Design of Retaining Structures in a Tropical Soil with the Use of the Marchetti Dilatometer

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ABSTRACT: This paper presents a case history of an instrumented pile curtain wall made by regularly spaced piles constructed to withstand a 11.5m high excavation in a tropical soil deposit. The wall was designed using conventional analytical methods, allowing it to be built with 4 anchorage beams. An alternative design approach for the wall was tested with the use of the Marchetti dilatometer test (DMT) through a recently defended Master thesis (Reyes 2012). The paper therefore presents the main results of this work and, although limited in its scope, highlights a new, or alternative approach, to design similar structures at such geotechnical conditions solely with the aid of the DMT. Numerically derived horizontal displacements were obtained and compare reasonably well to field surveys at inclinometer tubes, encouraging further application of this technique.

1 INTRODUCTION

Brasília is the Brazilian capital, and it is located in the vast Mid West area of the center of Brazil. It is a pre-designed city built in the early 60’s to house the main Governmental institutions and its public employees, and it suffers from a sheer expansion process that has accelerated since the last decades (from the 90´s). Several residential, administrative, industrial and overall purpose units are (or have been) under construction since then, demanding large excavation areas in the underground to keep in accordance with new building regulations that demand enough parking areas, or lots, in the subsoil. Indeed, vehicle space has become one of the major commodities in nowadays Brazilian cities.

In order to safely excavate the underground, with the aim to construct garage levels that can vary from 2 to more than 5 slabs, it is common to adopt retaining structures made of regularly spaced piles, sustained in place by anchored (active or passive) beams, with shotcrete and drainage tubes in between the piles (see for instance some details in Cunha et al. 2006). Off course, this is so given the particular characteristics of the “tropical, laterized, unsaturated, structured and porous” soil of this city, already described elsewhere (Cunha et al. 1999).

Given the necessity to better understand and to provide to the local practice some technical guidance lines based on well established research on this type of structure, the Geotechnical Program of the University of Brasília started in the last years of the last century applied investigations that focused on the behavior of these walls (for instance Medeiros 2005), or on the use of DMT in practical applications (for instance Jardim 1998 or Cunha and Mota 2000).

Therefore, this paper is another contribution to aforementioned line of knowledge. It presents full scale results of a retaining wall built in a relatively new and expanding residential district of Brasília. This particular wall was designed using conventional analytical methods, allowing it to be built with 4 anchorage beams (2 passive and 2 active) and 11.5 m in height. In order to study an alternative design method for the wall, and as part of an innovative Master research (Reyes 2012), DMT were executed in the soil before the excavation. The DMT results were used to furnish “p-y” (pressure vs displacement) curves, or variable horizontal subgrade reaction moduli (the inclination, given by “K_h” in kN/m^2), by a well-known technique proposed by Robertson et al. (1989) specifically envisaged for the DMT. With these curves at each depth, or the K_h defined moduli, it was possible to simulate the constructive sequence of the wall in a commercially available software that adopts a “winkler type” spring for the wall-soil interface, and coulomb's theory for earth pressures (Sheeting Check module from the Geofine package, Fine 2004). The results were then compared to horizontal displacements obtained via inclinometer measurements, allowing practical conclusions for the tested methodology.

2 SITE AND WALL CHARACTERISTICS

2.1 *Geographical and geotechnical characteristics of the construction site*

The construction site was located within a neighbourhood district of Brasília, just outside the main central area of the city (depicted in the left side of Fig.1 as a discontinuous circle). This district is denominated as Águas Claras, and it essentially comprises of a new & dynamic area under continuous expansion around 20 km from the capital centre. Several residential units are under development, and the site related to this paper is located just besides the main underground station of the district (as depicted in the right side of Fig. 1). The site is also marked as a star in this same figure.

Within the Federal District extensive areas (more than 80 % of the total area) are covered by a weathered latosoil of the tertiary-quaternary age. This latosoil has been extensively subjected to a laterization process and it presents a variable thickness throughout the District, varying from few centimeters to around 40 meters. There is a predominance of the clay mineral caulinite, and oxides and hydroxides of iron and aluminum. The variability of the characteristics of this latosoil depend on several factors, such as the topography, the vegetal cover, and the parent rock. In localized points of the Federal District, as in Águas Claras, the top latosoil overlays a saprolitic/residual soil with a strong anisotropic mechanical behavior, which is originated from a weathered, folded and foliate slate, the typical mother rock of the region.

The superficial latosoil is locally known as the Brasília "porous" clay, being geotechnically constituted by a sandy clay with traces of silt, forming a lateritic horizon of low unit weight and high void ratio, as well as an extremely high coefficient of collapse. These soil characteristics were already presented elsewhere (Cunha et al. 1999) and are not repeated here. In the particular conditions of the construction site, the porous clay also overlays a saturated (and soft) silty clay with undrained characteristics, a vestige from a geological "paleochannel" at the region. This latter stratum is followed by a compact sandy silt which, in turn, overlays a transition layer of a residual and structured sandy silt directly originated by the local mother rock.

Fig. 2 depicts the approximate geotechnical profile obtained with the use of several standard penetration tests (FS) carried out early at the site.

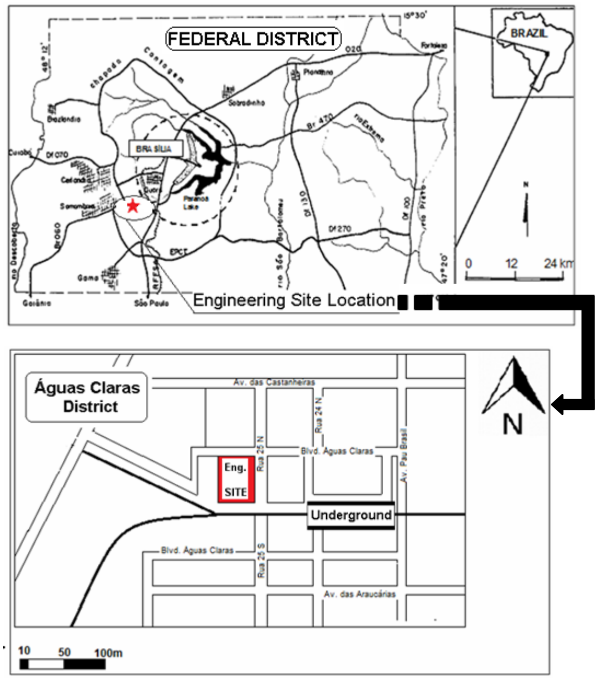

Fig. 1. Engineering site location within Federal District and Águas Claras district.

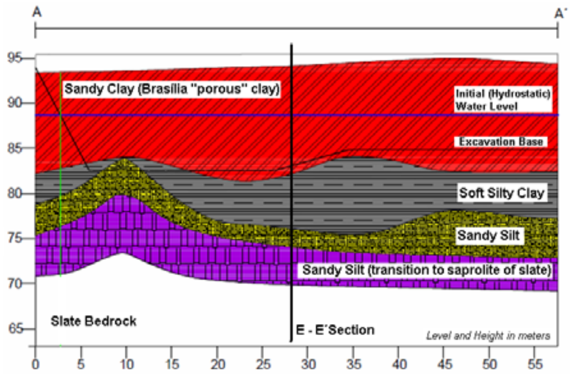

Fig. 2. Geotechnical profile of the front view (South Face and E-E´ section) of the curtain wall.

2.2 *Overall characteristics of the retaining wall*

The retaining wall constructed at the site was designed to withstand an excavation depth related to 4 underground levels, i.e., something around 12 m below ground surface. It was designed with 4 distinct faces, or front views, which orientations and individual outlines are described in accordance to the magnetic directions, from North to South. In fact, each view had its own characteristics given particular aspects as surcharge load, (underground) beam levels and neighborhood. Nevertheless, all faces were composed by equally spaced piles (in this case Continuous Flight Auger - CFA piles, similar to those used in the foundation of the edification) secured in place by either active or passive anchored beams. They were also designed under a temporary basis function, which means, once the excavation was done and underground columns, beams and slabs were constructed, the wall would be sequentially (bottom up) “locked” in place by the finished slabs, thus transferring horizontal loads from the anchors to the structure of the building. This allows the neighbors to eventually remove from subsoil the existing anchors executed in their own specific lots – adjacent to the site.

The overall characteristics of the construction in terms of both plant view and typical cross section at the region of interest (at South Face) are depicted through Fig.s 3a and b. Note that this face’s (front) view is perpendicular to the E-E´ section, as represented by Fig. 2. In this latter figure it is also possible to see the position of the section of interest (E-E´) where both DMT and inclinometer measurements were taken (see also Figs. 3a & b).

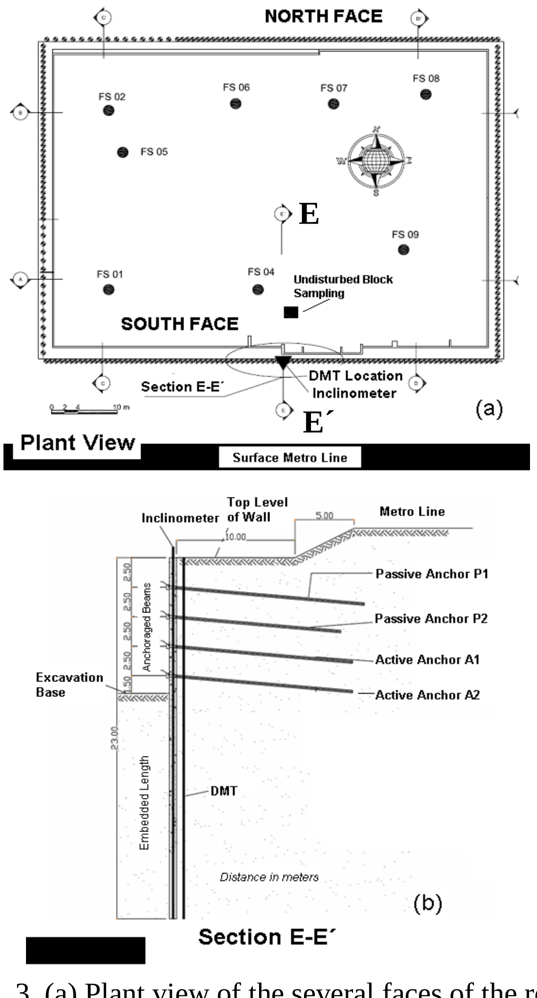

Fig. 3. (a) Plant view of the several faces of the retaining wall and (b) Cross section of studied region.

One also notices that this view was the one closer to the main underground Metro line of the district, which turns into surface line just after the main station (depicted in Fig. 1). Given this special feature (and associated risks), it was thoroughly studied and monitored, inclusive by geotechnical laboratory tests from undisturbed block samples retrieved at the site (location in Fig. 3a).

The South Face of the wall (at Section E-E´) was then constructed with the following characteristics:

- CFA piles of 60 cm diameter (ϕ) spaced at 75 cm from center to center, with overall length of 23 m in which 11.5 m relates to the excavation and the other 11.5 m to the embedment levels.
- Anchored beams with 2 top and 2 bottom levels respectively related to passive and active anchors. From top down it was used passive steel anchors of ϕ 32 mm each 2.25 m @ horizontal, respectively with 16 m (P1) and 14 m (P2) in length. Likewise, from top down it was used active INCO (Incotep Ltd.) 22D (A1) and 35D (A2) monobar steel anchors, respectively with incorporation loads of 150 and 280 kN each 1.5 m @ horizontal. All anchors are inclined at around 5° to horizontal.
- Excavation at sequential stage, i.e., at each excavated level (at the base of the beam) the anchoring system was constructed and incorporation loads applied (A1 & A2) before continuing to the next level. Hence 9 executive sequence stages were followed, from Oct. 5th 2006 to May 25th. 2007.
- Depleted water level from original conditions, given a sub-superficial dewatering system at the site. It was assumed that during working conditions the water level was around 9 m below ground surface.

3 ANALYSES AND RESULTS

3.1 *Software and parameters*

The construction site was located within a neighborhood district of Brasília, just outside the main central area of the city (depicted in the left side of Fig.1 as a discontinuous circle). This district is denominated as Águas Claras, and it essentially comprises of a new & dynamic area under continuous expansion around 20 km from the capital centre. Several residential units are under development, and the site related to this paper is located just besides the main underground station of the district (as depicted in the right side of Fig. 1). The site is also marked as a star in this same figure.

Numerical simulation of the wall construction was carried out with a commercially available software that adopts a “winkler type” (non linear or linear) spring for the wall-soil interface, and Coulomb’s theory for earth pressures. It also allows the incorporation of passive or active (pre stressed) anchors, as a linear spring with particular values of elastic modulus, cross sectional area and length. The software is a particular module (Sheeting Check) of an overall geotechnical package (Geofine, Fine 2004), that allows an interactive routine to obtain displacements, moments and shear forces into idealized nodes of the structure.

The actual analysis is basically carried out using the deformation variant of the finite element method, in which equilibrium of stresses are obtained at each node once the initial stress regime (at rest) and the excavation process is imposed. The analysis accounts for the construction process such as individual stages of progressive construction of the wall, including gradual evolution of deformations and eventual post-stressing of anchors. Stresses at each node are bounded by active and passive limits, being estimated by a given (input) relationship between pressure vs. lateral displacement, i.e., the "p-y" curve that is normally associated to "winkler type" non linear spring programs.

In the present series of simulations "p-y" curves were obtained for each soil strata in accordance to individual soil parameters and characteristics, i.e., drained (via friction angle φ´) or undrained (via shear strength Su). All parameters and the curves were based on DMT data interpretation (Robertson et al. 1989 for "p-y" curves, Marchetti 1980 and Marchetti & Crapps 1981 respectively for φ´ and Su).

The use of the Robertson et al. (1989) methodology to derive the "p-y" curves is well justified by previous experience published by Marchetti et al. (1991). These latter authors have concluded that the Robertson et al. (1989) method "has solved for good the problem of the link between p-y curves and DMT data", predicting therefore pile (lateral) displacements with "amazing accuracy".

Thus, with the derived "p-y" curves, it was possible to obtain at each layer the horizontal subgrade reaction modulus K_h (in MN/m^2), or per unit length k_h (in MN/m^3) once divided by pile diameter. Coulomb pressures were also calculated with the use of the cohesion parameter (c´) in drained layers, defined by a specific DMT correlation (Cruz et al. 2004). The k_h modulus was derived at particular positions of the curve, given 4 distinct methodologies tested in this phase: The CUR166 methodology that constructs its own pressure-displacement curve from k_h at 50, 80 and 100% of ultimate pressure (CUR 2005); An internal "own" curve (by Geofine) with 40, 60 and 90% of pressure; And constant values through length of k_h at 40 and 50% of maximum pressure.

In a step-by-step manner, the determination of the input parameters for the Geofine software can be detailed as follows:

- Division of the soil strata (see Fig. 2) in distinct layers of drained or undrained behaviour. For each of them, selected "average" values of drained (friction angle φ´ and cohesion c´) and undrained (und. shear strength Su) parameters are determined, by using DMT correlations (as cited before) and field DMT results. The results are shown in Table 1.
- For each layer, as previously determined, a unique cubic parabolic p-y curve for strain hardening soil was constructed (same curve as Fig. 2 from Roberton et al. 1989 paper). This curve simply relates a ratio of soil resistance (P/Pu) by a ratio of pile deflection (y/yc).
- In the construction of the unique P/Pu x y/yc curve for each layer, one had to determine the ultimate soil resistance Pu and the corresponding deflection yc. In this regard, if the layer is drained, values of Pu and yc were respectively determined by using Equations (10,11) and (15) from Robertson et al. (1989) paper. If the layer is undrained, values of Pu and yc were respectively determined by using Equations (8) and (7) also from Robertson et al. (1989) paper. In either drained or undrained layers, constant values of DMT derived field E_D (any case) and interpreted φ´ and Su parameters (depending on layer behaviour, averaged as previous commented) were adopted for the adopted equations.
- In the calculations of the Pu and yc variables from the previous item one must notice that, in the present paper, it was adopted the same suggestions from Robertson et al. (1989) for the values of all required empirical coefficients (N_P, F_C and F_ϕ) of the equations. Besides, a constant value of effective vertical stress at the mid height of each layer was adopted (as required by some of them).
- Once the unique cubic P/Pu x y/yc is constructed for each layer, a single P x y curve for this same layer (kN/m vs. m) is derived, by using the known values of Pu (kN/m) and yc (m) previously derived in the previous items.
- With this unique P x y curve at each layer, values of the horizontal subgrade reaction modulus K_h (in MN/m^2), and the modulus per unit length k_h (in MN/m^3) are defined at particular points of the curve – simply by respectively calculating P/y or P/y versus D (pile diameter).
- Since Geofine internally derives the required non linear or linear p-y curves, based on previously cited methodologies, an extra procedure was made for each layer. In other to "characterize" and run this software it is required as input value the modulus per unit length k_h (in MN/m^3). Once furnished, the software calculates internally the "p-y" curve for the

layer, and runs the analysis. Therefore, for each specific layer, a table of values of k_h at distinct levels of the ultimate soil resistance Pu were determined. For instance, for the CUR (2005) method, one has to furnish k_h values at 50, 80 and 100% of the ultimate resistance, or pressure, Pu. These values were then calculated at such specific points of the P x y curve, as cited in the previous item. The same was done for the other adopted methods (the "own" Geofine method and the method with constant values of k_h). Table 2 presents all adopted values of k_h, used in each particular methodology and valid at each specific layer.

- Finally, once Tables 1 and 2 were constructed, it was easy to run each analysis, simply by adopting at each layer appropriate values of geotechnical strength parameters (including the drained cohesion c´, employed for pressure calculations), and k_h (depending on the methodology for the internal construction of the p-y curve). Notice that once a particular methodology is adopted, as the CUR (2005) for instance, all the layers have equivalent values of input k_h values (in this case, derived at 50, 80 and 100% of Pu).
- Some final remarks shall be made regarding the analysis: It was done sequentially, trying to simulate the best as possible both field excavation sequence and anchor execution. For all anchors it was adopted a Young modulus of 210000 MPa, valid for the steel, and a nominal bar diameter that did not considered the cement grout around it. Besides, for the passive anchors, the pre load after numerical insertion was zero, whereas for the active ones a correct value of field incorporation load was inputted into analyses. Corrected field values of free and bonded length were adopted for each anchor.

3.2 *Results and discussion*

As previously stated, 9 executive sequence stages were adopted into numerical analyses. from Oct. 5th 2006 to May 25th. 2007, which respectively correspond to the beginning of the excavation process (terrain at ground surface) and the last stage of excavation (after anchored beam A2 was constructed and loaded). It also represents the dates of the first (baselines) and the last readings of the inclinometer. Unfortunately only 3 measurements were taken (one extra transitional reading at March 5th. 2007. Reyes 2012), which have not allowed the comparison of displacements throughout the intermediate stages of the excavation process.

Fig. 4 therefore presents the comparison between the experimental readings at the final excavation stage. and the numerical predictions using linear or non linear soil springs in accordance to aforementioned methods. It is clearly noticeable that reasonable comparative results were obtained for all methodologies, with perhaps a "slight" better accuracy for the CUR 166 technique. Nevertheless, in engineering terms. one can consider all results well acceptable for practical purposes.

Table 1. Adopted geotechnical parameters in each strata of Section E-E' for the numerical analyses

Soil Strata	Depth Level (m)	Strength Parameters		
		φ (°)	Su (kPa)	c´ (kPa)
Sandy Clay	0 to 9	32.5	0	20.4
	9 to 12	32.5	0	20.4
Soft Silt Clay	12 to 17	0	69.4	0
Sandy Silt	17 to 21	34.8	0	32.9
Sandy Silt (trans)	> 21	34.8	0	23.0

Table 2. Adopted k_h in each strata of Section E-E' for the numerical analyses

Soil Strata	Horizontal Modulus of Subgrade Reaction per unit length k_h (MN/m^3) (% of Total Stress Level)					
	40%	50%	60%	80%	90%	100%
Sandy Clay	70.3	49.0	32.5	19.7	15.1	12.3
	61.1	43.9	34.6	19.7	15.2	12.2
Soft Silt Clay	70.9	46.6	34.0	19.6	15.3	12.8
Sandy Silt	63.6	44.1	34.7	19.8	15.4	12.6
Sandy Silt (trans)	63.6	44.1	34.7	19.8	15.4	12.6

The good agreement, and the quite remarkable comparison between curves throughout excavation and embedment depths, were not expected beforehand. Indeed it came as a surprise, which has encouraged the Research Group leaded by the first author to further explore[1] this technique in other instrumented sites of the Federal District where DMT is also available.

[1] A French student from Clermont Ferrand University (Sarah Jaccaz) worked on a related subject for a final Engineering project in 2014, through a collaborative exchange program between this Institution and the University of Brasília.

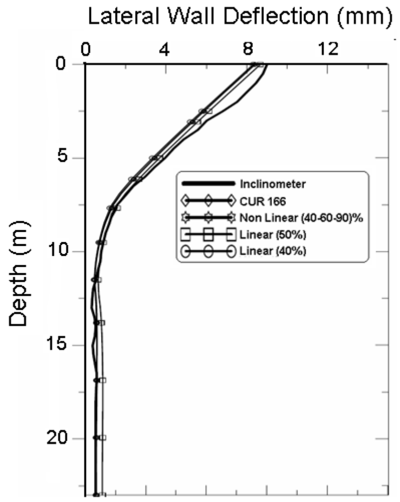

Fig. 4. Comparison of experimental and numerical horizontal displacements for final stage

4 CONCLUSIONS

The paper has highlighted the use of DMT derived parameters and “p-y” curves, initially developed for laterally loaded piles, for use into a new interpretation methodology for retaining walls executed in tropical soil deposits. Although initially valid, and “calibrated”, for the particular conditions of the Federal District of Brazil where it is common to construct pile curtain walls equally spaced, the methodology clearly has the potential to be adapted anywhere, from sedimentary soil deposits to other types of retaining structures (as diaphragm walls).

Although encouraging results have been obtained, demonstrating that the methodology can be someday largely used in practice (“aggregating” value and promoting the use of DMT into tropical regions of Brazil), caution is required. More research on the subject is needed, especially with data from other construction sites, and dissimilar execution procedures as those from the Brazilian capital. Efforts in this direction are underway by the Geotechnical Program of the University of Brasília.

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