Geotechnical Engineering Journal of the SEAGS & AGSSEA Vol. 51 No. 2 June 2020 ISSN 0046-5828

Evaluation of Analytical and Numerical Techniques to Simulate Curtain Pile Walls in a Tropical Soil of the Federal District of Brazil

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ABSTRACT: This paper is fruit of an active interaction between several universities/academics and the University of Brasília. It has focus on the design of retaining walls. This design is increasingly present in engineering projects for urban areas, given their continuous development. In the Federal District of Brazil, many retaining walls are built in order to optimize space. These excavations need retaining works in order to maintain the terrain in place and to avoid any collapse. The aim of this paper is to tentatively assess the behavior of retaining structures made of "pile curtain" by using existing commercial finite element (F.E.) and "winkler spring" type softwares. The case study is characterized by a 13m deep excavation founded in the well-known porous clay of Brasilia. The excavation´s instrumentation provided displacements along depth, used to evaluate the structure´s behavior. In terms of the approach by using a winkler type software, the best technique to evaluate the subgrade reaction coefficient (modulus) was an empirical equation presented by Bowles (1988). For the numerical predictions with finite element method, the Hardening Soil model tended to show a slight better result when compared to the traditional Mohr Coulomb model. It is concluded that simple techniques, and experience of the engineer, are sometimes fundamental to better simulate such types of structure founded on complex unsaturated tropical soils.

KEYWORDS: Retaining Structure, Numerical Simulation, Tropical Soil, Winkler Model, Subgrade Reaction Coefficient

1. INTRODUCTION

In the last decades, there has been an increase in the use of computational tools to predict the behavior of geotechnical structures. However, the existence of several factors that can affect, directly or indirectly, the results of these methodologies, requires most analyzes to be performed in a qualitative way, since a high dispersion of data between the predicted values and the measured *in situ* results can be observed (Simpson, 1992). It can be affirmed that this high data dispersion between observed and calculated results suffers a great influence of the constitutive model´s choice for the soil and the applied methodology to obtain soil parameters in terms of quality and quantity. In addition, simulating soil behavior is not an easy task, as it depends on several factors that may vary from one material to another (Ruge, 2014). This difficulty is especially true for tropical soils. Among tropical countries, Brazil has most of its area covered by lateritic soils, which usually present a high porosity and an unsaturated condition. By definition, tropical soils are those that are located between the tropics and exhibit a mechanical behavior and physicochemical properties differentiated from the Classical Soil Mechanics developed for temperate climate soils. Conciani *et al.* (2015) state that the conditions of formation of these soils and their evolution over time make their behavior better understood by the Unsaturated Soils Mechanics. Tropical soils can present a special behavior, governed by its response in the unsaturated condition that affects soil suction, influencing its shear strength and rigidity, besides of the presence of a metastable structure that can be degraded by external factors such as loads and moisture variation.

Regarding Brazilian tropical soils, the porous clays of the city of Brasilia stand out, which present peculiar characteristics, such as structural metastability that is directly linked to the collapse potential of these soils. In the Federal District of Brazil, many retaining walls are built in order to optimize space, maintain the terrain in place and avoid any collapse. The design of retaining walls is increasingly presented in engineering projects for urban areas, given their continuous development of large urban centers. Thus, the use of computational tools to estimate the behavior of retaining structures is becoming more frequent due to the excellent results obtained from these methodologies. Therefore, the aim of this paper is to evaluate the behavior of retaining structures that consists of retaining curtain walls made by adjacent piles (i.e. juxtaposed pile walls), by using an existing commercial numerical and Winkler type softwares. A 13m deep excavation founded in the well-known porous clay of Brasilia characterizes the case study. The excavation instrumentation provided displacements and stresses data in order to evaluate the structure´s behavior. Additionally, laboratory tests such as drained and undrained triaxial tests and direct shear tests were performed in order to obtain parameters for the numerical analyses. The numerical and analytical models of the structure are then developed and compared with the instrumentation. A meticulous study on different approaches to obtain the Young's modulus and the horizontal subgrade reaction coefficient, respectively for both methodologies, is carried out. It is verified that the experience of the designer represents a key aspect in those types of analyses.

2. SOIL CHARACTERISTICS

The city of Brasilia is situated in the Federal District, located in the Central Plateau of Brazil. This district has a total area of 5814 km^2 and is limited in the north by the 15°30' parallel and in the south by the 16°03' parallel. The region has elevations between 750 and 1,300 m. Within the Federal District extensive areas (more than 80 % of the total area) are covered by a weathered lateritic soil of the tertiary-quaternary age. This lateritic soil has been extensively subjected to a weathering process and it presents a variable thickness throughout the Federal District, varying from few centimetres to around 40 meters. There is a predominance of the clay mineral kaolinite, and oxides and hydroxides of iron and aluminium. The variability of the characteristics of this lateritic soil depends on several factors, such as the topography, the vegetal cover, and the parent rock.

The superficial lateritic soil is locally known as the Brasilia porous clay, which is constituted by sandy clay with traces of silt, presenting lateritic horizons of low unit weight and high void ratio, also a high collapse coefficient. According to Araki (1997), these clays present a very porous structure, low resistance to penetration (NSPT < 4) and are highly unstable when subjected to variations in stress state, displayed as a result of its contraction behavior. Although these characteristics vary in this city, its main geotechnical characteristics are generally similar. These characteristics were studied by Cunha et al. (1999) and are presented in Table 1 as an example of the range of values typically found for this clay.

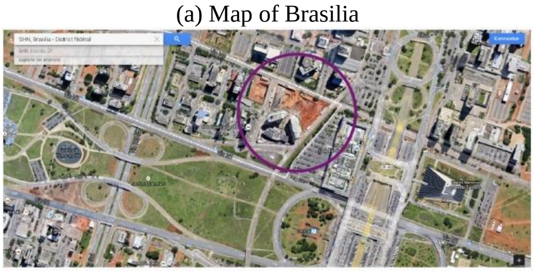
Table 1 Geotechnical typical parameters of the porous clay in Brasilia (Cunha et al., 1999)

Parameters	Values	Unity
Sand	12-37	%
Silt	8-36	%
Clay	80-37	%
Dry density	10-17	kN/m³
Bulk specific weight	17-19	kN/m³
Water contente	20-34	%
Degree of saturation	50-86	%
Void ratio	1.0-2.0	--
Liquid limit	25-78	%
Plastic limit	20-34	%
Plasticity index	5-44	%
Cohesion	10-34	kPa
Angle of friction	26-34	degrees
Young's modulus	1-8	MPa
Coefficient of collapsibility	0-12	%
Coefficient of earth pressure at rest	0.44-0.54	--
Coefficient of permeability	10^{-06}-10^{-03}	cm/s
Coeff, of consolidation	10^{-08}-10^{-05}	m²/s

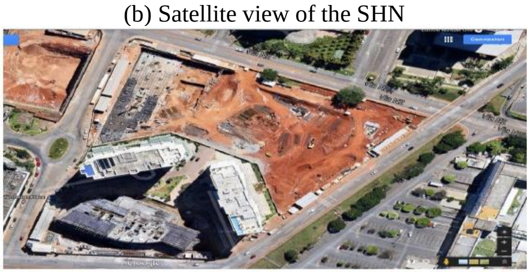
3. DESCRIPTION OF THE CASE STUDY

The construction site analysis was performed on the Northern Hotel Sector (SHN) of the Federal District of Brazil. Figure 1 presents the location of the case study.

(a) Map of Brasilia



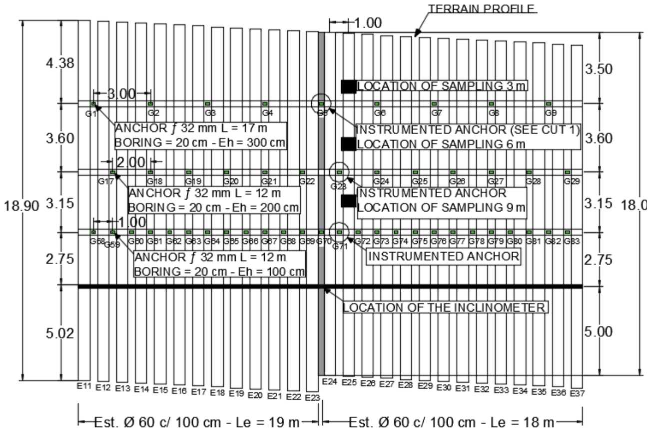
(b) Satellite view of the SHN



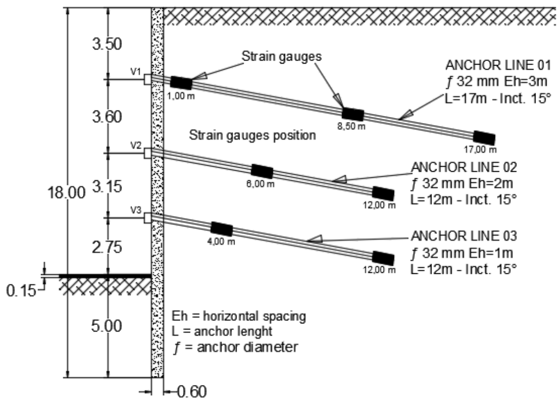
(c) Satellite view of the construction site

Figure 1 Location of the study at SHN (Google Earth, 2014)

The retaining structure consists of an anchored and juxtaposed pile wall of 18 m length, with piles of 60 cm in diameter and 5 m embedment length with reinforcement with three lines of passive anchors. Figure 2 depicts the main characteristics of the wall.



(a) Front view



(b) Cut view

Figure 2 Sections of the studied structure (Ruge, 2014)

As shown in Figure 2a, an inclinometer was installed with a distance of one meter from the top surface of the wall, in order to monitor the structure's displacements during the excavation phases, allowing verification of the work performance. The SPT test was performed in order to determine the site stratigraphy. Three distinctive layers were identified: from 0 to 6m; from 6 to 13m and then from 13 to 25m as shown in Figure 3.

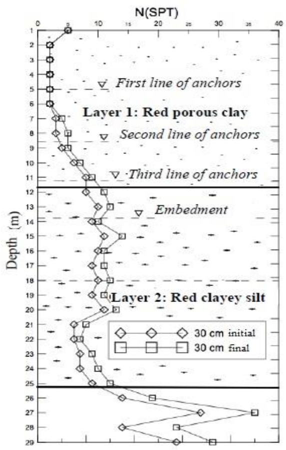


Figure 3 Typical profile of the SPT of the site

It is noticed that there was no use of a connecting beam on the top of the pile wall, as it is normal in cases like that. Moreover, the water level was not detected until a depth of 25 m below the level of the ground. It is also worth mentioning that the soil of Brasília is a tropical soil, structured, lateralized and normally found in the unsaturated condition, which justifies and allows the local practice of adopting a large value of clear spacing between adjacent piles when the soil is unsaturated. Such characteristics prevent soil loss or piping, given an existing arching effect. Besides, sprayed concrete between the piles is also applied to further protect the eventual loss of material.

3.1 Evaluation of Parameters

In order to characterize the soil and obtain its strength parameters, laboratory tests were carried out, such as drained and undrained triaxial and direct shear tests. The data obtained from these tests, were used to determinate the different parameters necessary for the computation of the tested models. Therefore, different combinations were used and then studied for the Sheeting Check module of the FINE software, as well as for elasto plastic models in the PLAXIS finite element software, employing different input parameters.

3.1.1 Consolidated Drained (CD) Triaxial Test

Triaxial tests in saturated and drained conditions were performed at 6 and 9m at three different confinement pressures (Figures 4a, 4b, 5a and 5b), respectively, 80, 200 and 400kPa; and 120, 200 and 400kPa.

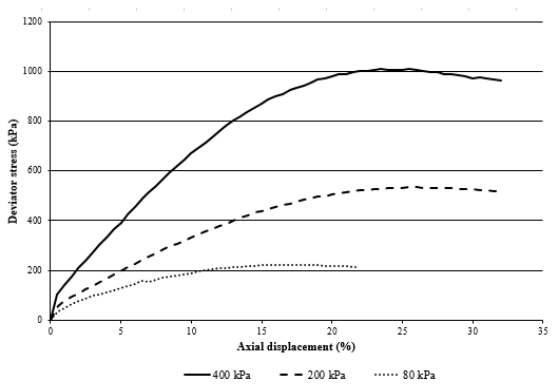


Figure 4a Deviator stress x Axial strain - 6m (Ruge, 2014)

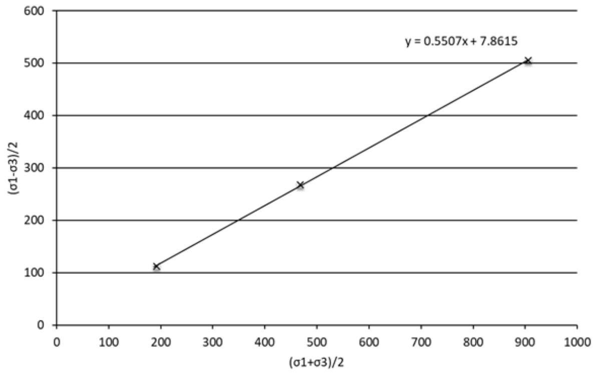


Figure 4b Shear stress x Effective normal stress - 6m (Ruge, 2014)

From the Figures 4b and 5b, the effective cohesion and friction angle were determined. Indeed, for 6m depth, the cohesion found was 7.8 kPa, which is quite low but can be justified by the sampling process or the manipulations and disturbance of the sample, and a friction angle of 22.84°. As for 9m depth, an effective cohesion of 32.66 kPa and effective friction angle of 25.55 ° were found. Using the tangent and the secant at 50% of the peak deviator stress allows to determine respectively the soil modulus tangent (Ei) and secant (E_{50}) for each confinement pressure, which has enabled to obtain soil modulus averages. Thus, for 6m depth, the soil modulus were defined as Ei_{av} = 2312.6 kPa and E_{50av} = 1709.0 9 kPa. For 9 m depth, it was verified that Ei_{av} = 6367.8 kPa and E_{50av} = 5180.2 kPa. Those values seem to be reliable as the average range of the soil modulus of Brasilia is situated between 1 and 8 MPa according to experimental data gathered from Cunha et al. (1999).

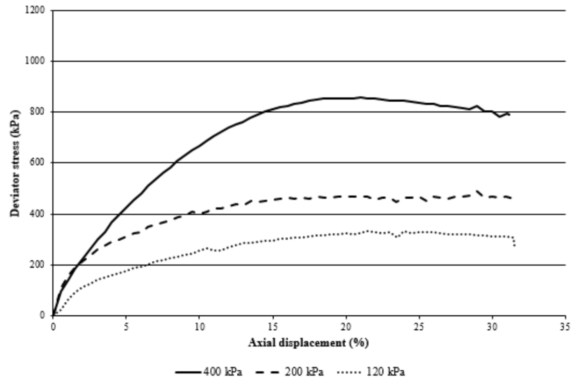


Figure 5a Deviator stress x Axial strain - 9m (Ruge, 2014)

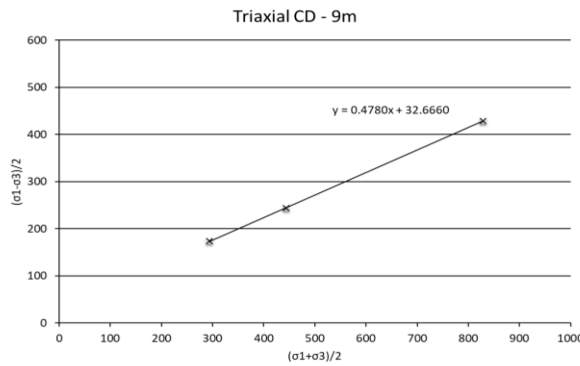


Figure 5b Shear stress x Effective normal stress - 9m (Ruge, 2014)

3.1.2 Consolidated Undrained (CU) Triaxial Test

Triaxial tests in saturated and undrained conditions (Figures 6, 7 and 8) were performed at 3, 6 and 9m at three different confinement pressures, 100, 200 and 400kPa.

It was adopted the same methodology used for the triaxial drained condition tests to obtain the soil modulus tangent and secant in the 3, 6 and 9 m of depth.

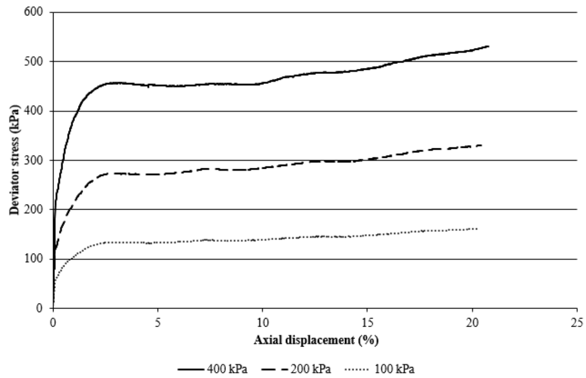


Figure 6 Undrained triaxial test for 3m depth

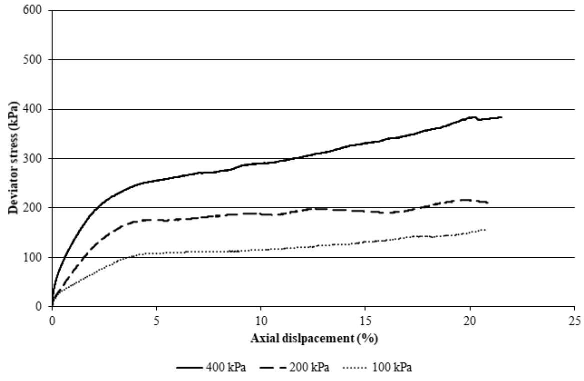


Figure 7 Undrained triaxial test for 6m depth

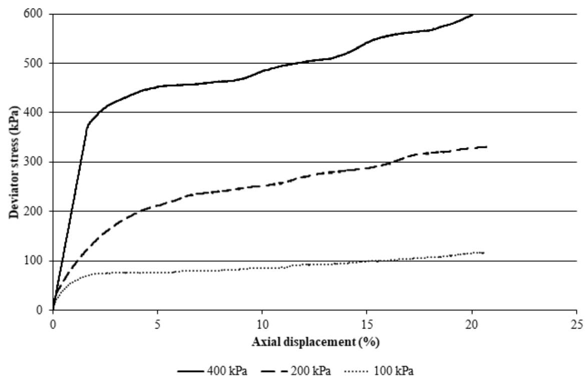


Figure 8 Undrained triaxial test for 9m depth (Ruge, 2014)

3.1.3 Direct Shear Test (DST)

Three direct shear tests were performed with a normal stress of 100, 200 and 400 kPa at depths of 3, 6 and 9m respectively. Figures 9, 10 and 11 present the cohesions and the friction angles of the samples obtained using the Mohr Coulomb equation.

All parameters obtained by the laboratory tests are summarized in Table 2.

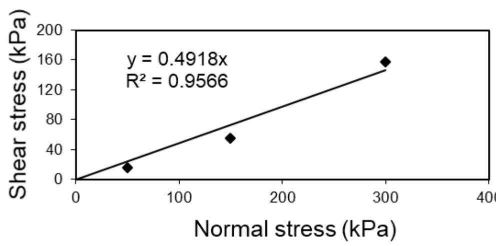


Figure 9 Direct shear test for 3m depth (Ruge, 2014)

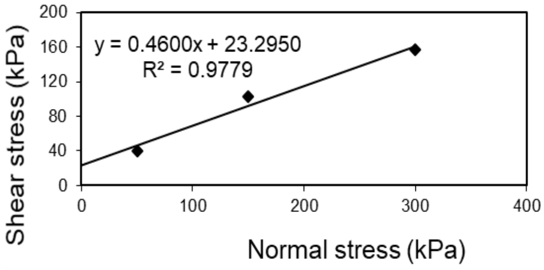


Figure 10 Direct shear test for 6m depth (Ruge, 2014)

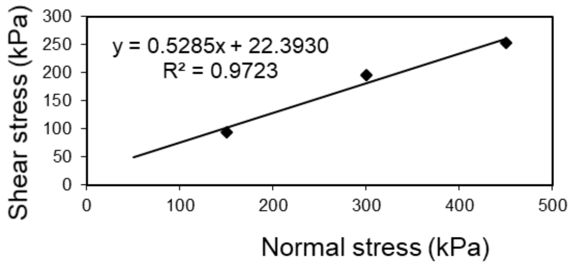


Figure 11 Direct shear test for 9m depth (Ruge, 2014)

Table 2 Soil parameters obtained from laboratory tests for 3, 6 and 9m depths

Depth (m)		3	6	9
c' (kPa)	Triaxial CD	--	8	33
	Triaxial CU	32	24	9
	Direct Shear	0	23	22
ϕ` (º)	Triaxial CD	--	29	26
	Direct Shear	26	25	28
Eiav (kPa) Triaxial CD		--	2313	6368
E50av (kPa) Triaxial CD		--	1709	5180
Eiav (kPa) Triaxial CU		7852	3457	3228
E50av (kPa) Triaxial CU		2243	2885	4485

In order to obtain representative parameters of the studied site, an average of all laboratory test parameters was considered. So, taking the three mean values of each parameter calculated for 3, 6 and 9 m of depth, linear functions were generated to determine their variation along depth. The coefficients of these linear functions were used to extrapolate the parameters to the other conditions. These extrapolations were performed considering three distinct layers that were identified by SPT tests, and hence provided the parameters presented in Table 3.

Table 3 Final parameters obtained for studied field

Depth (m)	c' (kPa)	ϕ' (°)	Ei (kPa)	E50 (kPa)
3	17	29	5329	2270
6	18	28	2828	2264
9	21	30	4703	4764
13	21	30	4798	4833
18	21	30	4798	4833

3.1.4 Estimation of the Subgrade Reaction Coefficient

It is known that the lateral/horizontal subgrade reaction coeff. is a conceptual relationship between soil pressure and deflection of the lateral soil surface. The calculation of the lateral subgrade reaction modulus of the soil (k_S) was done using the parameters obtained earlier and the rational/empirical methods proposed by Bowles (1988), using the bearing capacity factor including shape and depth effects proposed by Hansen (1970) and Meyerhof (1976), Glick (1948) and by “doubling” the original k_S equations for shallow subgrade reaction coeff. The coefficients were calculated from all laboratory data at each depth (Table 4) using the same previously methodology, in order to generate linear functions to determine the variation of the subgrade reaction coefficients along the depth.

It was observed that the coefficients obtained from the Bowles (1988) using Hansen (1970) and Meyerhof (1976) were 10 times larger than those obtained from the other methods.

Table 4 Final lateral subgrade moduli obtained for the studied field

Depth (m)	Ks (kPa)					
	Hansen (1970)	Meyerhof (1976)	Bowles Ei (1988)	Bowles E50 (1988)	Glick Ei (1948)	Glick E50 (1948)
3	59689	271810	5302	2947	4074	2265
6	77042	435880	3644	3019	2828	2264
9	127042	735880	5072	6352	4703	4764
13	193708	1135880	6237	6283	4793	4828
18	235114	1170000	6237	6283	4793	4828

3.2 Instrumentation Results

Instrumentation was installed with a servo accelerometer inclinometer probe located at one meter in the upper surface of the retaining wall, with the aim of monitoring the displacements of the retaining wall during the excavation phases. The readings made during the various phases are presented in Figure 12.

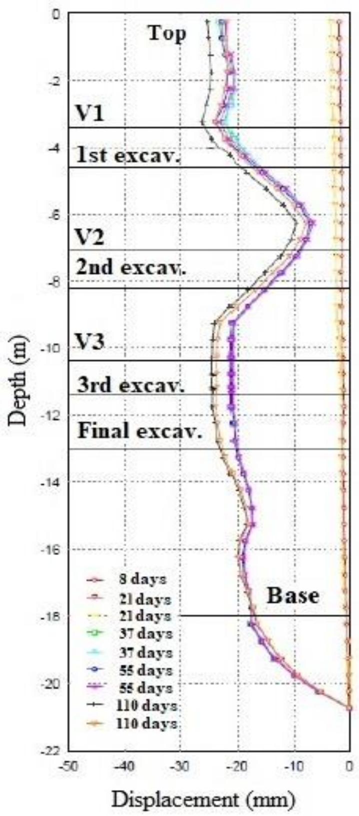


Figure 12 Results of the inclinometer (Ruge, 2014)

It was observed that the measurement made on the eighth day, one week after the first excavation of 4.50 m (one meter below the installation of the wall´s first anchor line), generated displacements of 2.50 mm. After the first excavation was completed, the first row of passive anchors was installed in the pile curtain, whereby the displacements stabilized until the beginning of the second excavation. The measurement performed on the twenty-first day after the second excavation showed minimum displacements of 4mm that maintained the trend obtained in the first reading. From the thirty-seventh day measurement, the behavior of the horizontal displacements revealed that maximum relative values during the executive process near the top of the curtain increased to approximately 22 mm. It is believed that the new excavation and the heavy rains that occurred during this period influenced the displacements. The maximum displacement measured at the top of the pile and after the last excavation was around 20.7 mm. Two weeks after this last excavation the displacement was stabilized at 24 mm. This lateral displacement is a weighted average deflection and it is not enough to compromise the behaviour of the retaining structure, especially because of the use of distinct anchored beams at different levels.

It is worth mentioning that, in the depth range between 4 and 8 m, the measurements presented a different behavior from the rest. One justification for the occurrence of these atypical values is the high sensitivity of the inclinometer sensor at any change in the inclination of the structure, and a possible local heterogeneity of the soil and structure at this particular position.

4. PREDICTION OF THE WALL´S DISPLACEMENT

The construction process has been simulated in 10 steps for Plaxis and Sheeting Check´s Fine tools, and are summarized in Table 5.

Table 5 Numerical stages of the problem

Steps	Description
01	Geostatic conditions
02	Pile execution
03	30 kN/m² overburden application
04	First excavation (until 4.50 m depth)
05	First line of soil nails installation
06	Second excavation (until 8.10 m depth)
07	Second line of soil nails installation
08	Third excavation (until 11.25 m depth)
09	Third line of soil nails installation
10	Fourth excavation (until 13.00 m depth)

4.1 Simulation with Fine Commercial Software

The Sheeting Check module of the Geofine commercial software (www.geofine.com) employs the method of dependent pressures. The basic assumption of this method is that the material (soil or rock) in the vicinity of retaining wall behaves as an ideally elastic-plastic winkler material, with “winkler springs”. The behaviour of this material is determined by its modulus of reaction, which characterizes and limits the deformation in the elastic region (assuming it as an idealized spring). When exceeding these deformations, the material behaves as ideally plastic. In particular, the loading due to earth pressure corresponds to the deformation of the structure, which is simulated by beam elements into a 1D finite element procedure. It further allows the verification of anchorage system internal stability. In this latter case, the anchor is also simulated by a unique winkler spring, which stiffness is calculated with its free length value and steel characteristics (modulus and area).

For simulation in the Sheeting Check module, geometric parameters were initially inserted in the model. The wall was constructed with a pile wall of 18 m in length, 0.60 m of diameter and 1 m spacing between piles. Then, based on the profile provided by SPT test, the terrain was divided into three layers: from 0 to 6 m, 6m to 13 m and then 13 m to 18 m. The construction process was simulated in 10 steps, considering three excavation stages and three rows of nails as shown in Figure 13.

Concerning the passive anchors, their properties should consider the contact between the steel of the anchor, plus the grout and the soil in order to input the modulus of elasticity of the system. Therefore, the modulus of the anchor should equal the steel-grout composite modulus. Thus, the steel-grout composite modulus was calculated as a weighted average between the steel and grout areas and their respective modules in relation to the total hole excavation area of 100mm in diameter. Finally, the input of the soil parameters was performed using the methods proposed by Bowles (1988) adopting the bearing capacity factors proposed by Hansen (1970) and Meyerhof (1976), Glick (1948) and Bowles (1988) for shallow subgrade reaction coefficient in order to compare the results of the model and the inclinometer measurements.

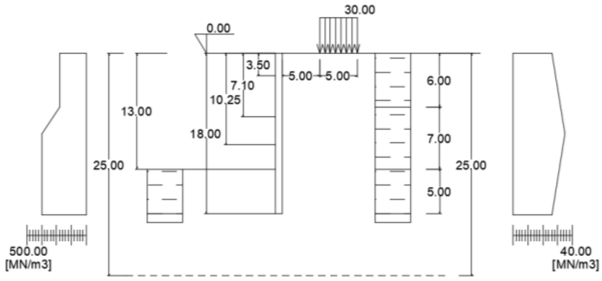


Figure 13 Ultimate stage of construction for Sheeting Check simulation

4.2 Simulation with Plaxis commercial Software

The numerical analyzes by the Finite Element Method were developed in the Plaxis software in order to determine the horizontal displacements of the analyzed curtain. As the evaluated section has a relatively large longitudinal length (around 100 m), and the instrumented zone was in between that, it was assumed that the 3D effects of the corners would not affect the estimation results. Therefore, the simulations were developed considering a problem of 2D plane strain deformation. The boundaries of the problem were considered sufficiently large to avoid the influence of boundary constraints, 50 m wide and 50 m high. Standard boundary conditions of the Plaxis software were applied in the analyses, that is, free ends only in the vertical direction and fixed base in both directions. The simulations were done for drained conditions, adopting the well-known constitutive models Mohr-Coulomb and Hardening Soil to represent the behavior of the ground layers of the terrain. Tables 6 and 7 summarize the main adopted geotechnical parameters.

Table 6 Soil parameters for Mohr-Coulomb model

Parameters	Layer 01 (up to 6m)	Layer 02 (between 6 and 12 m)	Layer 03 (below 12m)
γ (kN/m³)	14.70	14.70	14.70
c' (kPa)	17	20	21
ϕ' (°)	29	30	30
ψ (°)	0	0	0
E (kPa)	2270	4764	4870
ν	0.33	0.33	0.33

Table 7 Soil parameters for Hardening Soil model

Parameters	Layer 01 (up to 6m)	Layer 02 (between 6 and 12 m)	Layer 03 (below 12m)
γ (kN/m³)	14.70	14.70	14.70
c' (kPa)	17	20	21
ϕ' (°)	29	30	30
ψ (°)	0	0	0
E_{50} (kPa)	2270	4764	4870
E_{oed} (kPa)	2270	4764	4870
E_{ur} (kPa)	6810	14292	14610
ν_{ur}	0.20	0.20	0.20
p_{ref} (kPa)	100	100	100
M	1.0	1.0	1.0
K_0	0.52	0.50	0.50
R_f	0.90	0.90	0.90

The pile curtain wall was defined as a plate element and the anchors as bar elements. The properties of the piles and anchors are presented in Table 8. In the present work, the curtain weight was calculated taking into account the columns of reinforced soil existing between the piles. Due to the two-dimensional characteristic of the analyses, it was not possible to directly evaluate the spacing between anchors of the same horizontal line. Therefore, the axial and bending rigidities of them were averaged, i.e., divided by the total horizontal spacing (per meter). It is also worth mentioning that, when computing the anchor´s axial rigidity, the equivalent modulus of elasticity was determined accounting for the contribution of both elastic stiffness of steel and grout (differently to what was done in the Geofine program).

Table 8 Elastic properties of curtain and anchors

Pile Curtain (plate element + elastic model)	
Axial rigidity (kN/m)	7.07E+09
Bend rigidity (kN.m²/m)	1.59E+05
Weight (kN/m)	4.98
Poisson Coefficient	0.20
Anchor (bar element + elastic model)	
Axial rigidity 01 (kN/m)	8.20E+04
Axial rigidity 02 (kN/m)	1.20E+05
Axial rigidity 03 (kN/m)	2.50E+05

The final configuration of the problem is illustrated in Figure 14. A distributed overburden of 30kN/m², corresponding to an upstream construction on the wall, was considered as acting on the top surface of the excavation site. The simulations were carried out by trying to evaluate all the constructive processes of the work (Table 5).

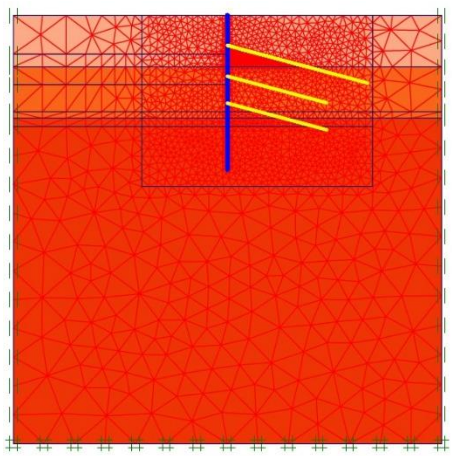


Figure 14 2D Plaxis finite element model

5. RESULTS AND DISCUSSION

This section deals with the accuracy of the predictions of the methodologies used in relation to the geotechnical instrumentation data.

5.1 Subgrade Reaction Coeff. with Winkler Approach

Using the four methods of calculation of the horizontal subgrade reaction coefficient of the soil, the results of the displacements at the top of the pile are shown in the Table 9 and Figure 15.

It was observed that the method of Bowles (1988) adopting the bearing capacity factors proposed by Hansen (1970) was the most accurate, knowing that the displacement at the top of the pile observed after the last excavation was 20.70 mm. Moreover, the deformations aspects of the simulation presented by Figure 15 showed that the predicted curve presents great similarity to the one obtained from the inclinometer, which validates the accuracy of this method. On the other hand, the bearing capacity factors proposed by Meyerhof (1976) did not reproduce the reality. It is assumed that this is due to the fact that very high coefficients of reaction were found by using this empirical method. Indeed, the method is based on an approximate assessment of the Meyerhof´s graphic technique and some interpretation errors can allow for the obtained high coefficient values. Concerning Glick (1948) and Bowles (1988) methods, overestimation of the horizontal displacements of the pile curtain wall were observed. This is explained by the fact that the coefficients found using these methods were probably very low.

Table 9 Displacement at the top of the pile curtain wall

Method	Description	Displacement (mm)
Hansen (1970)	--	24.66
Meyerhof (1976)	--	26.14
Bowles (1988)	Using Ei	44.22
Bowles (1988)	Using E50	47.70
Glick (1948)	Using Ei	48.04
Glick (1948)	Using E50	53.30

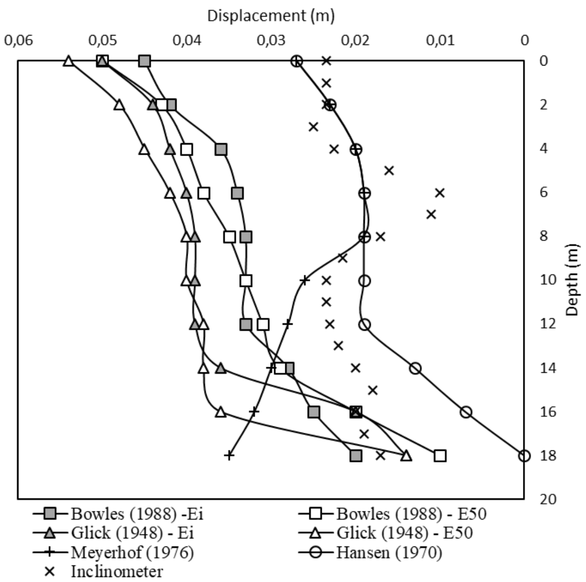


Figure 15 Horizontal displacements values of the pile curtain wall on the Sheeting Check software

It was observed that the method of Bowles (1988) adopting the bearing capacity factors proposed by Hansen (1970) was the most accurate, knowing that the displacement at the top of the pile observed after the last excavation was 20.70 mm. Moreover, the deformations aspects of the simulation presented by Figure 15 showed that the predicted curve presents great similarity to the one obtained from the inclinometer, which validates the accuracy of this method. On the other hand, the bearing capacity factors proposed by Meyerhof (1976) did not reproduce the reality. It is assumed that this is due to the fact that very high coefficients of reaction were found by using this empirical method. Indeed, the method is based on an approximate assessment of the Meyerhof´s graphic technique and some interpretation errors can allow for the obtained high coefficient values. Concerning Glick (1948) and Bowles (1988) methods, overestimation of the horizontal displacements of the pile curtain wall were observed. This is explained by the fact that the coefficients found using these methods were probably very low.

5.2 Finite Element Method Using Mohr-Coulomb Model

Figure 16 presents the displacements given by the Mohr-Coulomb model after the last excavation sequence. The results do not seem to be realistic though. Indeed, it is observed that pile displacements are much greater at the bottom of the curtain wall rather than on the top of it, allowing a perception that the modelling was unrealistic.

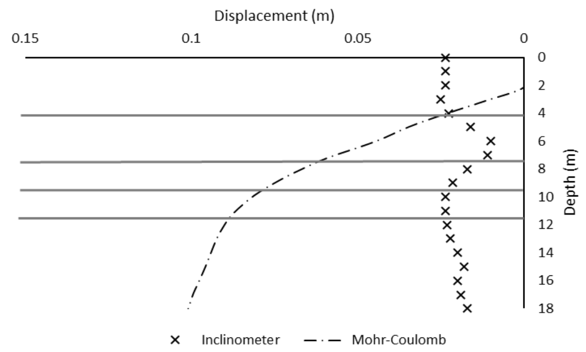


Figure 16 Horizontal displacements values - Mohr-Coulomb model

Indeed, the Mohr-Coulomb model presents some limitations, such as the overestimation over bottom heave, or heave of soil behind the wall, or the fact that excavations occasionally widen spontaneously (even with anchors). It can be numerically noticed some heave of soil behind the wall, and excavation´s expansion. This is due in part perhaps to the fact that this model is very conservative, i.e., the linear elastic perfectly plastic Mohr-Coulomb model is a first order model that includes only limited number of features that the soil behavior shows, and does not present the same behavior of stress-strain deformations as the real phenomena. The geotechnical problems must be analysed according to the type of deformations that occur, for example, for the foundation of a machine, an elastic model is enough to reproduce the soil behavior. However, a retaining structure is a complex geotechnical environment to be simulated, since various points of the structure behave differently on the stress-strain curve. For instance, in Figure 17, the point A of the real behavior of a soil mass, plotted on the stress-strain curve (excavation in active condition), represents a place where the soil is still far from yielding; at point B the soil is close to yield, and point C has already reached a softening stage.

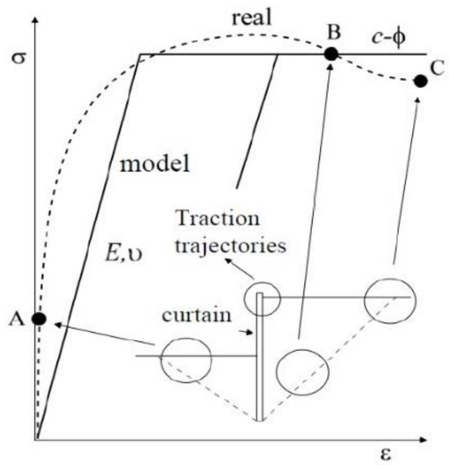


Figure 17 Stress-strain points on a typical soil behavior curve (Ruge, 2014)

The curve that reproduces the response of the Mohr-Coulomb model can hardly simulate "real" soil behavior at these points. This by both the choice of the constitutive model, which is not only an issue of simplicity and speed in geotechnical design, but it is also subject to a good judgment and experience of the engineer, in order to choose the parameters that adequately represent the range of deformations in a wise manner. Unfortunately, most of cases it ends in overdesigned geotechnical structures. For this reason, the simplest models require an experienced designer to select appropriate values to calibrate the input parameters of the evaluated problem. In contrast, more complex models that are calibrated appropriately with laboratory tests, and show a more realistic soil response, can perhaps better represent the behavior of the structure with respect to the range of deformations in different places along the profile.

5.3 Finite Element Method Using Hardening Soil Model

For the Hardening Soil model, Figure 18 presents the curtain wall displacements after the last excavation.

It is observed that the pile behavior seems to be closer to the "reality". Indeed, it is verified that the displacement is greater at the top of pile curtain wall. The advantage of the Hardening Soil model over the Mohr-Coulomb model is not only the use of a hyperbolic stress-strain curve instead of a bi-linear curve, but also the control of the stress level dependency. When using the Mohr-Coulomb model, a single and fixed value of Young's modulus is employed. However, for real soils this stiffness depends on the stress level. It is therefore necessary to estimate the stress levels within the soil and to use these to obtain suitable values of stiffnesses.

Although the Hardening Soil model gives the same failure as the Mohr-Coulomb analysis (since it uses the same failure criterion) it gives better prediction of displacements, especially when shear is dominant and unloading/reloading behavior is important. Therefore, for excavation problems, the Hardening Soil model is (theoretically) more recommended over the Mohr-Coulomb model.

In other words, the Hardening Soil model gives better "qualitative realistic deformations" than the Mohr-Coulomb model. Indeed, it presents a better bottom heave, but also shows an increase along the profile´s depth; It can be seen, as well that the displacements given by the Hardening Soil model are 3 times bigger than the experimental ones, even though the behavior is similar. This is due to the fact that the suction effects on the likely increase in soil rigidity were not considered, and besides average parameters were adopted. However, it is difficult to determine the soil parameters appropriately, even if laboratory tests can be made available. Firstly, because it is not known if the field subsoil will behave the same way as the sample, then, because it is also not known if the laboratory tests are indeed reliable or not (differences in stress paths, average consolidation stresses, disturbance, heterogeneity in the field etc.). Finally given natural simplifications of the numerical procedures. Moreover one must consider the intrinsic variability of the data, that causes some dispersion in the range of each one of these intrinsic input parameters. Some of these aspects, and difficulties, are didactically shown by Ruge et al (2018).

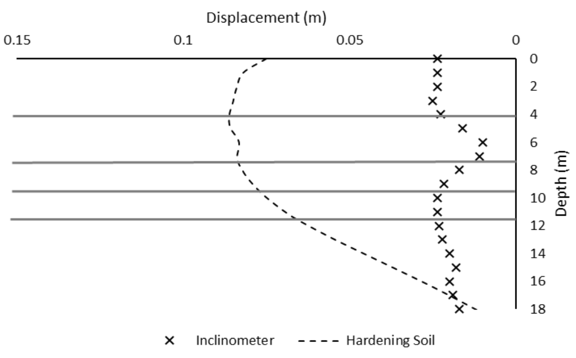


Figure 18 Horizontal displacements values - Hardening soil model

6. CONCLUSIONS

This paper presented an analysis of a pile curtain type retaining wall constructed with three lines of passive anchors, and founded in the tropical, collapsible and unsaturated typical soil of the Federal District of Brazil, most precisely in the city of Brasilia. The analyses were done by using existing commercial finite element and Winkler type softwares.

In order to verify the accuracy of the models with respect to the behavior of the pile curtain, their predictions were compared with the (actual) observed displacements of the retaining structure, from the single available inclinometer installed in the site, close to the wall. The parameters of the models were obtained from laboratory experiments, such as drained and undrained triaxial tests and direct shear tests. All the tests are related to the same depth, at the same place, and carried out with the same (field / lab) methodology. On the other hand, simple winkler type analyses were additionally done with the horizontal subgrade reaction coefficient of the soil, that has been evaluated from existing empirical techniques. The use of average input values was necessary in order to have "consistent" design parameters of the soil, and to handle the numerical techniques (finite element vs winkler type). Nevertheless, several influential external factors for such parameters, existing in the site, could not be taken on account, as for instance the effect of suction on the likely increase in the soil´s rigidity.

In terms of the numerical approach by using a winkler type software the best technique to evaluate the reaction modulus was an empirical equation presented by Bowles (1988) adopting the bearing capacity factors proposed by Hansen (1970) to calculate the coefficient of subgrade reaction. For the numerical predictions with the finite element method, both models did not consistently assess the displacements, although the hardening soil model seems to have presented a slightly better simulated displacement shape than the Mohr-Coulomb model. Nevertheless, in both cases the absolute displacements were overestimated, allowing the conclusion that the design experience of the engineer is a key aspect, and an important criterion, to define the success of those types of analyses.

The final conclusion, and lesson learned from the presented (and limited) exercise, is that sometimes it is better to evaluate the performance of pile curtain walls with simple winkler type techniques rather than via complex F.E. programs, especially on complex tropical environments where all the influence factors on the input variables cannot be appropriately taken on account.

7. ACKNOWLEDGEMENTS

The success of this interactive research cooperation between distinct universities in Brazil, Colombia, France and Germany is a product of the support of many individuals and institutions, most of them unfortunately not listed herein given space restrictions.

Nevertheless, special acknowledgements shall be given to some of them, particularly the Brazilian sponsorship organizations CNPq, CAPES and FAP-DF and Profs. Pierre Breul at Blaise Pascal University and Lothar Schmidt at Munich Univ. of Applied Sciences.

Student Leila Carvalho has provided a providential help on some of the figures, which is most appreciated here.

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