Deep Retaining Walls Analysis:

the Case of "Biblioteca Central e Arquivo Municipal de Lisboa "

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Abstract: This dissertation was intended to contribute to the development on retaining walls analysis, mainly when implemented in urban environments. The focus was primarily on the use of numerical modelling with the finite elements method applied to Mohr-Coulomb, Hardening Soil and Hardening Soil Small Strains constitutive relationships. We intended to reinforce the theoretical and practical background of the subject considering static and cyclic loadings.

We present a case study, of a bored piles wall built on the scope of the contract for the "Biblioteca e Arquivo Central de Lisboa" in 2006. The fact that the maximum deep of the structure is 40 m, a value outstanding when comparing with other structures in Portugal, and that the superstructure wasn't built yet makes this an odd case, which deserves a deep analysis.

The validation of the numerical model was made with Plaxis 2D© software. A sensitive analysis and a back analysis of the geotechnical parameters was carried out. The neighbourhood structures were subject to a damage assessment using empirical methods.

We evaluated the performance and made the safety check of an alternative solution for the support system. The alternative solution uses passive solutions rather than active anchors, manly jet grouting columns and buttress. This approach was made using the Eurocodes and other normative documents.

The structure performance when subject to type 1 and type 2 Eurocode regulator's earthquakes was evaluated.

Retaining Walls

Nowadays retaining walls plays an important role in the spatial optimization of urban areas. In order to maximize the useful space this structures must have small mass.

This paper is focused on flexible structures which adopt curtain as denomination. The reduced stiffness and the fact that sometimes is necessary to execute very deep structures, impose the use of support systems to restrain the curtain displacement.

The performance and behaviour of retain walls depends on a large number of factors, we highlight some of the most relevant.

Knowledge of the soil characteristics plays a crucial role because the interaction with the structure is key issue, manly the type of soil, stiffness, resistance, permeability, among others. Soil improvement techniques are also an important aspect, especially for cohesiveless or soft soils.

Minimizing the time between the decompression of the soil and the support´s installation decreases the movements due to creep deformations, relaxation, consolidation or other time pending phenomena.

The type of wall directly influences the systems behaviour, including the settlements. The construction sequence is extremely relevant, because the soils have plastic behaviour that depends on the load cycles. The construction of the walls prior to the excavation is a common solution to minimize settlements and soils displacements when the excavation is underway.

Pre-tensioning of ground anchors and struts is an effective solution to minimize movements, but it may adversely affect the performance, due to the disturbance of surrounding soil when drilling the anchor holes or during the process of cement injection. Keep in mind that drainage can cause settlements due to consolidation affecting an area larger than the excavation zone.

We must consider possible temperature changes in the presence of steel struts, due to the high coefficient of thermal. This factor is particularly important in steel struts subject to heavy loads.

Case study

The case study focuses on a retaining wall build for the first construction phase of the "Bibloteca Central e Arquivo Geral de Lisboa". The maximum deep is 40 meters and the site has an aggressive topography. The excavation plant is rectangular with 100 m x 40 m. Within the vicinity special attention was given to reinforced concrete buildings with eight floors above the ground at a minimum distance of 20 meters of the site with greater depth of excavation.

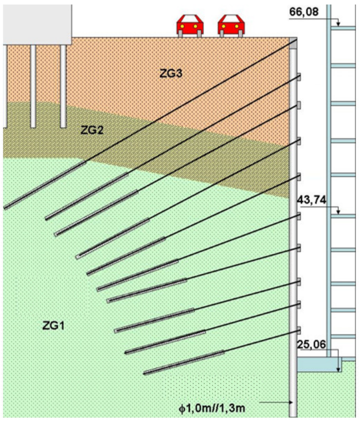

Figure 1 - Elevation southwest (Pinto et al, 2007)

Geotechnical exploration defined: ZG3: corresponds to the layer near the surface, consists of very heterogeneous landfills with variable thickness between 1,5 m and 19,6 m. ZG2: Underlying ZG3, is constituted by silt, clay sands, silty and sandy clay with NSPT values between 15 and 60 strokes. The maximum depth is 19,5 m. ZG1: Corresponds to the deepest layer, it consists of sand, sometimes clay, silt, fossil concentrations silty-sandy, clays and sandy silts. This area has a NSPT greater than 60 strokes.

The soil parameters were obtained from Pinto et al. (2006) or estimated from published correlations. We used the following with the HSSS model:

Table 1 – Geotechnical parameters for the HSSS model

	ZG1	ZG2	ZG3
γ_{unsat} (kN.m^{-3})	21,0	19,0	17,0
E_{50}^{ref} (kN.m^{-2})	140,0	40,0	7,00
E_{oed}^{ref} (kN.m^{-2})	140,0	40,0	7,00
E_{ur}^{ref} (kN.m^{-2})	420,0	120,0	21,0
c'_{ref} (kN.m^{-2})	80,0	20,0	1,0
φ' (°)	45,0	36,0	25,0
ψ (°)	17,0	7,0	0,0
m	0,49	0,52	0,53
υ'_{ur}	0,23	0,29	0,35
K_0^{nc}	0,30	0,40	0,55
R_f	0,85	0,87	0,82
p'^{ref} (kN.m^{-2})	216,6	131,7	54,7
G_0 (kN.m^{-2})	358,7	178,7	57,6
$\gamma_{0,7}$ x 10^4	2,85	1,94	1,87

The most sensible areas are on the southwest elevation near the place with biggest height and the concrete buildings. We considered two locations, AB4, away from the excavation corner and the buildings, this place served to validate the numerical model, to realize the sensitivity analysis and the back analysis. The other zone, AB5, is located near the corner and served to assess the risk of the concrete structures and to check the safety of the alternative solution.

In those zones the retaining wall is constituted by a curtain of bored piles, with a diameter of 1,0 m spaced 1,30 m apart. In order to increase tension redistribution effectively, the piles are locked by reinforced concrete distributions beams.

10 levels of active anchors are fixed on the beams, and are sealed in ZG1 with a 10 m sealing bulb. The spacing between the anchors is 2.6 m, except for the two deepest levels in which the spacing is 3.9 m.

Model Validation

We used Ou´s (2006) methodology to correct the corner effects. This is a simplified approach, but can minimize modelling mistakes. This methodology allows correcting displacements only. It was considered that the southwest and northeast elevations have the same height witch gave us a displacement correction factor of 0.9 for AB4 and 0.70 for AB5.

The differences between Plate elements and non-porous elements (NPE) for modelling the retaining walls was evaluated.. To choose the model that best fits the case study we study these simulations:

1. Model MC for all soils and Elastic linear (EL) Plate elements for the curtain, with reference MC;
2. Model HS for all soils and EL-Plate elements for the curtain, with reference HS;
3. Model HS for all soils, using the PLAXIS© values for the auxiliary parameters and EL-Plate elements for the curtain, with HS-PLAXIS reference;
4. Model HSSS for all soils and EL-Plate elements for the curtain, with reference HSSS Plate;
5. Model HSSS for all soil and EL-NPE model for the curtain, with reference HSSS NPE.

The horizontal wall displacements are represented in Figure 2, together with the measured values.

The MC results are too far away from the real values, manly in the top levels. In the lower elevations the model´s behaviour is quite acceptable because the volumetric strains are uncommon in ZG1.

Comparing HS and HS-Plaxis results we can observe close values, which indicates that the pre-defined parameters values could have been used with confidence.

The HS simulation provides the best results in the upper region, while the HS-Plaxis simulation adapted better in the lower zone. Since globally the HS results were better we chose to maintain the values as used in HS.

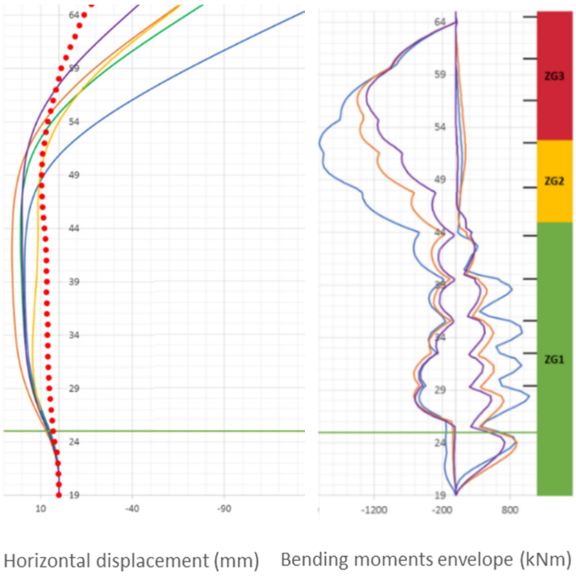

Figure 2 - Results obtained: left) curtain´s horizontal displacements; right) Bending moment envelop (red dots: monitoring; blue: MC; orange: HS; green: HS-PLAXIS; purple: HSSS-Plate; yellow: HSSS-NPE)

Similar results were obtained with HS and HSSS models. The difference between the measured value and the horizontal displacement at the top is about 5 cm in the HS simulation and 3 cm in HSSS-Plate simulation. In the lower levels the HSSS model also fits better.

The HSSS model is the best suited to the case study and we choose to use this to define all geotechnical zones.

We can see that the simulated displacements in HSSS-NPE adapt better than those of HSSS-Plate. We observe a decrease of 4.1 mm deviations from the monitoring data with the use of NPE elements. However, at the top of the curtain HSSS-Plate performs better than HSSS-NPE, with a deviation less than 20 mm.

The option was to simulate the behaviour of the structure through non-porous elements when the objective is to calculate the stress, strains and displacements of the system. Since it is not practical to extract the efforts from non-porous elements, we used plate elements simulations to extract the curtains efforts.

Sensitive Analysis

In order to quantify the sensitive analysis, we define the variation parameter as follows:

$$\lambda = V_{\text{analysis}}/V_{base} \quad (1)$$

Where V is the value of the parameter analysed.

In this paper we present the maximum wall displacement in the direction of the excavation, in the direction of the soil and the maximum absolute value of the bending moments envelop.

We calculated the average variation value response parameter per unit of the variation parameter. We present the values of linear correlation coefficient between the parameters and the values of the criteria to assess.

We pay a particular emphasis to the deformability modules through three approaches:

- applying λ values equally to E_{50}, E_{oed}, E_{ur} and G_0 from ZG1, ZG2 and ZG3 simultaneously;
- Applying λ values equally to E_{50}, E_{oed}, E_{ur} and G_0 for a geotechnical zone alternately, leaving the others zones with the baseline scenario values;
- For each zone simultaneously applying λ values equal to E_{50}, E_{oed}, then to E_{ur} and finally to G_0, leaving the other parameters equal to the base scenario values;

We highlight the sensitivity analysis of the ZG1 and ZG3 stiffness.

Figure 3 shows the influence on the horizontal displacement of the curtain and on the bending moment envelops.

ZG1´s stiffness commands the system in its lower part. We can see the low sensitivity of the upper zone, in contrast to the high sensibility in the coordinates belonging to ZG1.

In figure 4 we can realize that ZG3´s stiffness governs the system in the upper levels.

In Table 2 we can check the ΔV/Δλ for each analysis. We conclude that for this particular case, the increase of ZG1´s stiffness, E_{oed} and G_0 values the curtain displacement in the direction of the excavation decreases. A opposite effect is observed with E_{ur} or R_f values.

The reduction of displacement in the direction of the soil face occurs with the increasing of and ZG3´s stiffness, and the values of E_{oed}, G_0, c, v, $\gamma_{0,7}$, and R_f. A opposite effect is observed with E_{ur} or K_0^{NC} values. Increasing of ZG1´s stiffness decreases the maximum shear stress.

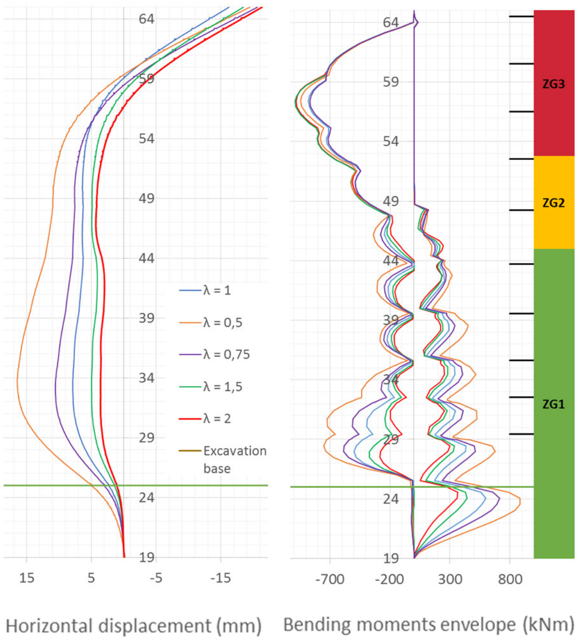

Figure 3 –Systems response as function of ZG1´s stiffness at the end of the excavation: left) horizontal wall displacements; right) Bending moment envelop

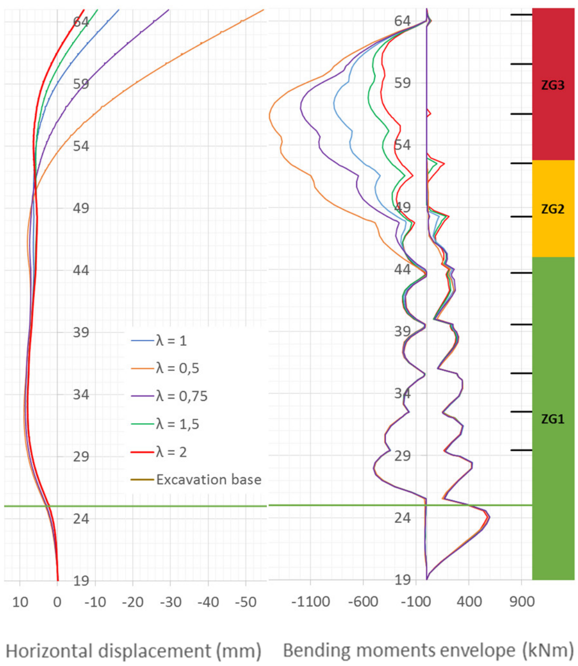

Figure 4 –Systems response as function of ZG3´s stiffness at the end of the excavation: left) horizontal wall displacements; right) Bending moment envelop

The maximum bending moment decreases when ZG3´s stiffness and E_{50}/E_{oed} increases and also when v or K_0^{NC} decreases.

Table 2- Sensitivity analysis results

	$u_{hmax,escavation}$ mm	$u_{hmax,soil}$ mm	|V| kN/pile	|M| kN·m/pile
All	-8,8	-32,7	-229	-685
R_{ZG1}	-8,1	1,3	-237	27
R_{ZG2}	-0,5	-1,3	16,6	39,9
R_{ZG3}	-0,6	-31,8	-7	-611
E_{50}	-2,8	5,7	-155	-30
E_{oed}	3,0	57,3	33,0	-518
E_{ur}	3,1	13,2	-24	90
G_0	-5,8	1,0	-174	-231
m	1,9	3,1	20	187
c	-0,5	-33,3	20	-101
ψ	-0,3	-2,0	61	-72
v	1,2	-46,6	-68	297
$\gamma_{0,7}$	-1,7	-11,8	-80	-245
R_f	4,0	-23,9	-78,8	-209
K_0^{NC}	0,7	48,4	26,1	372
R_{inter}	-0,8	0,4	11,4	163

Back Analysis

For the back-analysis we choose ZG3´s cohesion because there are some points that reached failure and that doesn't correspond to the soil´s real behaviour. We also choose the deformability modules of each geotechnical zone, because the system´s response is predictable and sensitive to those parameters. Also because they show a large dispersion in the geotechnical tests executed (Pinto et al., 2006).

In figure 5, the horizontal displacements of the curtain are represented. It was possible to minimize the deviations from the measured data from the average of 8.1 mm to 1.1 mm. The maximum deviation was reduced from 52 mm to 3.1 mm. The improved geotechnical values are presented in the following table.

Table 3 – Geotechnical parameters used in area AB4

	ZG1	ZG2	ZG3
E_{50}^{ref} (kN.m^{-2})	190,0	30,0	12,0
E_{oed}^{ref} (kN.m^{-2})	190,0	30,0	12,0
E_{ur}^{ref} (kN.m^{-2})	570,0	90,0	36,0
$c´ref$ (kN.m^{-2})	80,0	20,0	5,0
G_0 (kN.m^{-2})	486,8	131,0	98,7

We highlight the proximity of ZG1´s E_{50} new value when compared to the average of the pressuremeter tests, 190 MPa and 185 MPa, respectively. ZG3´s E_{50} was subject to a greater variation, from 7.0 MPa to 12.0 MPa. However that zone is characterized by a high heterogeneity and it is normal to assume that the stiffness vary significantly.

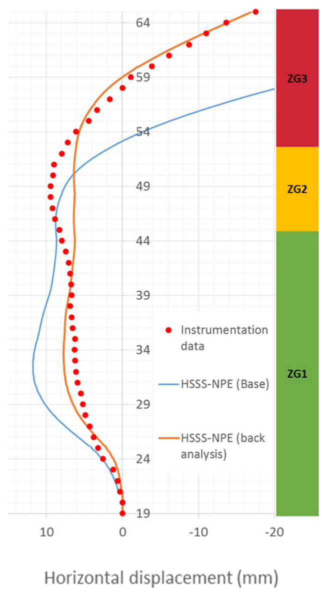

Figure 5 - Wall displacements at the end of the excavation

Curtain horizontal displacement evolution

The evolution of the horizontal displacements is shown in figure 6. The displacements at the top of the curtain stars on the first level and is heavily reduced after the third level. On the other hand the movements in the lower levels are only meaningful when the soil´s decompression allows an easier deformation of the curtain.

We can realize the high contribution of the 10th level for the displacement into the excavation, which is more than half of the total displacement.

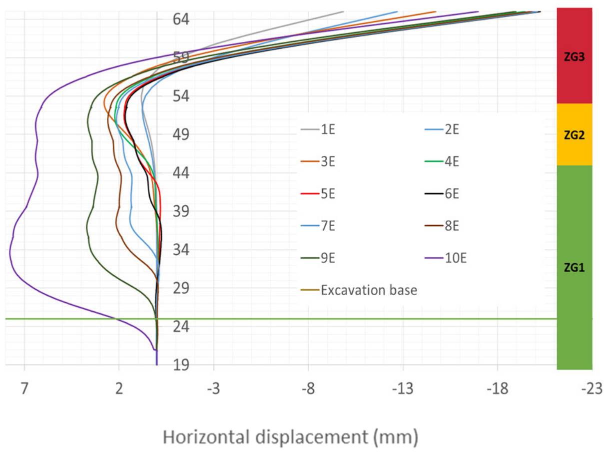

Figure 6 – Displacements after each excavation level

AB5 area

In figure 7 we can realize that when using data from AB4 back-analysis the displacement curves show an excellent agreement in medium and low levels, which reinforces the confidence of the parameters used to ZG1 and ZG2.

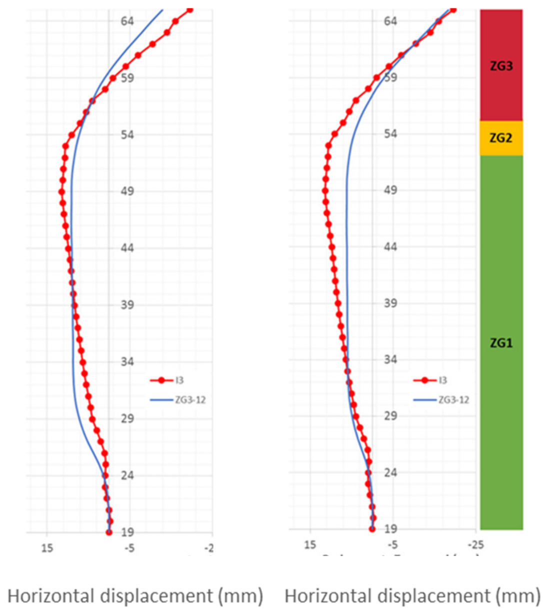

Figure 7 –Horizontal displacements, at the end of the excavation, obtained using; left) AB4 back-analysis; right) AB5 back-analysis

However, on the higher levels the system doesn't show this behaviour. ZG3 was stated as a high heterogeneity soil, so we assume that its stiffness is different from areas AB4 to AB5. Performing a new back-analysis we obtain the ZG3´s original stiffness values. With these values we can observe greater deviations on the middle levels, however, we can also observe a better agreement in the top levels of the curtain.

Concrete building damage

We used various methods to assess the damage of the near concrete structures. The results are presented on the following paragraphs.

The maximum relative deflection is 1/23172 and does not exceed any limit for generic or reinforced concrete buildings. The lower limit is 1/2000 relating to cracking in resistant walls, Meyerhof (1953).

The angular distortion limit of a building with *L/H* equal to 1 and hogging settlement surface, have a value of 1/2500, Burland and Wroth (1974), the simulated results was equal to 1/4267.

Regarding foundations from generic buildings the most restrictive values for settlements is 2.0 cm, Terzaghi (1948), much higher than the 0.114 cm obtained in numerical simulations.

If we consider the horizontal deformation we can use the abacus from Boscardin and Cording (2005) that estimates the damage as a function of angular distortion and horizontal deformation. The method is valid for a building with a length between 6 and 40 meters, *L/H* equal to 1 and *E/G* equal to 2.6. Assuming that the *E/G* is valid we have an angular distortion of $2.34 \cdot 10^{-3}$ rad and a horizontal deformation of $0.013 \cdot 10^{-3}$, therefore the damage is negligible.

When using Potts and Addenbrooke´s (1997) method to assess damage we assumed that the thickness of the slab is 20 cm and the concrete stiffness is 10 GPa. We obtained 2070 GN for *EA* and 415GN.m for *EI*. Using the method we got a value of ρ* equal to 0.179 and a value of α* bigger than 100. Where ρ* is the bending stiffness and α* is the axial stiffness.

We get a M^{DRsag} value of 0.1 and $M^{\varepsilon hc}$ value of 0.0. Where M is the ratio of deformation in the building and the "green field" deformation.

Correcting the parameters we have a deflection ratio equal to $3.8 \cdot 10^{-6}$ and a horizontal deformation of 0,0, using the abacuses of Burland (1995) that estimate the damage due to the deflection ratio and horizontal deformation we conclude that the damage is negligible.

Alternative solution

We evaluated the behaviour of an alternative solution, for the southwest elevation, presented in Pinto et al. (2006) outlined in figure 8. This alternative uses the same solution for the curtain, but a different support system.

The locking consists in ZG2 and ZG3 treatment with jet-grouting columns. In the foothills of the excavation it uses buttress and five levels of temporary ground anchors sealed in ZG1. The anchors are disabled after the installation of buttresses connecting the superstructure and the curtain.

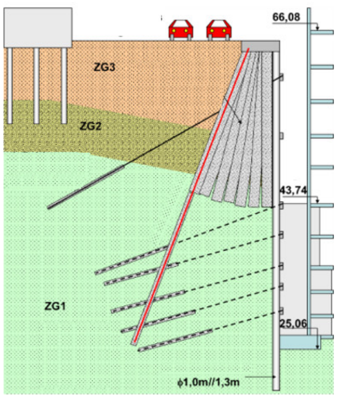

Figure 8 - Schematic representation of the alternative solution (Pinto et al., 2007)

In the end of the treated area there are TM80 ϕ127/9mm micropiles spaced 2.6 m apart and sealed deep in ZG1. To minimize the displacements in ZG3 a anchor level at elevation +60.50 (corresponding to the 1st level of anchors from the executed solution) is executed. The anchors have an inclination of 30 °, free body length of 23 m, they are sealed in ZG1 with a sealing bulb of 9 m, and spaced 2.6 m apart.

The five levels of temporary ground anchors are placed on the places corresponding to the five anchor lower levels of the executed solution. They have an inclination of 15° and are sealed in ZG1 with a bulb sealing of 8 m. The permanent and the provisional anchor consists of 7x0.62'' with locking force of 1000 kN.

The top level is united through a reinforced concrete beam with a height of 0.6 m and a thickness of 0.5 m. The axial force of the provisional anchors is distributed through two UNP280 steel profiles on the three top levels and with two UNP350 steel profiles on two lower levels.

We consider a thickness of 50 cm for the concrete buttresses and a spaced 5.2 meters apart. They have a development in the direction of excavation of 7 m in the 10 bottom meters and 5 m in top 8 m. For the struts we use 5 levels of 7 m HEB260 placed near the provisional anchors spaced 5.2 m apart.

The SLS and ULS safety check for the neighbour buildings was checked by a good safety margin.

For the other ultimate limit states design and safety verification were considered only persistent and accidental design situations. We analysed two accidental actions, the deactivation of the permanent anchor and the deactivation of the strut at the top level.

We didn´t consider EQU limit states, and the absence of groundwater level permitted us to avoid HYD and UPL limit states. For GEO and STR verifications we used design approach 1.

We considered a Δa value of 0.4 m decreased at the base of the excavation and applied the safety factors directly to the action and not the effects of actions. The partial safety factors for the materials were applied to the properties of the elements. We used the values from Eurocode 0 and Eurocode 7´s part 1 Portuguese annexes.

The pre-stressing of the anchors can have a negative or positive effect on the system behaviour, depending on the limit state considered. Therefore, we checked the safety for both hypothesis. For each check-up the degree of use (Λ) was calculated as follows:

$$\Lambda = X_{Ed}/X_{Rd} \tag{2}$$

The results are represented in table 4.

Table 4 – Results of the STR ULS safety check

		X_{Rd}	X_{Ed}	Λ (%)
Piles	*M*	1592	1412	88,7
	V	1942	1895	97,6
	N	15009	6728	44,8
Bottom buttresses	*M*	65335	60142	92,1
	V	8049	7108	88,3
	N	108671	67155	61,8
Top buttresses	*M*	16031	15578	97,2
	V	6899	4330	62,8
	N	49707	6155	12,4
Concert beams	*M*	222,9	207	92,9
	V	517	477,8	92,4
Steel beams from 1 to 3 levels	*M*	283,6	245,9	86,7
	V	930,4	546,5	58,7
Steel beams from 4 and 5 levels	*M*	505,9	474,5	93,8
	V	1613,2	547,5	33,9
Struts	N_b	2863	2760	96,4
Micropiles	N_t	476,2	448	94,1

In figure 9 we can observe the curtains horizontal displacements. On the alternative solution the top displacement is on the direction of the excavation with a relative value of 1/3000. While on the executed solutions the movement is in the opposite direction with a maximum relative value of 1/2000. Both solutions create very low displacements when comparing to others retaining walls projects.

It is possible to realize that the curtains displacements on the lower levels were bigger on the alternative solution, because the active anchors are more suitable to lock the displacements. Even so the alternative solutions displacements were very low.

The bending moments are much different because the jet-grouting column prevents the deformation of the wall which implies that there isn't curvature on the top levels. Another difference is related to the axial efforts because the alternative solution reduced que impulses transmitted to the wall in the top levels.

In lower levels we observe an intense gain in the axial efforts because the piles work as buttresses foundations. Even so, the maximum degree of use, when evaluation soils resistance to piles axial compressions is only 30%.

We can say that the alternative solutions would maximize the axial efforts and minimize the bending moment.

The fact that normally the bending moments safety check is more demanding could bring advantages for the alternative solution, regarding the piles design.

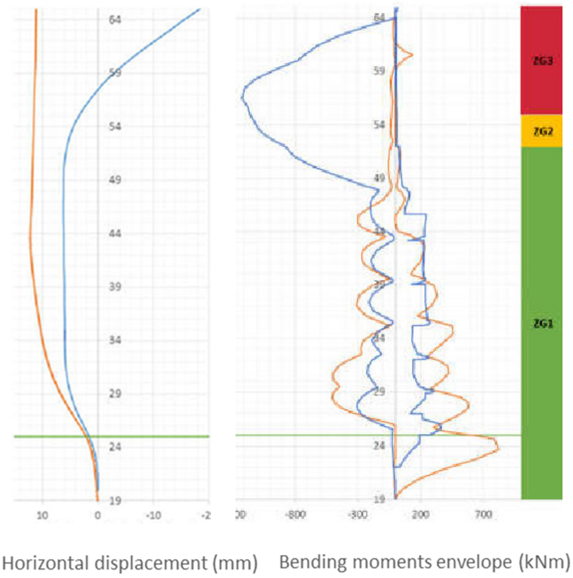

Figure 9 – Original (blue) and alternative solution (orange): left) curtains horizontal displacement; right) bending moments envelop

Dynamic

The system response to a type 1 and type 2 statutory seismic actions were evaluated. The actions were defined by artificial accelerograms using the Eurocode 8 part 1 approach.

To characterize the cyclical behaviour of soils the α and β Rayleigh damping coefficients were calculated. We used the Darendeli methodology (2001) to estimate the variation curves for the distortional stiffness and damping coefficient. The results are shown in Table 5.

Table 5 – Rayleigh Damping coefficients

ZG	ξ (%)	f_1 (s^{-1})	f_2 (s^{-1})	ω_1 (rad^{-1})	ω_2 (rad^{-1})	α x 10^2	β x 10^5
1	7,5	1	3,6	6,28	22,62	1,068	7,515
2	10,0	2,2	13,8	13,82	36,44	1,019	2,024
3	12,5	1,8	11,3	11,31	27,65	1,031	3,299

The seismic response of the structure was defined by the EC8-1 (2010) design spectrum, this requires the definition of the coefficient (q). The executed solution has a performance coefficient of 2.0 (Pinto et al., 2006).

It was assumed a class III for the structure importance. It was considered that the vertical action of the earthquake don’t affect this type of structure, therefore the analysis is restricted to the horizontal component of the seismic action.

The seismic actions were generated, using SeismoArtif version 2.1 software, from three artificial accelerograms for each type of earthquake. It was considered a duration of the stationary part equal to 45 seconds for the type 1 earthquakes and 15 seconds to the type 2 earthquakes.

The dynamic analysis were made using PLAXIS 2D version 2015 software. We used standard absorbent boundaries. The geometry of the network was extended 700 meters towards the masonry and 400 meters in the direction of excavation, which correspond to 1791 finite elements. The interfaces between structures and soil were removed.

Results

The relative displacements between the base and the top of the piles are shown in figure 10. Type 1 earthquake produces bigger displacements due to the longer duration of the action. It is shown that the effective time period of the earthquake is very important to characterize the system response.

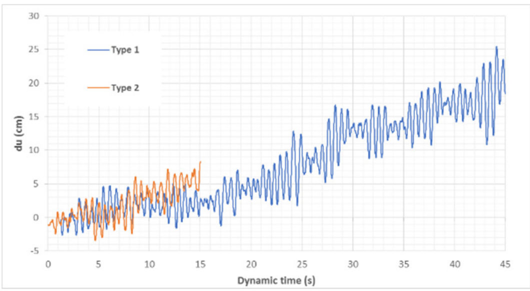

Figure 10 - Differential horizontal displacements: type 1 seismic activity (blue); type 2 seismic activity (orange)

Note that, for example, the structure subject to earthquakes type 1 with 30 seconds of duration would have a relative displacement of approximately 15 cm instead of the 25 cm calculated with the earthquake with 45 seconds of duration.

The value of *u/H* of the top wall due to type 1 earthquakes is 1/160 and for type 2 earthquakes is 1/500. All the previous values are higher than the values of *u/H* obtained with static actions.

In figure 11 we can observe that the bending moments envelop maintains their shape, however, with a significant increase in the values.

We can observe an increase of the maximum bending moment equal to 2.3 times in a type 1 earthquake and 1.6 times for a type 2 earthquake. Note that for the fundamental vibration period of the soil the spectral acceleration for a type 1 earthquake is bigger than the value for a type 2 earthquake (0.35g and 0.25g respectively), which may explain the greater severity of earthquakes type 1.

On the table 6 we present the maximum values for the efforts.

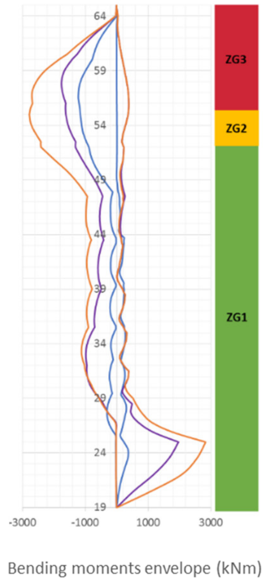

Figure 11 – Bending moments envelop for: static conditions (blue); type 1 seismic activity (orange); type 2 seismic activity (purple)

Table 6 – Maximum efforts values for 1 pile

	Static	***T1 seismic action***	***T2 seismic action***
N_{min} (kN)	-2685,2	-8177,9	-6864,6
N_{max} (kN)	61,6	783,1	783,1
V_{min} (kN)	-442,1	-981,0	-500,5
V_{max} (kN)	584,8	1434,1	1086,5
M_{min} (kN.m)	-1220,0	-2776,9	-1755,7
M_{max} (kN.m)	362,8	2833,0	1969,3
$\|N_{max}\|$ (kN)	2685,2	8177,9	6864,6
$\|V_{max}\|$ (kN)	584,8	1434,1	1086,5
$\|M_{max}\|$ (kN.m)	1220,0	2833,0	1969,3

The maximum axial loads of the ground anchors are represented in table 7 and figure 12. For type 1 earthquake we observed a axial load multiplier between 1.29 and 1.66 with an average value of 1.41. For type 2 earthquake we observed a axial load multiplier between 1.14 and 1.41 with an average value of 1.24.

We can observe that the multiplier effect increases with the decrease of the ground anchors head altitude.

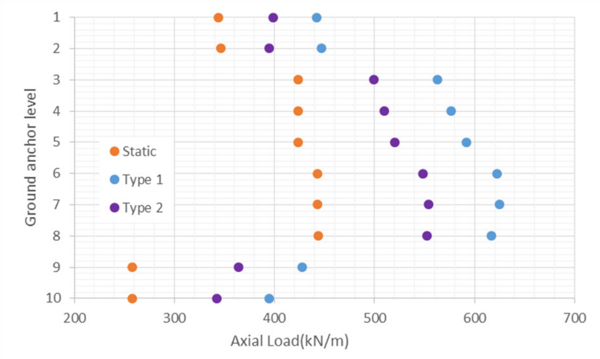

Figure 12 – Anchors axial loads: static conditions (blue); type 1 seismic activity (orange); type 2 seismic activity (purple)

Table 7 – Maximum anchors axial loads

Level	Static P (kN/m)	T1 seismic action P (kN/m)	T1 seismic action $P_{estático}/P_{sismo}$	T2 seismic action P (kN/m)	T2 seismic action $P_{estático}/P_{sismo}$
1	344,1	442,3	1,29	398,6	1,16
2	346,1	447,0	1,29	394,6	1,14
3	423,3	563,1	1,33	498,9	1,18
4	423,5	576,7	1,36	509,6	1,20
5	423,5	592,1	1,40	520,2	1,23
6	442,9	622,4	1,41	548,5	1,24
7	443,3	625,0	1,41	553,9	1,25
8	444	616,4	1,39	552,2	1,24
9	258,1	427,3	1,66	364,4	1,41
10	258,2	394,8	1,53	342,4	1,33

Conclusions and final remarks

The conclusions should be evaluated carefully if extrapolated to other scenarios. We only considered one case study with no statistical significance.

We used empirical relationships to estimate some geotechnical parameters, but It was not clear if this approach brought any advantages when compared to the pre-defined values of Plaxis© 2D.

The MC model overestimates the efforts and manly the displacements in ZG3, however in ZG1 it gave us reasonable results. When analysing the plastic points of the system we observed that only distortional hardening occurred in ZG1, while in ZG3 the hardening was caused also by volumetric deformations. This behaviour is not considered in the MC model.

Therefore, soils with good strength characteristics subject to median actions may be modelled by the MC model with reasonable results.

Comparing the HS and HSSS models we concluded that the HSSS model adapts better to the case study. However, the need to estimate $\gamma_{0,7}$ and G_0 without pre-defined values in Plaxis© 2D and the fact that both parameters, but mainly G_0, have a very significant influence on the system can jeopardize the use HSSS when there isn't data available.

We concluded that the constitutive models were able to predict the systems behaviour with reasonable results and with good results when using the back-analysis information. This demonstrates that numerical modelling is a credible geotechnical toll. However, the back-analysis results may be skewed by the set of assumptions and approximations made.

In the sensitivity analysis we concluded that the system depends fundamentally on the stiffness parameters of ZG1 and the ZG3. The rigidity of ZG1 controls the behaviour on the lower zone (horizontal displacements in the direction of the excavation, the heave at the base and the efforts on the pile lower levels). The ZG3´s stiffness controls the behaviour at higher elevations (wall displacements in the direction of the soil, the surface settlements and the efforts in the top levels of the pile). We also concluded that:

• The maximum values of displacements were more sensitive to the variation of geotechnical parameters than the maximum values of the piles efforts. The normal effort showed a very low sensitive when comparing to other efforts;

• The rigidity modulus that seems to dominate the behaviour is G_0 in ZG1 and E_{oed} in ZG3.

• $\gamma_{0,7}$, m, φ, c, K_0^{nc} showed a relatively significant influence on the system, however some parameters may be subject to a very low variability range;

• The parameter ψ has a reduced impact because its value is only meaningful in ZG1 and this soil didn't show significant volumetric strains;

• R_{inter} has a median influence on the system and maximizes the efforts, therefore the system can be modelled considered rigid interfaces if the objective is design or structural safety check.

Structural elements were modelled through plate and non-porous elements. The first ones were more suitable to estimate the efforts in the structures and the latter were more suitable for estimating the displacement and to analyse the overall system.

There are several methods to estimate the damage of adjacent structures without requiring detailed knowledge of the buildings structural components. The likelihood of the neighbouring buildings suffer damage is reduced, due to the very good characteristics of ZG1, the high levels of stress used in the anchors and the stiffness of the curtain.

The good performance of the case study reinforces the confidence and versatility of anchored bored piles curtains used as peripheral contention in urban areas.

The SLS and ULS safety of the alternative solution were checked. The fact that the alternative solution has only one level of definite pre stressed anchors suggests that in the long-term the alternative solution could have been more appropriate.

We realized that the alternative solution frees the piles from major efforts on the top levels, because of the jet-grouting columns, and that increases the axial efforts on the lower levels, because the piles worked as buttress foundations.

We found that the decrease in the buttress´s rigidity and the size of the embedment influences the safety check of the lower buttress, the micropiles and the struts. It was found that the system is more sensitive to the buttress´s stiffness when comparing with the embedment length.

Unfortunately it wasn't possible to analyse the system dynamic behaviour with the same detail used for the static analysis.

The maximum displacement of the top of the pile for the two solutions were higher in the case of the type 1 earthquake, the fact that the duration of the seismic action is bigger for this type of earthquakes is the major factor in this behaviour. All horizontal displacements at the top of the pile after the completion of the seismic action are greater than *u/H* values associated with static actions.

All efforts increased when the structure is subjected to seismic actions. The maximum bending moments were expanded 2.3 times for a type 1 earthquake and 1.6 times for a type 2 earthquake.

Future developments

In relation to the case study, some aspects were not considered, and deserved a analysis, including:

The displacements in the piles while the execution phase was underway were not considered. Despite the greater complexity of the modelling it would be an interesting improvement.

We considered a plane strain state witch can produce errors, mainly on the corners, therefore, an aspect to optimize would be through the use 3D models.

A future development could go through the detailed analysis of anchors behaviour. With particular interest we could try to understand the relationship between wall displacements rates with the variations measured in the anchors load cells.

A more comprehensive and deeper back-analysis could be done by variation of a larger number of parameters. It would also be desirable to evaluate the use of different constituent relationships for each geotechnical zone including models not used in this text, namely viscoplastic.

The recovery of the original data from the geotechnical exploration could allow an interesting statistical treatment. The analysis of the variability of the geotechnical parameters obtained from tests or empirical relationships with the variability of response criteria would permit the quantification of a risk associated with the use of each geotechnical parameter.

Finally, it would be interesting to compare the dynamic results with the ones obtained from an explicit plastic non-linear dynamic analysis.

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