Design and construction of the substructure of a commercial development in Bogotá City using a Top-Down sequence

Dimensionnement et construction

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ABSTRACT: This paper presents the design, construction and back analysis of the basement retaining wall of a major commercial development in Bogota, Colombia. The development consists of 4 eleven-story office towers, and 4 basements for car parking use, occupying the entire venue area (18,000 m² per basement). A permanent structural diaphragm wall allowing the construction of the slabs in a Top-Down sequence was designed for a maximum depth of excavation of 15.0m. The ground profile mainly consists of very soft clay with natural water content up to 200%. An important number of in-situ shear vane tests and laboratory tests were performed as part of an additional ground investigation in order to have more accuracy in the undrained shear strength of the clays. The project is heavily instrumented with inclinometers, piezometers, extensometers and topographical survey. The excavation is complete and the theoretical results obtained during the design stage are compared with the instrumentation measurements. This back analysis was used in the form of adjusted sets of soil parameters for traditional elasto-plastic modelling and finite element modelling of the excavation sequence.

RÉSUMÉ : Cet article présente le dimensionnement, la réalisation et rétro-analyse d'un soutènement pour plusieurs niveaux de sous-sol d'un complexe commercial à Bogota, Colombie. Le projet vise à construire quatre tours de 11 étages et 4 niveaux de parking chacun s'étendant sur une surface de 18000 m². L'excavation se fait en taupe s'appuyant sur la paroi moulée définitive. Le fond de fouille est à 15.0m de profondeur. Une grande partie du terrain est composé d'argiles très molles avec une teneur en eau allant jusqu'à 200%. La cohésion non drainée des argiles a été déterminée à partir d'un grand nombre de d'essais incluant une campagne additionnelle d'essais en laboratoire ainsi que des scissomètres. Le projet est très instrumenté : inclinomètres, piézomètres, extensomètres et topographie. L'excavation étant terminée les mesures sont comparées aux résultats théoriques utilisés pour le dimensionnement. Le but étant d'obtenir des modèles élasto-plastique et aux éléments finis comparables à la réalité.

KEYWORDS: Soft Clays, Bogotá, Inclinometers, Modelling, Back-analysis, Elasto-plastic, FEM, Mohr-Coulomb, HSM.

1. INTRODUCTION

1.1. Project Information

The Project located in Bogota, Colombia, consisted in the construction of 4 eleven-story office towers and 4 basements levels, occupying the entire venue area, 18.000 m2.
For basements construction purposes a 60 cm thick diaphragm wall (D-wall) going 40 m deep was built with a Top-Down excavation-construction sequence. As deep foundations, 40 m deep "barrettes" – rectangular bored piles – were constructed. A temporary steel column was installed in each barrette from the basement slab (B4) level up to the first floor slab (F1) level.
Even if the basement 4 level was 13.5 m deep, considering the additional excavation due the construction of this foundation slab, the Maximum Excavation Level (MEL) considered in the project was 15.0 m deep. (Figure 1 Figure 2).

1.2. Geotechnical Characterization

To establish the stratigraphy a geotechnical survey was performed including standard penetration tests (SPT) and cone penetration tests (CPT).

To improve the available geotechnical information, additional laboratory tests and in situ Vane Shear Tests (VST) were performed in different soil units. Triaxial tests shown higher resistances compared to traditional SPT-derived strengths considered for projects in Bogota. On the other hand, the VST underlined the existence of a residual state in clay materials showing substantial clay sensitivity.

The project stratigraphy is presented in Table 1. The water table (WT) was detected at 4.0 m depth.

Table 1. Design: stratigraphy and soil parameters

	Description	γ (t/m^3)	C' / C_u (kPa)	ϕ (°)	E_{MC} (kPa)
U1	Backfill.	1.8	18 / -	13	15,000
U2a	High plasticity clay with organic lenses, soft consistency.	1.5	- / 47	0	6,000
U2b		1.4	- / 40	0	6,500
U2c		1.4	- / 60	0	7,000
U2d		1.4	- / 50	0	7,500
U2e		1.5	- / 44	0	8,000
U3	Clayey Sand, medium dense to dense.	1.8	10 / -	25	30,000
U4	High plasticity clay, soft to medium stiff consistency.	1.8	- / 40	0	8,000
U5	High plasticity clay, stiff consistency.	1.6	- / 80	0	8,500
U6	High plasticity clay, soft to stiff consistency.	1.8	- / 42	0	8,000

2. PROJECT DESIGN

To design the project, the analysis of the construction sequences was made with Finite Elements Methods (FEM), using the computer program PHASE2®.
To consider the fact that the unload/reload modulus of elasticity

is higher compared to virgin compression, across the soil column within the excavation area, the modulus of elasticity is multiplied by a factor of 3 but the same strength parameters shown in Table 1 were considered both inside and outside of the excavation. The peak undrained cohesion values were considered taken from UU triaxial tests.

Total theoretical displacements of about 15 cm estimated in the last excavation stage (-15.0 m), seems mostly due to bottom excavation heave.

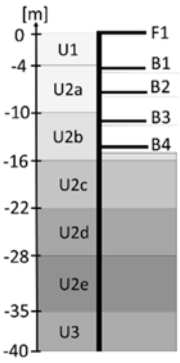

Figure 1. Project Cross-section

Figure 2. Excavation/Slab construction stages

3. CONSTRUCTION

Local practice of deep basement excavation takes place in plots smaller than 400m², the innovation for this project was to perform the excavation in plots of 1000m² which required the use of the observational method with extensive monitoring of the excavation. There was actually no restrictions in terms of areas excavated during works. The toe of the D-wall was embedded in dense sands (U3) to ensure bottom stability during excavation.

4. INSTRUMENTATION

The instrumentation installed to monitor the excavation was composed of the following elements: Inclinometers in the D-wall panels, Piezometers equipped with "divers", inside and outside the excavation and extensometers to measure the bottom heave.
The piezometers indicated that water pressure in the permeable layer U3 located between the -35.0 to -40.0 m was lowered by pumping in deep wells, but the groundwater level was not affected outside the excavation. Thanks to extensometers located inside the excavation at three different depths (17m, 27m and 37m), it was possible to measure the maximum expansion of the bottom of the excavation, between 9 and 24 cm. The behavior recorded by inclinometers in the D-wall, is detailed below.

5. REAL D-WALL BEHAVIOR. BACK-ANALYSIS

5.1. Calibrated PHASE2® analysis

Based on the measured behavior of the D-wall, the initial geotechnical model was "calibrated", in order to represent with FEM, the actual behavior observed. The only modified parameter compared to initial geotechnical model was the Young Modulus (Table 2).
Based on the "calibrated" model, the theoretical behavior of the D-wall was estimated and compared with the real behavior measured with inclinometers. It was assumed that the fixed point of integration was at the foot of the wall for the first excavation stage and either B1 or F1 (Figure 1) for the remaining stages of excavation. However, the graphs are interpreted by analyzing the curvature of inclinometer curves.

Unit	E_{ext} (MPa)	E_{int} (MPa)
U1	15	30
U2a	5	10
U2b	5	11
U2c	6	18
U2d	12	36
U2e	30	120
U3	400	1,600
U4	100	400

Table 2. Proposed "calibrated" values of Modulus of Elasticity (outside and inside the excavation)

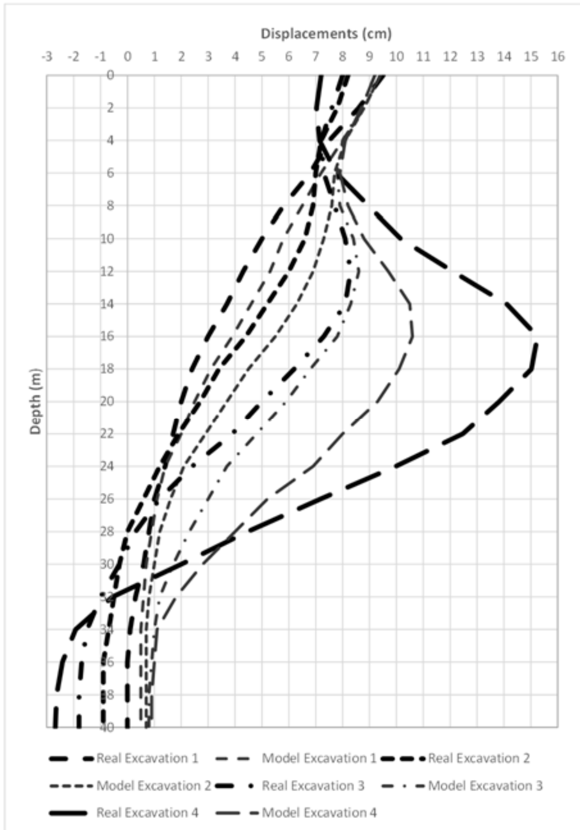

Figure 3. Measured and calculated curves (Constitutive Model Mohr-Coulomb with calibrated parameters). Measured displacements integrated from B1.

Figure 3 presents a comparison between FEM calculation results and measured curves. There is a good match between measured and calculated displacement curves for the first three excavation stages, however, for the last stage the measured curvature is greater than the calculated one. This may suggest an evolution towards residual characteristics when displacements increase.

5.2. PLAXIS 2D® FEM analysis

Given the previous results obtained it seems suitable to consider shear strengths of Clays closer to residual values to estimate the D-wall behavior for higher displacement ranges (last excavation stage). For that matter FEM computations were performed using PLAXIS 2D®. Gathering all the geotechnical data including tests performed during excavation a new set of C_u was estimated as shown in Figure 4.

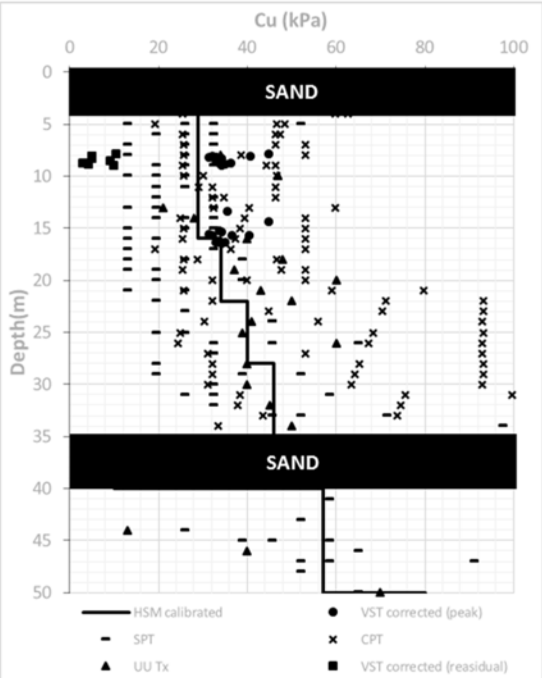

Figure 4. All undrained shear strengths values taken from tests

The Hardening Soil Model was chosen to account for the unload/reload phenomenon with 3 different moduli E_{50}, $E_{u/r}$ and E_{oed}. All these moduli are estimated with common correlations from the recalculated C_u as presented in the following equations (based on pressuremeter tests correlation used later for the elasto-plastic model):

$$p_l = 5.5 * C_u \ \ and \ \ {}^{E_m}/_{p_l} = 10 \qquad (1)$$

(Cassan 2005)

And,

$$E_{MC} = 4\frac{E_m}{\alpha} = 2 * E_{50}^{ref} = {}^{2}/_{3} * E_{ur}^{ref} \qquad (2)$$

(Plumelle, Serrai and Schmitt 1995)

Therefore for Clays:

$$ {}^{E_u}/_{C_u} = 110 \qquad (3)$$

This result is in the lower range of Duncan & Buchignani (1976):

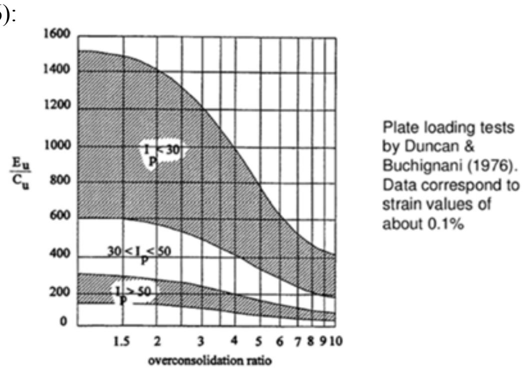

Figure 5. Duncan and Buchignani (1976)

The obtained parameters are summarized in Table 3.

Unit	C_u (kPa)	ϕ (°)	E_{50}^{ref} (kPa)	E_{oed}^{ref} (kPa)	$E_{u/r}^{ref}$ (kPa
U1	18	13	20,000	20,000	60,000
U2a	29	0	6,400	6,400	19,100
U2b	29	0	6,400	6,400	19,100
U2c	34	0	7,500	7,500	22,400
U2d	40	0	8,800	8,800	26,400
U2e	46	0	10,100	10,100	30,400
U3	10	25	180,000	180,000	540,000
U4	57	0	45,600	45,600	136,800

Table 3. PLAXIS 2D® proposed "calibrated" soil parameters (HSM)

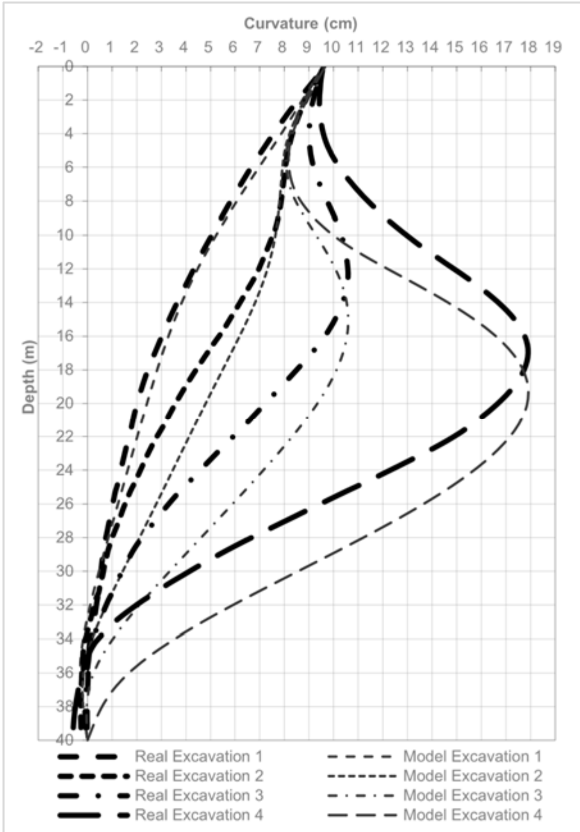

Figure 6. Measured and calculated curves (Constitutive Model Hardening Soil with parameters closer to residual strength). Measured displacements integrated from F1.

In terms of curvatures, this FEM analysis matches the measurements.

5.3. Elasto-plastic analysis

Even if FEM computations give good results, they are time consuming and often difficult to implement. Thus an alternative using elasto-plastic soil structure interaction was implemented using the Soletanche Bachy software PARIS.

An estimate of the coefficient of subgrade reaction given by the Schmitt formula (Schmitt 1995):

$$k_h = 3.6 * \frac{{}^{E_m}/_{\alpha}}{a} \qquad (4)$$

For standard projects, the interaction height "a" necessary to mobilize passive pressure below the excavation level may be derived from empirical relationship:

$$a = 1.5 * l_0 = 1.5 * \left(4 * \frac{EI}{k_h B_0}\right)^{1/4} \qquad (5)$$

So that finally:

$$k_h = 2 * \frac{\left(E_m/\alpha\right)^{4/3}}{\left(EI/B_0\right)^{1/3}} \qquad \text{(AFNOR 2009)} \qquad (6)$$

For this project, inclinometers clearly show that most of the reaction is taken in deep sands (U3) which mobilizes the full height of soft Clay layers. Therefore the interaction height is significantly higher compared to the empirical one derived from Eq. 5, so that Eq. 6 is no longer valid (AFNOR 2009). Corrected coefficients of subgrade reaction are presented in Table 4 and results are shown in Figure 7.

Unit	C_u (kPa)	ϕ (°)	α	E_m (kPa)	K_h (kN/m^3)
U1	18	13	0.33	3,300	700(a)
U2a	29	0	0.50	1,600	300(a)
U2b	29	0	0.50	1,600	300(a)
U2c	34	0	0.50	1,900	400(a)
U2d	40	0	0.50	2,200	500(a)
U2e	46	0	0.50	2,500	600(a)
U3	10	25	0.33	30,000	104,000(b)
U4	57	0	0.50	11,400	2,100

Table 4. PARIS proposed “adjusted” parameters

(a) The coefficient of subgrade reaction considered is consistent with an interaction height of around 30m.
(b) The coefficient of subgrade reaction considered is consistent with an interaction height of 3m.

This analysis gives good results. But its best advantages are its simplicity and its rapidity of implementation compared to FEM. Nevertheless actual measurements to confirm/adjust the interaction length during construction is crucial given the Bogota Clay sensitivity.

6. CONCLUSIONS

The Top-Down procedure is effective for building basements in unfavorable geotechnical conditions such as those encountered in Bogota.

The characterization of Bogota soils should not be based only on traditional field tests such as SPT and CPT, but also on laboratory tests on undisturbed samples and direct field measurements such as the shear vane tests.

The determination of the “strain related” shear resistance of the clay at the project site, allowed the definition of a safe and efficient construction process.

In order to ensure stability with regards to the uplift failure mechanism, enough pressure relief wells inside the excavation need to be installed.

These FEM back-analysis shows that “small” displacements using initial shear "peak" resistance in the analysis is appropriate, however, for “larger” displacement considering parameters closer to residual soil shear strength gives better results.

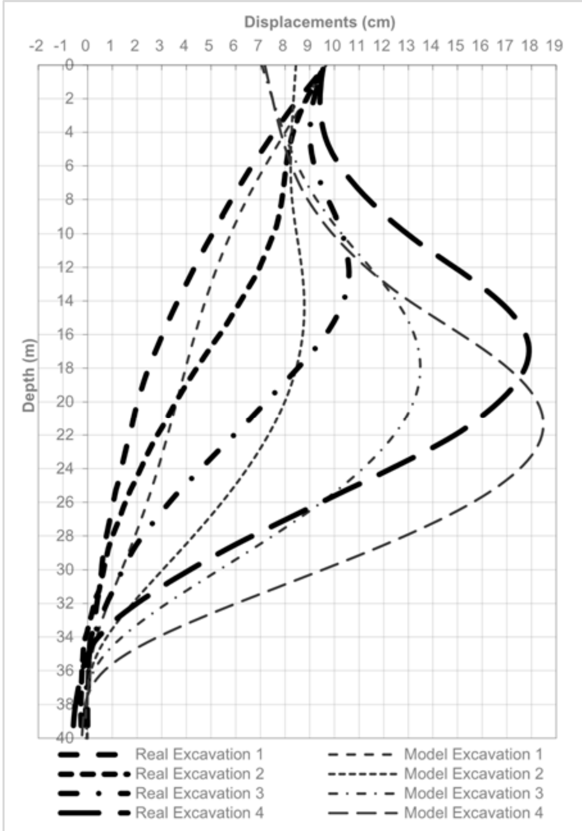

Figure 7. Measured and calculated displacement (Elasto-plastic Model with parameters closer to residual strength). Measured displacements integrated from F1.

An elasto-plastic method, if correctly adjusted given the geotechnical environment (reliable geotechnical survey and good understanding of the D-wall behavior), is a good alternative compared to heavy FEM computations.

In any case the observational method is required to ensure that models are consistent with the actual behavior on site.

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