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Class A prediction of a retaining structure made by a pile curtain wall executed on a tropical soil

Juan Carlos Ruge [a], Renato P. Cunha [b], Julio E. Colmenares [a] & Cristhian C. Mendoza [c]

[a] *Universidad Nacional de Colombia, Bogotá, Colombia. jcruge@unal.edu.co, jecolmenaresm@unal.edu.co*
[b] *University of Brasilia, Brasilia-DF, Brazil. rpcunha@unb.br*
[c] *Catholic University of Colombia, Bogotá, Colombia. ccmb80@gmail.com*

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Abstract
Some retaining structures can be designed on a temporary basis, in accordance to the geotechnical design and a predefined construction plan. Off course this may not be valid elsewhere, but in Brasília, where residual and laterized soils do prevail, best mechanical resistances are obtained in the porous clay setrata along the dry season. That means, it is controlled by soil´s suction, i.e. soil behavior presents a dependency on the unsaturated response of the medium. In numerical terms, to predict the soil-structure behavior it is necessary to include the atmosphere-soil interaction in the computational tool, using a hypoplastic model. The paper thus details on this aspect and on the numerical simulations of an existing retaining structure founded in the tropical soil of Brasília. The structure was monitored in terms of displacement, along local dry and wet seasons, and was simulated by a constitutive law calibrated by means of unsaturated laboratory tests.

Keywords: unsaturated response; numerical prediction; hypoplasticity; retaining structures; pile curtain.

Predicción clase A de una estructura de contención en pilotes ejecutada en suelo tropical

Resumen
Algunas estructuras de contención de acuerdo con su diseño geotécnico y plan de construcción predefinido. Por supuesto, esto no extrapolable a todos los sitios de análisis, sin embargo, en Brasilia, donde predominan los suelos residuales y lateralizados, se obtienen las mejores resistencias mecánicas en los estratos porosos durante la estación seca. Esto significa que es controlada por la succión del suelo, es decir, el comportamiento del material presenta una dependencia de la respuesta no saturada del medio. En términos numéricos, para predecir el comportamiento suelo-estructura fue necesario incluir la interacción atmósfera-suelo en una herramienta computacional, utilizando un modelo hipoplástico. El trabajo detalla de esta manera las simulaciones numéricas de una estructura de retención fundada en el suelo tropical de Brasilia. La estructura fue monitoreada en términos de desplazamiento, a lo largo de las estaciones locales secas y húmedas, y fue simulada por una ley constitutiva calibrada mediante pruebas de laboratorio no saturadas.

Palabras clave: respuesta no saturada; predicción numérica; hipoplasticidad; estructuras de contención; pilotes tangentes.

1. Introduction

In recent decades, the numerical models had a very important evolution from the point of view of prediction tools for the behavior of geotechnical structures. However, most of the analyses are still evaluated on qualitative rather than quantitative manner, as several external (and unknown) factors can directly affect the results, turning it difficult to consider all physical factors into simulations. Indeed, for non-classical soils behavior, a high dispersion of the data tends to be achieved when cross comparing numerical predictions with experimental in situ results [1].

Therefore, one of the main objectives that geotechnical engineers want to achieve is the prediction or validation of real case problems in full-scale dimension. That´s why the usage of numerical modelling techniques has become so

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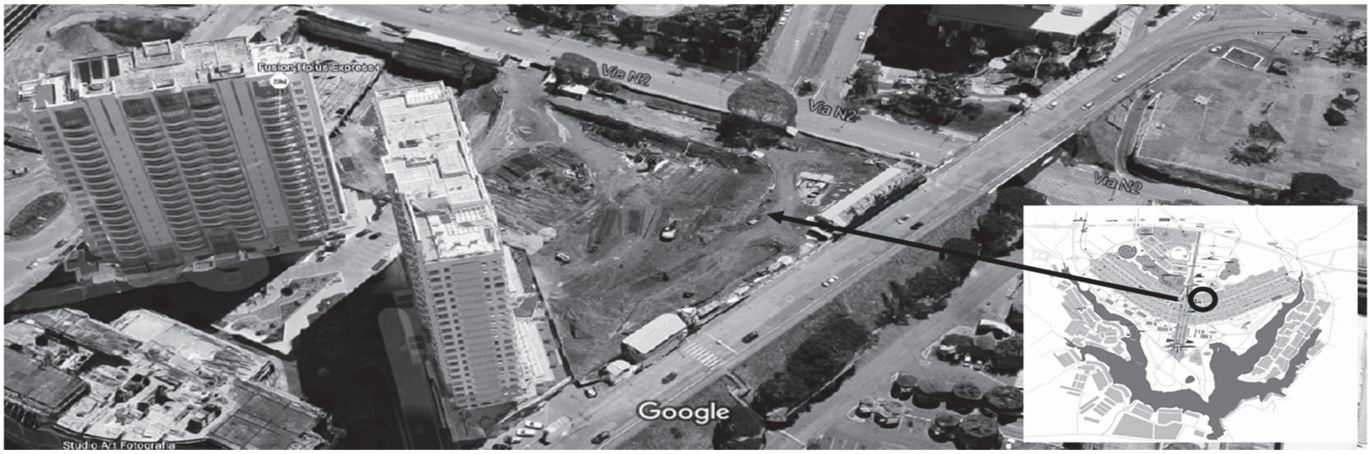

Figure 1. Overall situation of the excavation work in the city of Brasilia
Source: Taken of google maps.

increasingly important geotechnical researchers, albeit it is quite a challenging task. A good simulation depends on many aspects that affect the problem, among them the numerical model and input parameters, the correct execution sequence and other variables to properly address the geotechnical case under consideration [2]. As it will be exposed in this paper, traditional theories may eventually have disadvantages to perform analyses that properly predict the behavior of a geotechnical structure, especially when the soil is unsaturated or has distinctive features as structure degradation, cementation, and so on. Therefore, a hypoplastic constitutive model for clays that reproduces the unsaturated soil response of the material [3] is adopted herein. The final objective is the simulation of deep excavations made of anchored pile curtains in the tropical soil of Brasília, a soil that do conform to typical formations found in the Brazilian Midwest deposits.

2. Site Analysis

The construction site in question was an excavation executed at the hotel sector of Brasília, where a business related commercial building (Fusion Plaza) was to be built (Fig.1). In this site, by means of geotechnical investigations, it was possible to identify two main geotechnical strata (Fig.2). The first layer of 11 m o thickness approximately is composed by the typical red "porous collapsible" clay of the Federal District, geotechnically described as a clay with contents of sand, silt and gravels, soft to medium consistency. Beneath this overburden layer lies a clayey silt with sand lens (saprolitic soil), with medium to stiff consistency. The designed retaining structure (in the analyzed section) consisted of an evenly spaced 60 cm diameter "pile curtain" with total free + embedment length of 18 m spaced 1 diameter center to center apart. The embedment length was 5 m with several levels of anchored beams holding back the structure, tied to the ground by "passive" anchors that have been built using the standard execution technique of soil nails [4,5]. A typical cross section is sketched later in this paper.

In the localized area of the excavation (Fig. 2), in the upper strata, it was possible to observe the typical mechanical characteristics of this subsoil already seen elsewhere in Brasília, i.e. a very low strength to penetration in the SPT test (varying of 1 to 6 blows), a low bearing support, and an unsaturated region with high porosity, low density and relatively high permeability which overall details are found elsewhere [6,7,8,9]. Due to tropical laterization and lixiviation processes that this material has undergone, it presents a quite unstable open structure that "collapses" (sudden decrease of volume) when simultaneously subjected to wetting and loading [10]. Another research on a nearby site [11] has also demonstrated that the superficial weak strata varied up to 7 m in depth in 62 SPT borings. A highly-weathered material with oxides and lateritic concretions, that would justify the weak bonding between minerals, was also found by this latter author. The underneath saprolitic layer possibly comes from the weathering of the local mother rock, a metamorphic slate typical from all the region.

2.1. Basic characterization of the material

A complete characterization of the material was done in terms of physical and chemical tests. Table 1 shows some of the basic properties and geotechnical parameters of one sample of soil along the studied curtain section, where it could be possible to assess the correspondence between these values and those already published from the typical laterized "porous" clay of Brasília (see for instance [6,9,10,11])

In Fig. 3, some particle size distribution curves are shown for three distinct depths, denoting that perceptible differences in results are found when the tests are carried out with, or without, any dispersing agent to "break down" cementing bonds on the particles. As already noticed before for this material [10], the granulometry changes given the existence of "lumps" of bonded particles made of clay and silt minerals, forming false large particles that are interpreted as sand grains when analyzing the soil without the dispersing agent. There is a clear distribution of macro and micropores (bimodal) on the tested sample, despite of the granulometric curve being unimodal in both cases. Moreover, [12] inferred

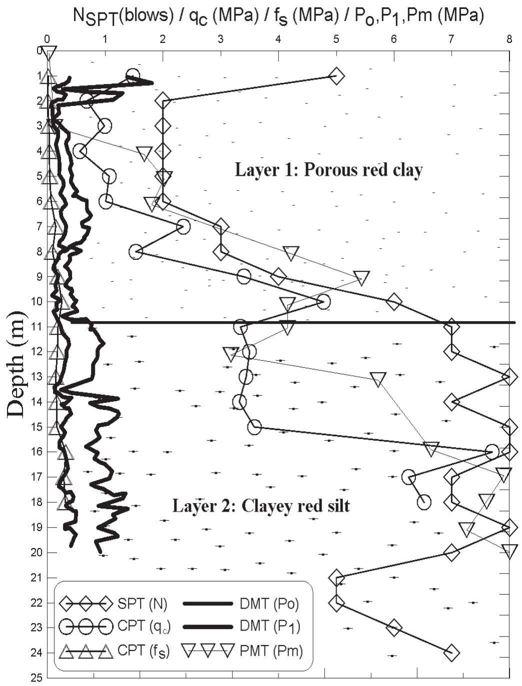

Figure 2. Results of several in situ tests in site under study
Source: The Authors

Table 1.

Basic characterization of the material under analysis

Sample	1B	2A
Depth (m)	6,0	9,0
w (%)	26,8	19,6
γ_s [kN/m^3]	28,01	28,02
γ_d [kN/m^3]	11,39	12,88
γ [kN/m^3]	14,70	15,51
Gs	2,81	2,82
LL (%)	48	50
PL (%)	29	30
PI (%)	19	20
IC	1,13	1,55

Source: Modified of Ruge *et al.* [4]

that the soil of the Brasilia-DF is composed by microconcretions of clay shaped by grains of silt and sand, producing an internal porous structure, that tends to “collapse” under a simultaneous load and wetting stage.

2.2. Microstructural analysis of the material

A mercury intrusion porosimetry test was also carried out with undisturbed samples from the site. It allowed the determination of the natural size and distribution of pores of the system (soil), as well as the bulk density and packaging density of a porous media core. This process is based in the

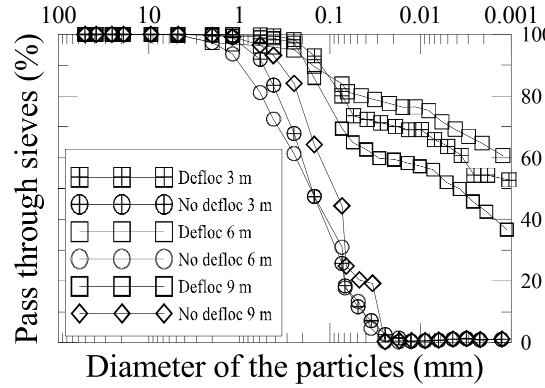

Figure 3. Particle size distribution at the samples
Source: The Authors

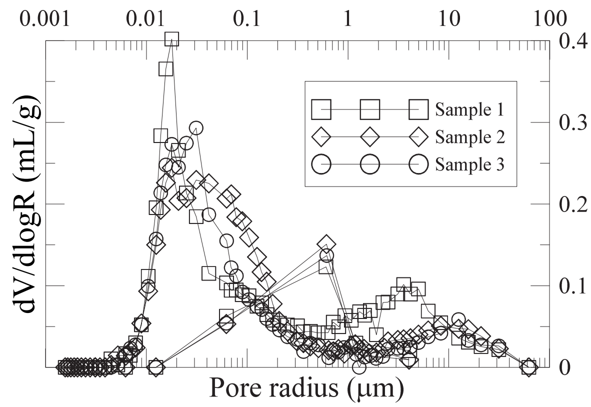

Figure 4. PSD of different samples at 9m of depth
Source: The Authors

injection of mercury within the pores of the sample, in order to estimate its capillary pressures. As a general rule, the size of the pore inversely proportional to the applied pressure. Hence, the tests allowed the determination of the pore´s volume and the distribution of size (PSD-Pore Size Distribution) (Fig.4). It is important to note in this former figure, where the derivative of the accumulated pore volume vs. pore radius is plotted against radius, that the variations of the inflexion points (in the source graph) are reflected in Fig. 4 by peaks related to the existence of micro and macropores. In the present case, the results evidenced a predominance of micropores in the studied sample.

Fig. 5 depicts a 15000x image of the SEM test, evidencing the lumps of clay of the sample, and the related macropores. It shows such lumps as aggregations that behave as cementing agents around of the minerals. In some cases, the soil´s minerals form links between two or more aggregations. It can also be noticed from this image that the water and the bonding aggregations can be located in three kinds of pores: macropores, mesopores and micropores. The macropores have diameters higher than 50 microns. The mesopores have dimensions between 50 and 2 microns, and the micropores less than 2 microns [13]. In this particular image of 15000x, the micropores are revealed and can even be measured with diameters between 200 nm and 800 nm (see Fig. 5).

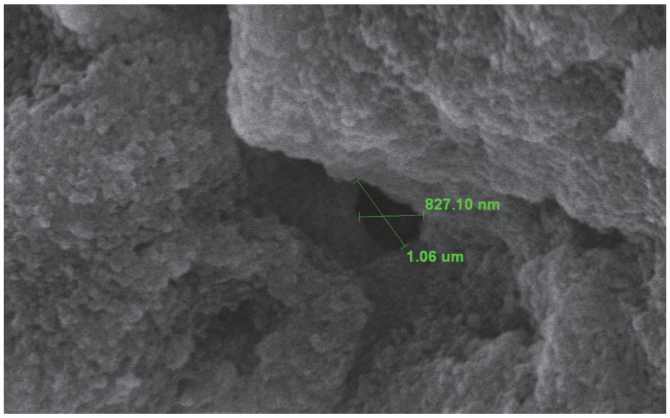

Figure 5 SEM image for 15000x in sample to 3 m of depth
Source: The Author

By adopting the combined data and results from both the porosimetry and the retention curve tests, one can effectively conclude that the macropores, the mesopores and the micropores have an important relevance in the mechanical and hydraulic behavior of this particular tropical material. Therefore, it is clear that the entrapped water from the macropores can be eventually mobilized to easily flow within the structure of the material, as infiltration water, thus directly affecting the variation of suction on this soil profile. Within the meso and micropores the water is more difficult to be mobilized as it is retained by other physical forces, thus helping to preserve the natural saturation of the soil. This tendency of having double porosity may show a distinct behavior of the soil´s stored water during the buildup of pore pressures that can take place during the load process. This is so because the small pores (micro and mesopores) will probably continue to drain the excess of pore pressure long afterwards the bigger pores (macropores).

The water that is stored in the micropores (adsorbed) is difficult to remove due to electrochemical attraction between anions of the water elements and the cations of the clayey particles. Thus, the cementing mineral aggregates that are possibly located in the skeleton of the material can be founded in the percolation water. It is clear the cementation in soils like the one under study has been well developed, originated from pedogenetic and lixiviation (combined) effects on minerals with carbonates and oxides, typical from climatic conditions of tropical regions [13].

The presence of different cementing minerals can increase an elastoplastic response in the soil. This effect is evident as a phenomenon known as collapse, previously cited, allowing the existence of plastic strains, due to the fragile nature of the cementing bonds. An important factor that defines the hydraulic and mechanical behavior of the porous clay is the fact that metastability and partial saturation provide similar effects. Therefore, the numerical modelling of this soil using a physical-mathematical model is a complicated task. The suction effect also influences in the hardening of the non-soluble cementing minerals [14]. For that reason, only in a saturated stage, where the suction is nonexistent, can one assume that the real cementation and the non-soluble minerals do act in a separate manner.

2.3. Mineralogical and chemical characterization

The X-ray diffractometry (XRD), which is a valuable tool to the mineralogical characterization of clayey minerals and other components, was also employed with the retrieved samples. The tests were carried out for samples at 6 m and 9 m deep. Using these technique, it was possible to identify minerals as quartz (63%), kaolinite (21%), gibbsite (11%) and hematite (5%), which goes along with results reported by [15]. In addition, it shall be highlighted that, although the quantity of these minerals is relatively low, their presence can have an influence on the cementation of the clay particles, probably generating a metastable behavior on the whole structure as suggested by [16]. Besides, collapsible soils form part of a metastable soil group, where the initial structure is broken due to environmental variations, in agreement with [17]. This latter author has also defined collapsible soils as the entire lateritic porous surface, characterized by the partial saturation and bonding.

2.4. Unsaturated and saturated laboratory experiments

2.4.1. Strength and consolidation tests

Fig. 6 shows the results of oedometric tests with 9 m deep samples at saturated, unsaturated and reconstituted conditions.

In the saturated curve, it is possible to confirm the response from a partially structured soil, where the bonding has an influence on the derived strains of the sample (at natural fabric conditions). It is readily shown that the Brasilia clay presents a metastable structure in the *ln p vs e* space,

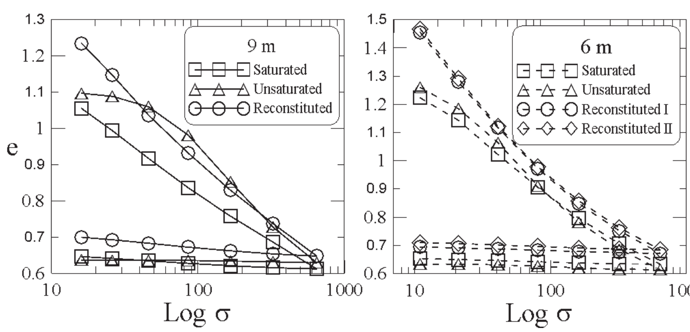

Figure 6. Oedometric test of Brasilia porous clay
Source: The Authors

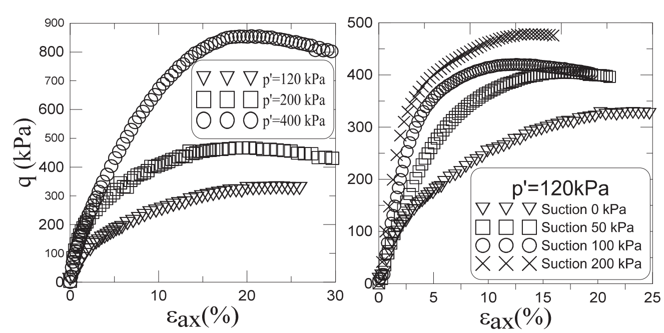

Figure 7. Triaxial test of Brasilia porous clay
Source: Modified from Ruge *et al.* [4].

where the stress sensitivity decreases along the loading stages, a characteristic response of normally to lightly overconsolidated clay materials. In relation to drained triaxial tests with samples under both saturated and unsaturated conditions, as presented in Fig. 7 (a, b), it is possible to visualize responses for respectively distinct (increasing) confining stress and suction levels.

2.4.2. Soil Water Retention Curve

The relation between suction and humidity represents an important function in the characterization of an unsaturated soil, this relation can be evidenced by means of the comparison between the stored water and the soil suction, usually denominated as the "suction water characteristic curve" or SWCC. Fig. 8 shows the SWCC of the soil through the axis translation technique, with a maximum matric suction of 1500-kPa that corresponds to a 35% of residual volumetric wet content, according to the distribution of pores and the retained water. The mathematic model proposed by [18] by means of the eq. (2) was then selected to simulate the behavior of the SWCC in the material. This model simply relates the saturation degree with the pressure head.

$$S_{(\phi_p)} = S_{res} + (S_{sat} + S_{res})\left[1 + \left(g_a|\phi_p|^{g_n}\right)\right]^{g_c} \quad (2)$$

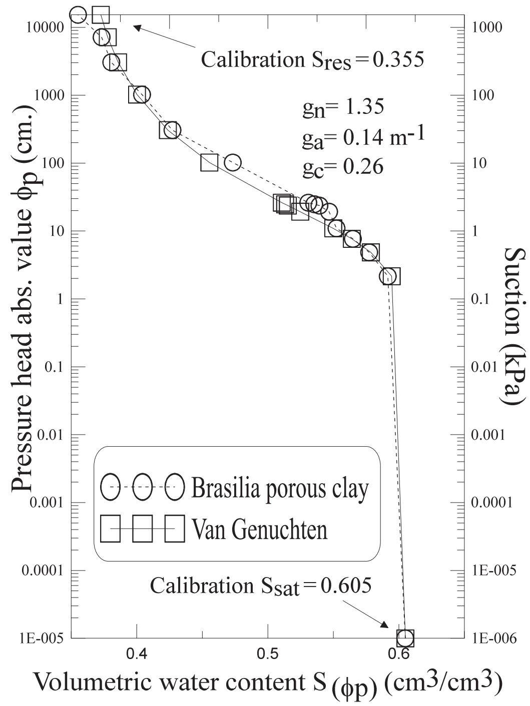

Figure 8. SWCC of the Brasilia porous clay.
Source: The Authors

3. Instrumentation of the excavation

Three lines of (passive anchor) nailings of respectively 17, 12 and again 12 m composed the typical supporting device of the excavation/retaining structure. In the selected section for instrumentation, it was installed an inclinometer unit that was located one meter behind the equally spaced piled structure. This was done in order to monitor the deformations in this structure during the different stages (or execution events) of this particular excavation/wall development (Fig. 10).
They were pinned to the piled curtain through anchored beams at distinct depth levels (V1, V2 and V3).

The execution procedure was quite straight forward. The piles were firstly executed and excavation was proceeded until each of the anchored beams. The nails were inserted and the beams were subsequently constructed and attached with bended bars to the recently executed nails (the whole process in sequence, at each excavation level). Inclinometry and horizontal displacements were taken throughout this sequential series of events, and afterwards (see timeline in Fig. 9).

3.1. Inclinometer measurements

Fig. 15 presents the inclinometer results sorted for time in days, where the maximum displacement of 24,00 mm was measured in a date very close the end of the 13-m excavation depth below ground level (at 115 days since beginning). It is also noticeable the low level of horizontal displacement of around 2,5 mm that was measured just after the first excavation sequence (up to V1 – 8 days' value), before the installation of the nail.

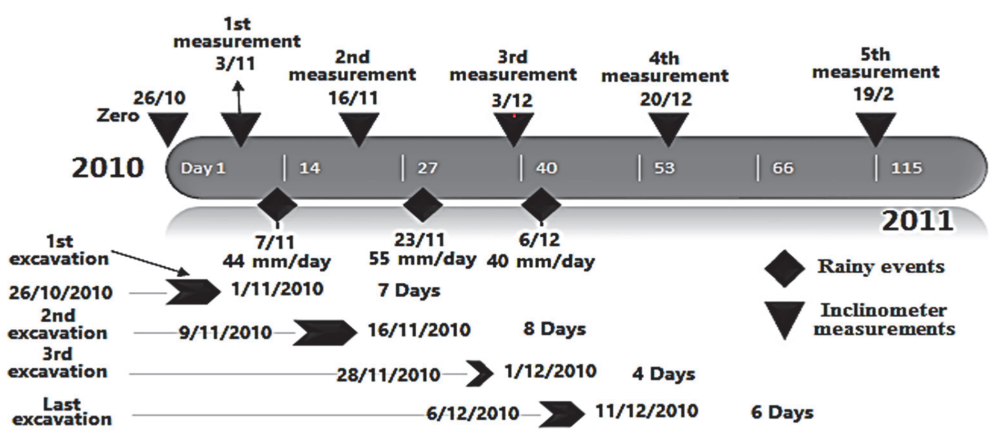

Figure 9. Timeline of the excavation under analysis
Source: The Authors

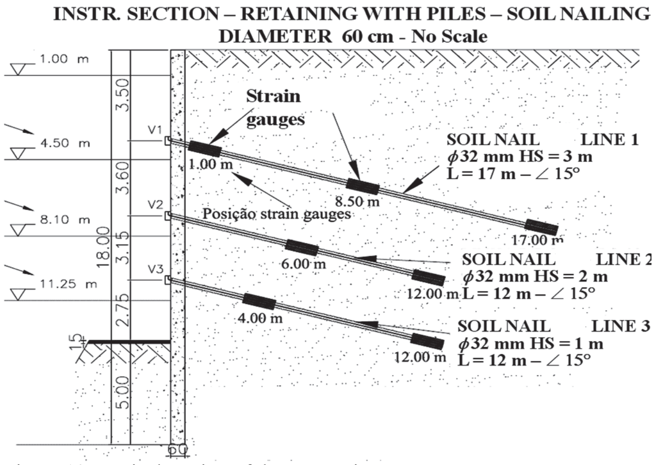

Figure 10. Typical section of the excavation
Source: The Authors

A total displacement of around 4 mm was measured just after the second level of excavation (up to V2 – 21 days' value), where the first upper level of nail was already in place. Between the 21th and 37th day occurred the maximum rainy events on the site, which caused a significance difference among the measurements observed in the Fig. 16. Also, this situation can be analyzed in the timeline shown in the Fig. 9. From the 37th. day until the 55th. (last excavation) the same pattern of behavior was noticed for the measured displacements along depth, with a maximum value of around 22.00 mm. Surely, the observed behavior was simultaneously affected by the excavation sequence and the presence of strong and continuous rains in the region. However, at such time there was a rain event of relatively strong magnitude, that a priori does not seem to have significantly affected the measured displacements, given the fact that dry weather prevailed during all remaining excavation period

4. Constitutive model and parameters calibration

4.1. Reference model

This research adopted as reference model the hypoplasticity theory for unsaturated soils. It also used constitutive equations that were developed with the critical state concept and effective stress principle, considering the effect of an increasing stiffness that was governed by the variation of suction on the mechanical response. The phenomenon of collapse by wetting [3] was also considered.

4.1.1. Constant suction model

As previously mentioned the reference model is supported by the concept proposed by [3]. In this case, suction influences both effective tension and normal forces on the interparticle contacts, hence improving the stiffness of the soil´s skeleton, as the particles are closer and interact on a rather stronger manner. In other words, this behavior expands the State Boundary Surface (SBS), likewise bonded soil particles within cemented saturated geomaterials [19]. Under such conditions, the SBS size for unsaturated soils is defined by the NCL, which on the other hand is based on the compression law given by [20] for the critical state line, as described in the following equation:

$$\ln(1+e) = N(s) - \lambda^*(s)\ln\left({p}/{p_e}\right) \tag{3}$$

From where e is the void ratio, considered as a state variable in the constitutive model. The expressions *N(s)* and *λ*(s)* predict the position of the inclination of the compression virgin line in the space $ln(p/p_r)$ *vs* $ln(1+e)$ for a particular suction s. Equation 4 shows the expression for an equivalent Hvorslev type tension on the NCL at a given suction s [21].

$$p_e = p_r . e^{\left[\{N(s)-\ln(1+e)\}/\lambda^*_{(s)}\right]} \tag{4}$$

[22] modified the values of the barotropy f_s and picnotropy f_d, since it was demonstrated that the incorporation of the virgin compressibility in the intercept *N(s)* affects the original values proposed by [23]. It is important to consider that the reference model describes the behaviour of the material for any state of overconsolidation. Therefore, some few extra assumptions need to be formulated in order to define the constitutive equation (eq. 5) at any stress state, as follows: (1) the stability of the interparticle contacts is governed by the suction level; (2) in more open soil structures the coordination number tends to be lower, hence the contacts support larger shear forces [3]. Therefore:

$$\dot{\boldsymbol{T}} = f_s(\boldsymbol{L}:\boldsymbol{D} + f_d N \|\boldsymbol{D}\|) + f_u \boldsymbol{H} \tag{5}$$

Where s was introduced to consider the first assumption, and f_u, is the modified picnotropy factor that controls the tendency of the soil´s structure to collapse, as reproduced by the second assumption. When the latter factor is equal to 1 in the SBS, the structure tends to be as open as possible, and collapse is solely controlled by the $\boldsymbol{H}$ variable. Instead when f_u tends to zero, $OCR=p_e/p$ tends to infinite, hence no wetting induced interparticle slippage occurs in a highly overconsolidated soil.

$$f_u = \left(\frac{p}{p^{SBS}}\right)^m \tag{6}$$

Where p^{SBS} is the effective mean stress at the SBS, that corresponds to the current normalized stress *T/trT* and the current void ratio e. The variable m is a model related parameter that controls the influence of overconsolidation on the wetting-induced collapse. It shall be pointed out that the model for the unsaturated response of the soil do contain some saturated related parameters (ϕ_c; N; λ; κ; r), according to [24]. Besides, variables N; λ; κ: are calibrated based in a simple isotropic loading and unloading experimental test. These parameters must also be calibrated with basis on the inclination of the isotropic unloading line (details in [22] and Fig.11).

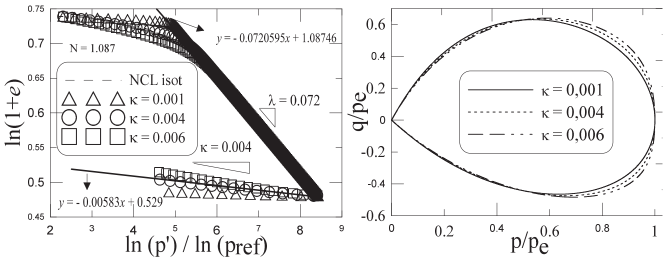

Figure 11. Definition of parameters N, λ^*, κ^* and quantities p_{cr}, p_e^*
Source: The Authors

The critical state friction angle (ϕ_c) is found by using a linear regression with the points of the critical state line, which is derived by available experimental shear tests. The parameter r must be evaluated directly with the ratio of the bulk and shear moduli, which are obtained with the compressed normally stress state line from such tests. Since the model predicts a gradual degradation of the shear stiffness, it is advisable to find an appropriate value of its parameters by using a parametric study. The extension of the proposed model will require to take on account the influence of the suction variable on both N and λ values, respectively denominated as $N(s)$ and $\lambda(s)$. m controls the collapse of the structure along the wetting paths. Se is the value of suction in the entry and/or exit of the air pores. The scalar quantities n and l are included in the formulation of $N(s)$ and $\lambda(s)$, so to control the unsaturated NCL.

4.2. Calibration of the parameters

It is necessary to adjust the parameters obtained at the laboratory through the constitutive law selected for the numerical phase. The calibration of the saturated parameters (λ^*, κ^* and N) are shown in Fig. 11. The friction angle of the critical state is obtained (ϕ_c) from results in triaxial tests. The unsaturated parameters m, l and n of the model can be computed from the saturated and unsaturated normal compression lines, using isotropic compression tests and eq. 7.

The parameter m is presented in Fig. 12. It is possible to analyze the saturated and unsaturated NCL and to obtain the value of $N(s) = 1{,}14$. From the equation of NCL_{unsat} (Eq. 7), can also found the value of $n = 0{,}032$ by solving for $s = 200$ kPa and $S_e = 39$ kPa (from the SWCC). Since both κ and $\kappa(s)$ values are equal, as depicted in Fig. 12, the parameter l takes the value of zero.

$$N(s) = N + n \ln\left(\frac{s}{s_e}\right) \qquad (7)$$

5. Computational model

As previously cited, a large excavation pit for a commercial building was constructed in the North Hotel Sector of Brasilia. The pit was around 13-m deep by +100 m wide in a rectangular format (see Fig. 1). A contiguous spaced pile curtain wall was constructed using 60 cm diameter bored piles 1 diameter apart. This wall was designed to be temporarily sustained by three levels of passive anchors

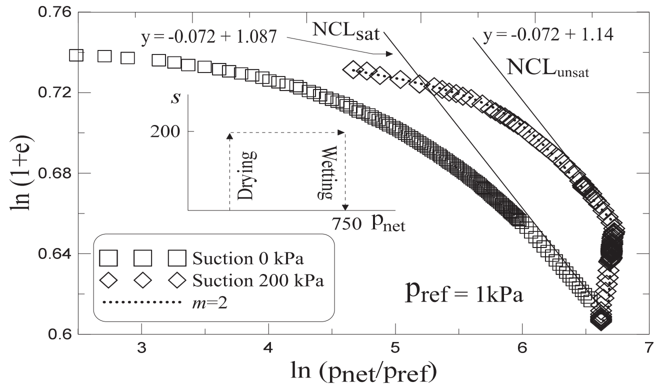

Figure 12. Calibration of the parameter m.
Source: The authors

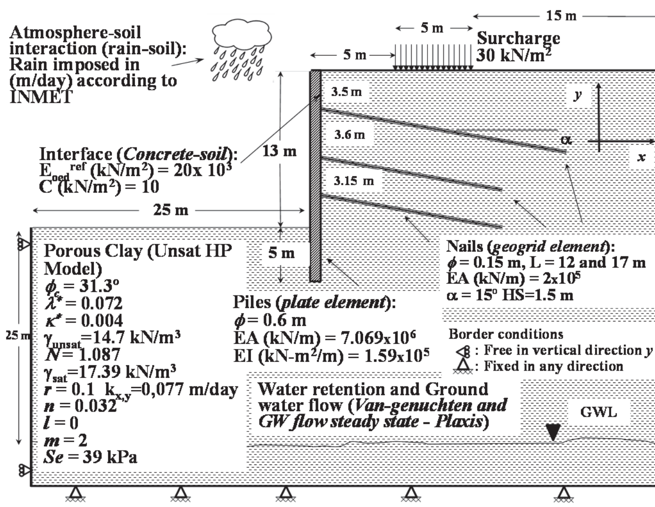

Figure 13. Computational model of the geotechnical problem. The excavation sequence is reproduced by the time line history of Fig. 9.
Source: The authors

(nails) and beams, as represented in Fig. 10 – later to be held in place with the structural slabs of the edification. The subsoil consisted of a homogeneous layer of the typical unsaturated Brasília porous clay, with a natural specific weight of around 14,70-kN/m^3. Other parameters were duly estimated via laboratory results or local experience, as depicted in Fig. 13.

Table 2 complements de previous cited figure, where it is possible to observe the properties of the some of the variables

Table 2.
Elements in the computational model

Curtain of contiguous pile	
Material behaviour	Elastic
Axial Stiffness EA [kN/m]	7,069 x 10^6
Flexural Stiffness EI [kN/m^2/m]	1,59x10^6
Weight [kN/m/m]	6,78
Poisson ratio	0,15
d [m]	0,52
Soil nailing (geogrid element)	
Axial Stiffness EA [kN/m]	2 x 105

Source: The authors

Table 3.
Numerical sequences in the excavation process

Stage	Description	Average rain (mm/day)	Duration (days)
1	Geostatic stress balance	1,70	------
2	Pile borehole and surcharge	1,70	2
3	First excavation - 4.50 m.	8,30	5
4	1st row of nails	12,60	6
5	Second excavation - 8.10 m	3,90	7
6	2nd row of nails	7,03	12
7	Third excavation - 11,25 m	21,63	3
8	3rd row of nails	21,13	3
9	Final excavation - 13.00 m	10,06	5

Source: The authors

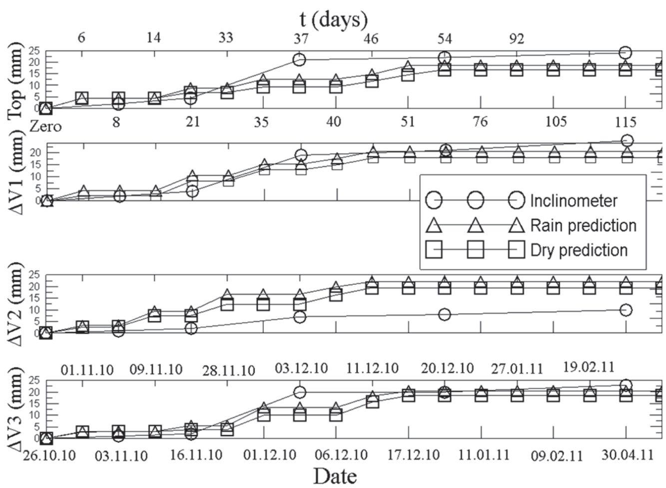

Figure 14. Numerical predictions according to depth of the soil nailings
Source: The Authors

from the proposed model for the numerical analysis. It shall be noticed that the pile wall was introduced as an equivalent plate element with thickness *d* but similar flexural rigidity, as a 2D modelling was adopted rather than the more realistic 3D case.

One shall realize, on the other hand, that besides of the constraints of the bi-dimensional analysis, it was possible to include the "environmental factor" (rain simulation) in the simulations, for each constructive excavation sequence, as stated in Table 3.

Among the objectives, it was tried out the comparison between simulated and "real" (inclinometer related) horizontal displacements along the wall. The simulations took into consideration both the rain and execution events (Table 3), and a hypoplastic model [3]. The horizontal measurements were made by a total station device and brass pins on top of the piles, as well as on top of each of the anchored beams. Fig. 14 depicts the evolution of displacements along with the execution sequence (in days from time zero – first reading before excavation – see Fig. 9). The predictions included the measured rain from the period and a non-realistic "dry" scenario, just to contrast the results from the constitutive model, i.e., to check upon the sensibility of considering or not suction variations into the soil.

Table 4.
Predictions of geotechnical structures

Prediction	Moment	Results
A	Before event	----------
B	During the event	Unknown
B1	After event	Known
C	After event	Known
C1	After event	Known

Source: Lambe [23]

6. Numerical predictions using the hypoplastic model with an unsaturated response

Given the fact that the numerical analyses consisted of 2D simulations of aforementioned geotechnical problem *before* the real excavation process took place, one can conclude that "type A" predictions in accordance to [25] (Table 4) were actually carried out in the present research.

Plane strain conditions were considered to represent the problem, and the wall geometry and surrounding elements were modeled with 1682 finite elements of 15 nodes. The piles (or equivalent plate) were simulated using appropriate values of flexure and axial stiffness, as shown before. Interface elements were also placed around the plate´s boundary so to adequately reproduce the pile-soil interaction slippery effect.

As previously commented, the excavation sequence was simulated in numerical stages that tried as faithfully as possible to reproduce the real sequence in situ. Atmospheric conditions that may also influence the soil suction and overall behavior of the structure were considered "as close" as possible to real (regional) measurements of pluviosity and weather. For instance, it was introduced a rain value in [m/day] along the analyses, based on the instrumented data. Given the specific characteristics of this soil in terms of mineralogy, structure, depositional aspects and geotechnical behavior, and the intrinsic aspects of the site and work (execution sequences and timing, rain events and so on), a fully drained condition was envisaged and hence adopted.

Although not explicitly commented before, one of the final aim of this exercise was to develop and validate research tools to properly simulate, by means of traditional FEM analyses, a real geotechnical structure of the region. This structure, as usually done in the region, was constructed under unsaturated tropical soil conditions, as well as variable rain fronts (soil humidity events). An important issue was the possibility of developing a temporal evaluation of the execution that included "real" precipitation stages. As it will be demonstrated soon in this paper, the increased complexity of the numerical model has indeed paid off in terms of accuracy of results. It shall also be emphasized that, in this type of validation, the qualitative tendency is also of importance, as the site behavior do suffer from many external factors that cannot be completely inserted into the numerical model – hence accounts for simplifications must always be done [1].

6.1. Deformations in the curtain

Fig. 15 presents the measured displacements and the predictions over time for the top of the pile and the distinct levels of the beams in the wall. The predictions are tied to similar dates (in days) to the execution sequence, so to get comparable forecasts on the analysed problem. HP stands for the predicted horizontal values, whereas Inc relates to the measured inclinometer values. It can be initially noticed in this graph the expansion of the displacement level from day 21 to 37 (more pronounced in the measured data rather than in the predicted one). Indeed, this period coincides with the third stage of the excavation, where high rates of rain with values close to 50 mm/day took place. This is considered as heavy rain event, since in Brasilia most of the events last short time spans, sometimes of 50 (+) mm in less than 60 minutes [26].

It is also significant to note that the typical Brasilia soil material have permeabilities that are relatively high, in the range of $9x10^{-5}$ cm/s (so similar to fine sand, given the porous structure at the top surface layers). Therefore, heavy rain effects may eventually increase rapidly the humidity at the surface levels, especially during the continuous rainy season (that goes from around November to April depending on Pacific conditions – “el niño” or “la niña” effects). This effect will change suction values along depth, consequently the behaviour of the structure and may cause a high impact in the suction when the saturation increases. This is revealed in Fig. 15, where one notices the continuous increase of displacement measurements from day 37 and beyond (day 37 relates to the beginning of December).

Nevertheless, rains of short duration on a high temperature environment (annual average above 20-mm), typical of Brasilia, can also cause a fast evaporation of the

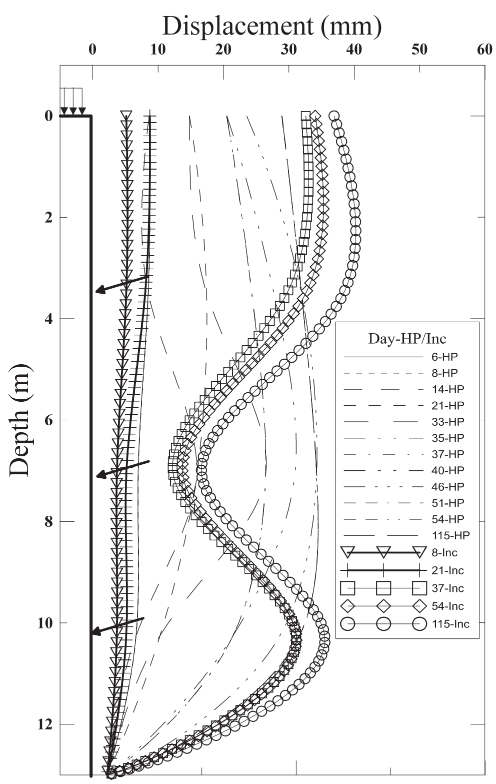

Figure 15. Displacements and numerical predictions in the curtain
Source: The authors

water that eventually runs off to surface or percolates into the soil. Probably the downwards wetting front from rainy events will solely affect the soil´s suction at the shallowest depths of the deposit.

7. Conclusions

In this research, it was possible to confirm the presence of macropores and micropores in the soil structure with the use of porosimetry results and SEM observations. This distinct presence and variability will undoubtedly affect the mobility and retention of water in the soil – hence changing its behaviour under shear or loading conditions, as it was demonstrated herein.

This is so especially for particular “tropicalized”, laterized and unsaturated soils as the Brasília porous clay, where the influence of the water content, hence soil´s suction on the overall macro behaviour, is real. Matric suction also influences the mechanical properties of the deposit, which leads to the initial conclusion that numerical analyses of such geotechnical media and buried structures must take on account, somehow, this effect on the rheological model. Experimental as well as numerical (forecast) results did demonstrate the clear necessity to include the suction phenomena within the hypoplastic model, leading to distinct results (or forecasts) in accordance to the input rain throughout the study period of the site/structure. The problem however is not so simple as it seemed at first view.

The limited analyses of this paper shed some light on this difficult aspect, and besides of (preliminary) “reasonable” comparative results, it is also concluded that other aspects must have been incorporated in the model, as evaporation rates, unsaturated flow through the surface soil layers, solar irradiance degrees, air humidity and so on. Indeed, not an easy problem to deal with. Nevertheless, as previously stated, encouraging results have been found, even with the usage of a “simple” geotechnical model that could take into consideration (at least) the soil´s suction phenomena. There was a direct correspondence between displacements and suctions in the derived predictions, i.e., the displacements increased after rain events. The variation was not so abrupt, but it has demonstrated, particularly in the first top layers, where the saturation of the soil was more susceptible to changes, that displacements could indeed be influenced by such parameter.

Yet to be experimentally proved in the future, or by further (more complex) analyses, the percolation phenomena coupled to the evaporation of the soil (plus run off effects) tended to normalize the deposit´s matrix suction after short period of rainy events, especially when such events were sporadic (non-continuous) and limited in time and reach (solely affecting the superficial layers). It can be concluded therefore that although simple, the constitutive hypoplastic model adopted herein suffice to simulate the behaviour of a typical pile curtain wall in the city of Brasília, thus opening the possibility to allow local designers to forecast tendencies of performance at distinct temporal, seasonal and execution ranges.

The predictions also highlighted the importance to qualitatively study the results, given local variations in experimental x forecasted results that are of difficult matching (by localized effects or things that cannot be properly simulated in engineering terms). On overall terms, however, a reasonable match was found and the results did agree in qualitative terms, besides of small but perceptible quantitative differences (again denoting localized problems and/or limitations of the model). One point of curiosity was duly noticed herein: The numerical predictions tended to be less conservative than the experimental data results. This point however needs further confirmation.

The hypoplastic constitutive model, also proved to be a useful tool useful when it was necessary an approach to the modelling and simulation of a real case geotechnical structure built in a tropical laterized and unsaturated deposit. The input of the (close to real) rainfall patterns of this region in Brasilia during the wall´s construction did indeed aggregate value to the analyses, increasing its accuracy power. Perhaps in the future, as stated before, other variables will also be taken on account.

Thus, depending on accuracy aspects or detailed (or not) behavioural tendencies that may be sought in such analyses, models with the soil/environment interaction will have to be adopted (somehow as done herein in a simplified manner). This aspect is of upmost importance when seasonality effects around the year need to be considered, or when the excavation takes place at a particular condition and a temporary curtain wall interacts with the surrounding environment at another distinct condition (dry x wet seasons, like the case of the present study). Perhaps other aspects will be required in future analyses, particularly when someone takes on consideration possible greenhouse influences that the climate might experience (or is already been experiencing).

Much can still be improved on top of the discussed preliminary results of the present paper, but one is already crystalized: the necessity of adopting more complex and rational models that do take on account the main aspects of behaviour of unsaturated tropical soils when loaded or unloaded, among them the soil´s suction and the correct execution sequence of the problem.

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J.C. Ruge, received the BSc. Eng in Civil Engineering in 2002 from Universidad Francisco de Paula Santander (UFPS), Colombia, MSc. degree in Civil Engineering with emphasis on Geotechnics in 2005 from Universidad de Los Andes, and the PhD degree in Geotechnics in 2014 from University of Brasilia (UnB), Brazil. Formerly postdoctoral researcher fellow in the Civil and Agricultural Department, at the Universidad Nacional de Colombia. He is currently assistant professor of the Universidad Militar Nueva Granada, Bogotá D.C., Colombia.
ORCID: 0000-0002-9100-6058

R.P. Cunha, received the BSc. Eng in Civil Engineering in 1985, the MSc. degree in Civil Engineering by the Coordination of Post-Graduation Programs in Engineering – COPPE/UFRJ (1986), PhD in Geotechnics by the University of British Columbia – UBC in Canada (1994), and a Post Dr. at the University of Sydney – USYD in Australia (1999). At present moment, he is a Professor of the University of Brasília (UnB).
ORCID: 0000-0002-2264-9711

J.E. Colmenares, received the BSc. Eng in Civil Engineering in 1989 and the MSc degree in Geotechnics in 1996 from the Universidad Nacional de Colombia, MSc, in 1997, and PhD, in 2002, degrees in Soil Mechanics from Imperial College London. He is currently titular professor in the Department of Civil and Agricultural Engineering at the Universidad Nacional de Colombia, Bogotá D.C, Colombia.
ORCID: 0000-0002-1485-0327

C.C. Mendoza, received the BSc. Eng in Civil Engineering by Universidad Distrital (UD) in 2005, the MSc. degree in Civil Engineering with emphasis on Geotechnics by University of Los Andes (Uniandes) in 2009, and the PhD degree in Geotechnics University of Brasília (UnB) in 2013. Formerly postdoctoral researcher in the Civil Department, Universidad de Los Andes, Colombia. He is currently assistant professor of the Universidad Nacional de Colombia, Manizales.
ORCID: 0000-0002-2888-6507