavior of the soil, the $m$ values were obtained from the slopes of the double logarithmic trend lines of the graphs of Fig. 10.

As shown in Table 3, the $E_{oed}^{ref}$ modulus values for depths of 1 to 6 m vary from 1.25 to 10 MPa, with a mean value of 4.7 MPa, while from depths of 8 to 10 m, the mean value is approximately 6.6 MPa, almost 1.4 times greater than the mean superficial value. In contrast, for the $E_{u, oed}^{ref}$ modulus, the mean value from depths of 8 to 10 m is just 8.7 MPa, while the superficial obtained mean value is approximately 29.5 MPa, 3.4 times greater than the mean deep value. For the $E_{u, oed}^{ref}/E_{oed}^{ref}$ ratio, it can be seen that the superficial soils (porous sandy clay) show considerably higher values (up to 15.82) than the deepest ones (up to 1.18). This may be because superficial soils may collapse during primary loading, inducing major changes in the soil response for unloading conditions. On the other hand, the average obtained value of the exponent $m$ for primary loading is approximately 0.3, near the values obtained by Jambu et al.
(1963) for Norwegian sands and silts \((m = 0.5)\). This is not the case for the average value of the exponent \(m\) for unloading conditions, where the obtained mean value is approximately 1.3, closer to the behavior of a normally consolidated clay \((m = 1)\). The results reported by Surarak et al. (2012), for Bangkok stiff clays, show values of the exponent \(m\) for primary loading from 0.5 to 0.7 and from 1.0 to 1.2 for unloading conditions. Additionally, studies developed by Kempfert (2006) in three lacustrine soft soils demonstrated that the exponent \(m\) can be greater for unloading conditions than for primary loading.

4.2. Information obtained from triaxial tests

The CIU triaxial tests (consolidated under isotropic conditions and failure under undrained conditions) were conducted at depths of 2, 4 and 6 m, and the CK0U triaxial tests (consolidated under \(K_s\) anisotropic conditions and failure under undrained conditions) were conducted at depths of 8 and 10 m in undisturbed samples under natural moisture conditions. The shear strength parameters \((c'\) and \(\phi')\), the reference modulus at 50% strength \((E_{50}^{ref})\), and the modulus \(m\) were calculated; Table 4 summarizes the obtained values. The parameter \(m\) and modulus \(E_{50}^{ref}\) were obtained as follows:

1) The secant stiffness modulus \(E_{50}\) for each triaxial deviatory stress \((\sigma_1' - \sigma_3')\) vs. axial strain \((\varepsilon_1)\) curve of each confining stress pressure \((\sigma_3')\) was determined.
2) As shown in Fig. 11, the \(E_{50}^{ref}/p^{ref}\) normalized modulus was plotted vs. the \(\sigma_1'/p^{ref}\) normalized confining stress on a double logarithmic graph considering a \(p^{ref}\) value of 100 kPa; 3) Finally, the values of the \(E_{50}^{ref}\) modulus were found for \(\sigma_1'/p^{ref}\) (stiffness modulus for the reference isotropic stress value). Because the exponent \(m\) (Eq. 5) represents the amount of stress dependency, to simulate the logarithmic behavior of the soil, the \(m\) values were obtained from the slopes of the double logarithmic trend lines of the graphs of Fig. 11.

Unfortunately, some \(\sigma_1'/\sigma_3'\) vs. \(\varepsilon_1\) curves had to be discarded because they were inconsistent with the other results, so for the depths of 4, 8 and 10 m, only two points were plotted, and for 6 m, all points were discarded.

Table 4 shows that for superficial soils (2 and 4 m depths), the mean \(E_{50}^{ref}\) modulus is approximately 2 MPa, while for the deeper soils (8 and 10 m depths) the mean value is 28.3 MPa, 14.3 times greater. Additionally, for superficial soils, the obtained mean \(E_{50}^{ref}/E_{50}^{ref}\) ratio is ap-

![Figure 11 - Variation in \(E_{50}\) with confining pressure.](image-url)
approximately 1.17, close to the default value of 1.25 proposed by the software; this is not the case for the deeper soils, where the mean $E_{50}^{\text{ref}} / E_{\text{oed}}^{\text{ref}}$ value is approximately 4.3, very far from the proposed value. The high $E_{50}^{\text{ref}}$ moduli for the 8 and 10 m depths are because, despite the undrained condition of the tests, the large air volume present in the macropores of the superficial soils ($S_r < 50\%$) prevents the generation of a significant pore pressure, thus generating a behavior closer to the drained condition. However, at greater depths, the porosity decreases, and the degree of saturation considerably increases ($S_r > 80\%$); therefore, the porosity values generated are more relevant, and the stiffness of the material is closer to that of the undrained condition.

5. Calibration of the Parameters Obtained for the HS Model

5.1. Numerical modeling through FEM

To obtain the best representation of the stress-strain curves and the stress paths, once the compressibility and strength parameters were evaluated from the laboratory tests, it was considered important to simulate those tests using the HS constitutive model to see if it was necessary to make adjustments to those parameters. Mainly, two techniques can be used for this simulation to calibrate the initially obtained parameters: one is the explicit simulation of the test by finite element software, and the other is the SoilTest module of Plaxis software (Brinkgreve et al., 2014). To show the use of both techniques, in this work, the modeling of the triaxial test was carried out considering the explicit numerical modeling and the one-dimensional consolidation tests using the SoilTest module.

The numerical modeling of the triaxial tests was performed considering the axisymmetric geometry of the problem. Figure 12 shows the developed finite element mesh and the boundary conditions considered. Because the tests were performed for soils under natural moisture conditions ($S_{\text{m}} = 34$ to $84\%$), in the model, the test type was considered as drained (CD), and therefore the development of a positive pore pressure was not allowed.

For the tests corresponding to depths of 2, 4 and 6 m, the initial stage was simulated considering isotropic loading conditions (distributed load system $A = B = \sigma'$, Fig. 12), whereas for the depths of 8 and 10 m, the loading was anisotropic ($B$ equal to $K_A$, Fig. 12); the considered stress values are shown in Table 5. The failure was generated under drained conditions by increasing the value of the distributed load $A$ that, in this step of the test, represents the deviatoric stress ($A = \sigma'_1 - \sigma'_3$, Fig. 12).

5.2. Calibration of the obtained parameters

The compressibility parameters obtained from the consolidation tests (Table 3) and the stiffness and strength parameters obtained from the triaxial tests (Table 4) were used for the calibration of the HS model for the soils of the CEGUaB.

To obtain the best representation of the stress-strain curves and the stress paths of the laboratory tests, the following parameter-adjusting criteria were adopted:

a) The $c', \phi'$ and $E_{50}^{\text{ref}}$ parameters obtained from laboratory tests (Tables 3 and 4) were used as initial values and were kept (as much as possible) without major changes during the calibration process;

b) The modulus $E_{50}^{\text{ref}}$ was defined equal to $E_{50,\text{oed}}^{\text{ref}}$ (Table 3) as the initial value and was considered one of the main parameters of adjustment during the calibration process;

c) The modulus $E_{50}^{\text{ref}}$ obtained from laboratory tests (Table 4) was defined as the initial value and was considered one of the main parameters of adjustment during the calibration process;

d) An initial value of $m = 0.5$ was considered (sand behavior) and was kept (as much as possible) without major changes during the calibration process;
e) An initial value of $R_f = 0.9$ was considered (default setting) and was kept (as much as possible) without major changes during the calibration process;
f) The default settings for the parameters $\psi$, $\nu_{ur}$ and $K_{nc0}^\infty$ were considered without changes during the calibration process.

Table 6 shows the parameters that best fit the deviatoric stress vs. axial strain, stress path ($p$ vs. $q$) and one-dimensional compressibility (axial strain vs. vertical stress). As part of the obtained results, Figs. 13 and 14 shows the adjustment curves of the CIU and consolidation tests for a depth of 2 m and for the CK0U and consolidation tests for the depth of 10 m, respectively.

In general, good agreement is observed between the laboratory results and the HS model, but it is important to identify and analyze the major differences observed. However, for illustration purposes, only the CIU test for the 2 m depth and the CK0U test at a 10 m depth are presented and discussed:

a) In the $\varepsilon$ vs. $q$ curves (Figs. 13a and 14a), the material develops softening during failure, especially for the CK0U tests at 8 and 10 m in depth, and this behavior cannot be simulated with the HS model.

b) The strain by material collapse during saturation under a stress of 200 kPa (Fig. 13b) was not simulated; however, the predictions of the model before saturation and during unloading show strong correlations.

c) There is a strong correlation between the strains obtained in the $\varepsilon$ vs. $q$ diagrams for the 2, 4 and 6 m depth tests (Fig. 13a), however, for the CK0U tests at 8 and 10 m depth (Fig. 14a), there is a noticeable deviation especially for the $\varepsilon$ vs. $q$ diagrams.

Table 6 - Parameters obtained for the HS model that best fit the laboratory tests.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Type</th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ (°)</th>
<th>$\psi$ (°)</th>
<th>$E_{vo}^\infty$ (MPa)</th>
<th>$E_{vo}^{ref}$ (MPa)</th>
<th>$E_{vo}^{ref}$ (MPa)</th>
<th>$m$</th>
<th>$\nu_{ur}$</th>
<th>$K_{nc0}^\infty$</th>
<th>$R_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>CIU</td>
<td>0</td>
<td>25</td>
<td>0</td>
<td>3.2</td>
<td>4.9</td>
<td>14.0</td>
<td>0.5</td>
<td>0.2</td>
<td>0.58</td>
<td>0.8</td>
</tr>
<tr>
<td>2</td>
<td>CIU</td>
<td>0</td>
<td>25</td>
<td>0</td>
<td>2.5</td>
<td>1.45</td>
<td>14.0</td>
<td>0.5</td>
<td>0.2</td>
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$^{a}$parameters adjusted from the results of the 2 m triaxial test and the 1 m consolidation test, $\psi$ = dilatancy angle, considered = 0 (default setting).
$\nu_{ur}$ = unloading/reloading Poisson’s ratio = 0.2 (default setting).
$K_{nc0}^\infty$ = coefficient of earth pressure at rest for normal consolidation = 1-sin$\phi'$ (default setting).

Figure 13 - Laboratory results and adjustment curves obtained with the HS model for the CIU test and for the 2 m depth consolidation. (a) Axial strain vs. deviatoric stress ($\varepsilon$ vs. $q$); (b) Compressibility curve ($\varepsilon$ vs. $\sigma_3'$); (c) Stress path ($p$ vs. $q$).
depths (Fig. 14a), the displacement obtained by the HS model is considerably greater than that of the triaxial tests. As mentioned before, this is because, despite the undrained condition of the test, the large air volume present in the macropores of the surface soils ($S_r < 50\%$) prevents the generation of a significant pore pressure, thus generating a behavior closer to that of the drained condition, obtaining a better prediction of the initial stiffness of the material ($E_50$). However, at greater depths, the porosity decreases, and the degree of saturation considerably increases ($S_r > 80\%$); therefore, the porosity values generated are more relevant, and the stiffness of the material is closer to the undrained condition, obtaining high initial stiffness values far away from the drained adopted condition.

For future studies, it will be advisable to carry out a calibration process by triaxial CD tests to avoid the influence of excess water pore pressure generation during the failure step and to obtain a more realistic stiffness modulus.

6. Proposed Geotechnical Model

According to the stratigraphic profile of the CEGUnB (Fig. 2) and the parameters obtained for the HS model (Table 6), the proposed geotechnical model is presented in Table 7. The cohesion values for the soils of layers 1 and 2 were modified, which is explained in greater detail in the following section. The POP (pre-overburden pressure = effective preconsolidation stress - initial effective stress = $p_{o}^{\prime} - \sigma_{v,o}^{\prime}$) values were obtained from the compressibility curves ($\sigma_{v,o}^{\prime}$) of the consolidation tests and initial stress profile ($\sigma_{v,o}^{\prime}$); the $K_0$ values (lateral earth pressure at rest for normal consolidation) were based on Jaky’s criterion (1944); and the $K_0$ values (lateral earth pressure at rest) were based on the equation proposed by Mayne and Kulhawy (1982).

7. Validation of the Proposed Geotechnical Model

7.1. Validation through load testing of a single footing

The first part of the validation of the proposed geotechnical model and the mechanical parameters obtained for the HS model was performed for the porous clay surface layer through the numerical simulation of a single-footing load test conducted by Sales (2000) at CEGUnB. The general characteristics of the test are described in section 2.3.

The numerical simulation was performed using the 3D finite element method (Plaxis 3D, Brinkgreve et al., 2015). As shown in Fig. 15a, the symmetry conditions of the problem were considered. Fig. 15b shows the finite element mesh developed and the boundary conditions considered. The medium was discretized by a finite element mesh with more than 161,907 10-node tetrahedral elements, and the footing was discretized by six-node triangular elements. The densification of the mesh around and under the footing was considered. The lateral boundary conditions were fixed in the horizontal direction, and the bottom boundary conditions in both directions. Sensitivity analyses showed that the mesh density was sufficient to obtain accurate results.

The test was simulated considering the following steps:

1) Excavation to 0.8 m depth.
2) The displacement values were reset, and an incremental vertical load was placed above the footing until the maximum site value was reached (140 kN, Fig. 5); 3) Total removal of the load.

To simulate the excavation and get the best fit of the simulation with the loading vs. settlement graph obtained on-site, it was necessary to increase the cohesion value of the porous clay determined in the triaxial tests, \( c' = 0 \) kPa (Table 6, 1 and 2 m depth samples) to \( c' = 5 \) kPa (Table 7, layers 1 and 2). Figure 16 shows the comparison of the simulated and on-site load vs. settlement curves, and good agreement is observed between them. The increase considered in the cohesion value seems reasonable since it is possible to observe that this superficial soil in the city of Brasília generally maintains verticality in cuts without any type of support up to 2 m height. Likewise, due to the complex structure of this type of porous and collapsible soil, a loss of cohesion may occur during the collection, transport, and assembly of the undisturbed sample.

7.2. Validation by load testing on piles

The complete stratigraphic model and the mechanical parameters obtained for the HS model were validated through the numerical simulation of load tests on rein-
forced concrete piles conducted at the CEGUnB by Guimarães (2002; Table 2 and Fig. 4). The piles were built by the excavation method, with lengths of 7.25 to 7.8 m and diameter of 0.3 m.

Due to the cylindrical geometry of the problem, the model was considered axisymmetric (Fig. 17). The medium was discretized by a finite element mesh with 4,298 15-node triangular elements. Densification of the mesh around the pile was considered. The lateral boundary conditions were fixed in the horizontal direction, and the bottom boundary conditions in both directions. Sensitivity analyses demonstrated that the mesh density was sufficient to obtain accurate results. The pile concrete was assumed to be linearly elastic, with a stiffness modulus of 25 GPa and a Poisson ratio of 0.20. To adequately consider the interactions between the pile surface and the soil, five pairs of node interface elements were added.

The simulation was performed considering the following analysis steps:

1) Pile construction via direct replacement of the soil by the pile material (reinforced concrete).
2) Incrementation of the external loads in the same sequence as applied in the load tests (30, 60, 90, 120, 150, 180, 210, 240 and 270 kN).
3) Total unloading of the pile.

Piles 1 and 5 were simulated (Table 2). Figure 18 shows the comparison between the simulated and on-site

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**Figure 16** - Comparison of the load vs. settlement curves obtained from the experimental test and the explicit numerical modeling of the problem.

**Figure 17** - Finite element mesh and boundary conditions considered for the pile load test.
Obtaining the Mechanical Parameters for the Hardening Soil Model of Tropical Soils in the City of Brasília

Figure 18 - Load vs. settlement curves obtained from on-site load testing and numerical modeling.

load testing results, and good agreement is observed between the two load capacity assessments.

8. Conclusions

In this study, based on laboratory and field tests conducted as part of previous experimental field studies at the University of Brasília (CEGUnB), the mechanical parameters of characteristic soils of the city of Brasília were obtained, adjusted and validated for the Hardening Soil (HS) model of Plaxis software.

The methodology and main conclusions are summarized below:

a) With the information available from the different research studies, it was possible to define a stratigraphic profile typical of the CEGUnB, which is considered to be characteristic of the city of Brasília.

b) The compressibility parameters \( (E_{\text{v, ref}}, E_{\text{w, ref}} \text{, and } m) \) were obtained from six “simple” one-dimensional consolidation tests (under natural moisture and saturated conditions until reaching a stress of 200 kPa). The parameter \( m \) and moduli \( E_{\text{v, ref}} \) and \( E_{\text{w, ref}} \) were obtained by plotting the moduli \( E_{\text{v, ref}} \) and \( E_{\text{w, ref}} \) vs. \( \sigma'_v \) on a double logarithmic graph.

c) The shear strength \( (c', \phi') \) and stiffness \( (E_{50}, m) \) parameters were obtained from CIU (consolidation under isotropic conditions and failure under undrained conditions) and CKOU (consolidation under anisotropic conditions and failure under undrained conditions) triaxial tests. The parameter \( m \) and modulus \( E_{50} \) were obtained by plotting \( E_{50} \) vs. \( \sigma'_v \), on a double logarithmic graph.

d) Through the explicit numerical modeling of triaxial tests and the use of the SoilTest module of Plaxis software for one-dimensional consolidation tests, the parameters obtained for the HS model were adjusted to obtain the best representation of the curves of deviatoric stress vs. axial strain, isotropic stress vs. deviatoric stress (stress path) and axial strain vs. vertical stress (compressibility curve). In general, good agreement was observed between the laboratory results and the HS model. The main differences are because the HS model cannot simulate softening during failure or material collapse during saturation.

e) Based on the stratigraphic profile and on the parameters obtained for the HS model, a geotechnical model of the CEGUnB was defined.

f) The first part of the validation of the proposed geotechnical model and the mechanical parameters obtained for the HS model was performed for a porous clay surface layer through the numerical simulation of an isolated footing load test by Sales (2000) at the CEGUnB. The numerical simulation was performed using the 3D finite element method (Plaxis 3D). To simulate the excavation and obtain the best fit with the on-site loading vs. settlement curve, it was necessary to increase the cohesion value of the porous clay obtained in the triaxial tests to \( c' = 5 \text{ kPa} \). This cohesion value seems reasonable, since it is possible to observe that this superficial soil in the city of Brasília generally maintains verticality in cuts without any type of support up to 2 m heights. Likewise, due to the complex structure of this type of porous and collapsible soil, a loss of cohesion may occur during the collection, transport, and assembly of the undisturbed sample.

g) The complete stratigraphic model and the mechanical parameters obtained for the HS model were validated through the numerical simulation of the load testing on reinforced concrete piles conducted at the CEGUnB. Due to the cylindrical geometry of the problem, the model was considered axisymmetric. The loading vs. settlement graphs obtained for the on-site load testing and for the numerical simulation show good agreement between the two cases.

Finally, it can be concluded that the mechanical behavior of the soils of Brasília under natural moisture conditions can be modeled using the HS model. The parameter values obtained herein can be considered as representative of soils of the city of Brasilia, but they must be determined for each particular site and project, and the methodology presented in this study may help in their determination and validation.

Acknowledgments

The authors acknowledge the Coordination for the Improvement of Higher Education (CAPES), the National Council for Scientific and Technological Development (CNPq) and the Federal District Development Support Fund (FAPDF) for their support and partnership and Dr. Renato Cabral Guimarães for sharing laboratory test results performed at the UnB Experimental Field site as part of his Master’s thesis.
References


