Excavation in urban areas using slab bands

Sara Condeixa Fernandes

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Instituto Superior Técnico, Lisbon, Portugal

Nowadays the construction in urban centres has been growing exponentially and it's even more necessary to use the subsoil. To protect the deep excavations and when the use of surrounding soils is forbidden earth retaining structures braced by ground anchors can't be used, so the solution is support structures that can't surpass the excavation area, for example using slab bands or shoring.

This thesis studies the use of slab bands as bracing solution for earth retaining structures in urban areas, the case study of the at *Fontes Pereira de Melo* Av. nº 39 to 43, building basements, in Lisbon..

From the data of the monitoring plan, it was possible to make a comparative analysis between the data from the real work and those obtained by the numerical modelling, using the software Plaxis 2D. There were made two type of numerical modelling, one using soil characteristics from SPT tests and other with values from MASW tests and both were compared with the displacements measures at the site.

Keywords: *Excavation and peripheral earth retaining structures, piles curtain, slab bands, instrumentation, numerical modelling*

1. Introduction

This thesis follow the excavation, thought weekly visits, of 6 basements in centre of Lisbon, which use slab bands in wall that is near the metro line in *Fontes Pereira de Melo* Avenue. This condition demands special attention to displacements in order to guarantee the safety of the metro line.

The use o slab bands requires knowledge about the geotechnical characteristics of soil and a careful construction plan, so the slab bands don't occupied too much space, which could delay the excavation works.

To complement the knowledge of the soil, MASW tests were performed at the study area and compared with the SPT tests made in the 5 different boreholes. This way it was possible to study the soils with tests based on small deformations (MASW) and on large deformation (SPT).

2. Peripheral earth retaining structures

Flexible support structures are preferable to use in urban centres because they don't require a high area to be constructed. It allows vertical excavation using adequate earth retaining structures with the main function of control the horizontal displacements of the contention structures, thus ensuring the safety of the shell-work, (Matos Fernandes, 1983). There are three types of flexible structures: freestanding, mono supports and multi supported. In this paper the attention will fall mainly in multi supported structures.

Multi anchored walls, the use of ground anchors to support walls is usual in excavations from simple trenches to deep excavations. Due to the better mechanical behaviour it’s usual to use ground anchors, however if the geotechnical conditions aren’t the indicated they should be replaced by struts. (Guerra, N., 2014).

The most frequent cases struts are install in levels, supported in opposing faces of the excavations, but sometimes because the excavation geometry, the struts must be install in leaning positions connected to concrete blocks in the ground or in the corners.

The study of earth retaining walls supported by ground anchors can’t be done through the use of Rankine/Coulomb theories, because these walls do not suffer from rotating on the base structure, (Matos Fernandes, 1983). It can be considered that the walls displacements are relatively small on the top, growing in depth. The displacements are small near the support, but due to the wall movement they tend to grow. This situation appears to happen every time its install a new set of ground anchors and this are the reasons it can be use the Rankine/Coulomb theories.

Terzaghi and Peck conclude that the way to study earth pressure in anchored walls couldn’t involve earth pressure calculation theories, because these are dependent of the system localization, allowed deformations and wall stiffness. Terzaghi and Peck analyzed real cases and summarize it though the diagrams that vary thought-out the same excavation due to factors related to the constructing process.

King Post Walls or definitive Berlin wall are a traditional solution as a foundation and soil earth pressures control. It characterize as reinforce concrete panels supported by metallic profiles, it’s influenced by the soil-structure iteration and should be use in compacted soils. It has an economical constructive process and allow the simultaneous excavation and wall construction, (Brito, 2011).

Bored piles curtain, it’s a solution where the piles have the goal of transmit the vertical loads to competent soils and accommodate horizontal forces, delimiting the construction area. The use of spaced bored piles is defined according to its diameter and the geotechnical conditions. This space is usually filled with shot concrete reinforced with steel mesh. The use of this technique is proper in stable soils that offered stiffness towards larger diameter piles, (Cortez, 2010).

Slab Bands are the support structures in focus in this these. Slab bands are a set of beams which resist to the earth pressures and can remain in the final structure of the building. They take advantage of high stiffness of its elements and high capacities to control displacements to assure the safety of the area (Pinto, et al., 2010). Because this structure will be included on the final and it controls the soil decompression. And for the same reason it should represent the underground levels. If the conditions tolerate the excavation below the slab bands, it will be possible to do a better formwork, improving the quality of the slab band bottom face finishing. It’s important to assure that the width of the slab bands isn’t bigger than they have to, so other works aren’t penalized. To minimize the vertical deformations along the width of slab bands, during the excavation process be vertical steel profiles should temporally install, as illustrated in Figure 1.

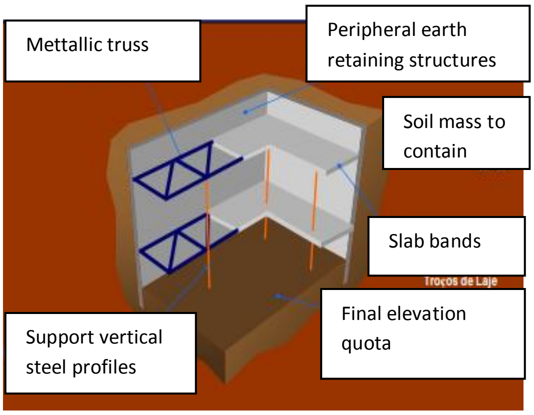

Figure 1 Scheme of slab bands.

MASW (Multichannel analysis of Superficial Waves) is a non evasive seismic test allows to evaluate the elastic conditions(stiffness) of the ground, which allows the study of initial shear modulus (G0).Therefore this is a tests that doesn't neglect the small deformations like SPT test that study bigger deformations. One of the disadvantages is the complex interpretations that those tests need.

MASW is based on the study of the phenomenon of scattering of surface waves in vertically heterogeneous soil. The test is performed using two vertical receptors, which record the impulses caused in the ground by a source. The wave train is transformed to obtain a dispersion curve, Rayleigh wave's velocity as function of frequency. After treatments using an inversion algorithm, it's obtained the evolution of shear velocity in depth. The tree main steps referred were: data acquisition, dispersion analysis and inversion, (Lopes, I.; Santos, J.; de Almeida, Isabel M., 2008).

3. Study Case

The case study is in a very busy area of Lisbon with different types surrounding buildings, an underground parking lot, two buildings with different age of construction and two important avenues with metro a line below the *Fontes Pereira de Melo* Av. The design for the area in focus is a tower with 17 levels above the ground and 6 basements to be use as a parking lot, loading to about as 32407 m^2 of gross construction area. The Figure 2 shows the site area described.

Figure 2 Aerial view of the study case.

3.1. Geotechnical Conditions

For the geotechnical point of view the area under study is located in a Miocene layer called *"Argila dos Prazeres"*. However above this layer it was found a layer of landfill, with a variable thickness of 2 to 5m. This is a quite heterogeneous layer, which consist mainly by silty-sandy and clayey-silty materials, this classification is represented by values range between 4 and 19 NSPT blows.

The *"Argila dos Prazeres"* layer is represented by a sequence of cohesive materials, silty-clay and loamy clay interspersed with carbonaceous clay and very resistant lenticules. The NSPT values in this layer vary as the depth increases. So in the firsts meters the NSPT value range from 13 to 60 blows, so this layer can be considered to have geotechnical behaviour of a hard to stiff soil. In the inferiors levels the stiff silty clays with values ranging from between 26<NSPT<60 and the intercalated layer sand stones and marls lenticules has values ranging from 35<NSPT<60. So both this layers can be considered stiff material (Pinto, A.; Pita, X., 2014).

Table 1 Estimated soil parameters.

	Soil	N_{SPT}	γ_t (kN/m^3)	Φ' (°)	c' (KPa)	E (MPa)
At	**Sandy Clay**	4 - 19	19	25	-	5
MPR	**Margo-silty Clay / Carbonated Clay**	13 - 24	19	26 - 28	5	10-20
		25 - 37	20	30 - 32	10	20-30
		41 - 60	20	32 - 34	20	35-50
	Margy Silt	> 60	20	38	20	60

3.2. Utilities

Due to the project dimension many services were affected and needed to be replaced in a way that didn't disturb the normal life of the neighbour population. Because the work site is going to cut the traffic at the *5 October* Avenue, the referred traffic had to be redirected to alternative streets, which implied extra work in these streets so they can accommodate the increase of the traffic. Other service that had to be transferred, to different places were the gas, water, communications and IP from that *5 October* Avenue segment to *Fontes Pereira de Melo* Avenue.

3.3. Adopted Solution

There are several conditions in the study area that demand different types of peripheral walls. And these different techniques have to assure not only the soil retaining, but also guarantee an effective control of the land and nearby structures. Besides these two conditions, the defined peripheral walls must ensure an easy execution, fast and safe with minimums cost possible.

Since the water level is 22,3m below the surface, it was decided to use bored piles curtain, the piles have a diameter of 0,60m and a distance of 1,20m between axes, except AB side in which the distance between piles axes decreases to 0,80m. The length of the piles varies between 25 to 28m, with 7m or 10m below the final excavation level. The exposed ground between piles is lined with shotcrete reinforced with metallic fibres with a thickness of 0,08m. In the CD side due to the proximity of the underground parking lot, the use of pile curtain isn't possible, so it's necessary to use another technique, such as King Post wall, the reinforced concrete panels have a minimums thickness of 0,30m.

The bracing systems used are mainly corner shoring and anchorages. These last are composed by 5 wires of 0,6'' that can accommodate a load of 780 kN and a pre stress of 700 kN, separated by 3,6m of each other and have variable length and inclination so they cannot intersect existing structures.

The focus of this paper is the AB view due to its proximity to the metro line (less than 15m), therefore the use of ground anchors is not advisable. So the choices of the bracing solution felt on reinforce concrete slab bands.

The project defines that the slab bands keep the inclination design in the architectural project, so slabs bands start in the north side of wall at the level-1 connecting to level -2 in the south side. And the same happens between the levels -3 and -4. The slabs bands were pre-design considering the resistance and stiffness needed to resist the earth pressure and having in the consideration that a very large slab band would delay the remaining excavation works. They have a variable width of 16m near the lateral sides and gradually decrease to 9,75m in the central 44m, with a constant thickness of 0,30m. To minimize the vertical deformations along the width of slab bands, during the excavation process there were installed temporally vertical steel profiles (HEB260) distributed in 2 lines with different distances from AB, (Pinto, A.; Pita, X., 2014).

During the visits at the site, the constructive process suffered some setbacks therefore it didn't precisely follow the one defined in the project. This happened due to bureaucratic conditionings that delayed the closing of the 5 October Avenue segment. Thereby the finalization of BC wall and the construction of the wall CDE were postpone until the conclusion of the traffic readjustment.

3.4. Monitoring and survey Plan

Any urban excavation works these days' demands the use of a monitoring and survey plan so the displacements are controlled to avoid collapses and problems that can penalize the deadline of conclusion. In this site, especially the AB wall (Figure 3), there were install 2 inclinometers, 6 topographic targets. And there were also created alert and alarm criteria to be a reference to the displacements measured. These values differ according to the structure, shown at Table 2.

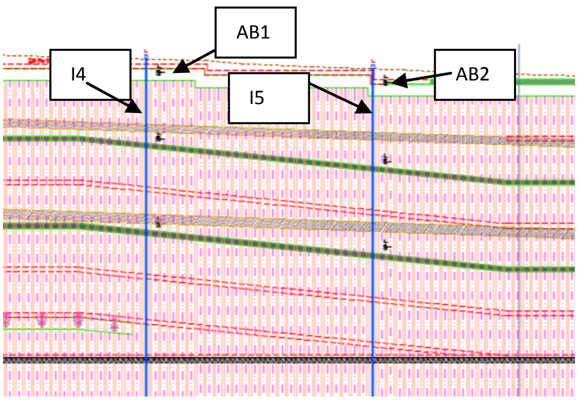

Figure 3 Inclinometers localization

Table 2 Alert and alarm displacements.

Structure	Alert		Alarm	
	Horizontal	Vertical	Horizontal	Vertical
Earth retaining structure	20mm to 10m excavated	15mm to 10m excavated	30mm to 10m excavated	22,5mm to 10m excavated
Metro tunnel	7mm	7mm	10mm	10mm
Nearby structure	20mm	15mm	40mm	30mm

Inclinometers are instruments used to measured horizontal movements in depth and are installed to control the structure and soils deformations. (de Carvalho, 2013). They were installed inside the bored piles demanding these bored piles to have 2 meters extra in order to assure its base as a stable point.

In the AB wall both inclinometers suffered damage doing the piles construction. So in the first measurement displayed an unconformity of horizontal displacements comparing with the ones presented by the topographic targets, in particular at the depth of the crown beam. The inclinometer I4 showed displacements in the crown beam in the order of 29mm. High values that lead to the works to stop. The I5 presented displacements towards the centre of the excavation inferiors to 10mm, which shows that both I5 and AB2 displayed similar displacements.

Topographical targets were installed in 3 different elevations, as well as in the neighbouring structures. During the weekly readings the displacements in the 2 targets at the crown beam, were relatively low being under the parameters previously defined. However after February 4 the movements towards the centre of the excavation site started to increase to higher values.

In the target AB1 the horizontal displacements (yy) towards the centre of excavations increased from 2mm at February 11th to 15mm a value that surpass the alarm criteria to this depth. The target AB2 also had an increase of the horizontal displacements but its values raised from 2mm to 7mm, smaller values that go according to the project. For the other directions both targets presented displacements below the alert displacements. The displacements present in AB1 target are similar to the ones presented by the I4 inclinometer, Figure 4.

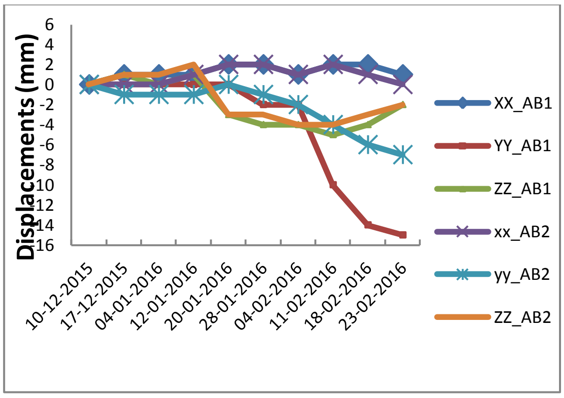

Figure 4 Target AB1 displacements.

Metro tunnel is a sensitive infrastructure that needs special attention due to its high sensitivity to displacements. So there were installed 18 topographic targets along the 62m of length that the AB wall.

From the measuring made until the date, the metro tunnel does not present displacements in any direction bigger than 5mm, which show that the tunnel is displaying a good behaviour, Figure 5.

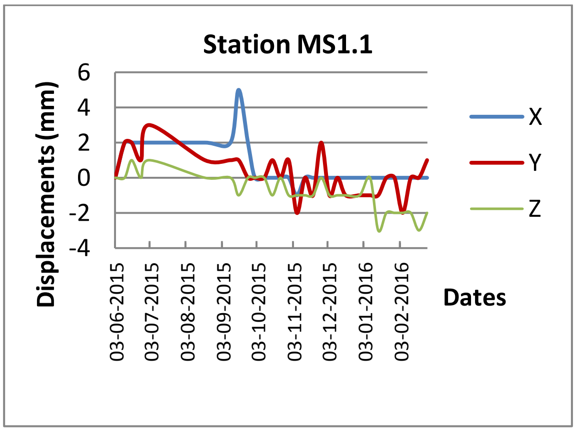

Figure 5 Tunnel (MS1.1 station) displacements.

4. Solution Modelling

The *software Plaxis 2D* is usually used to analyze different aspects of geotechnical structures, including the construction process with a good precision. In this paper there developed made two numerical models, one using the geotechnical values of the soil from the SPT tests and another from the MASW tests, both have the same structure with 18m of excavation depth, peripheral retaining structure made by bored piles with 0,6m of diameter and apart from each other 1,20m.

The mesh of finite elements is also the same to both models, which is an isoperimetric triangular element of 15 knots and it's used a window of 50x45m.The water level it's a depth of 22,3m and the surcharge load used is about 10kPa.

The constitutive model used was the Hardening Soil Model because it considers the soil hardening and plasticity.

4.1. Structural modelling with values from SPT test

In this model two layers were used, one represents the landfill and the other represents "*Argilas dos Prazeres*" simplified, grouping the four layers described before.

The tunnel was represented by a thickness of 0,7m, axial stiffness (EA) of 2,3e+07 kN/m and bending stiffness(EI) of 9,4e+05 kNm^2/m.

The bored piles curtain was characterized by the values of axial stiffness (EA) of 1,2e+07 kN/m and a bending stiffness(EI) of 2,6e+05kNm^2/m.

To calculate the slab bands stiffness, it was selected the central 44m horizontal span point because it corresponded the none flexible point with constant, from there it was calculated the stiffness (EA) obtaining a value of 78340,9 kN/m^2.

Table 3 Soil Parameters used in *Plaxis software*.

Parameters	*Landfill*	*Clay Layer*
γ_{unsat} [kN/m^3]	19	20
γ_{sat} [kN/m^3]	19	20
E_{50}^{ref}[kN/m^2]	5000	42500
E_{oed}^{ref}[kN/m^2]	5000	42500
E_{ur}^{ref}[kN/m^2]	15000	127500
c'[kN/m^2]	-	17
ф'[°]	25	35
m[-]	0,5	0,5
Ψ	0	0
v_{ur}	0,2	0,2
k0	0,577	0,426

The construction process defined in the software was similar to the one characterized in the project, with excavations levels varying between 1,5m and 2,3m.

The obtained results were inferior to the displacements from the alert criteria. The soil maximum horizontal displacement occurred below the second level of slab bands with a value of 36,16mm towards the centre of excavation and the maximums vertical displacements was 40,38mm at the base of the excavation.

The study of wall displacements shown that the horizontal displacements grow in depth reducing at the slab bands levels, its maximum value is 36,16mm and bellow the second slab bands because that is the biggest vertical span presented. The maximum vertical displacement has value of 7mm. The longitudinal forces in 1st and 2nd slab bands are 235,4 kN/m and 253,5 kN/m, Figure 6.

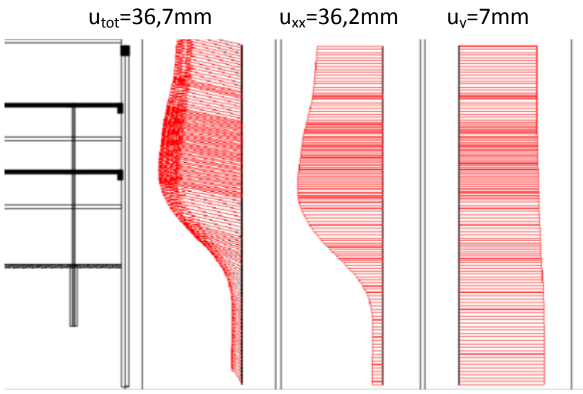

Figure 6 Wall displacements, approach 1.

The metro tunnel also showed acceptable displacements in both direction with a maximum value of 7,5mm, assuring its safety Figure 7.

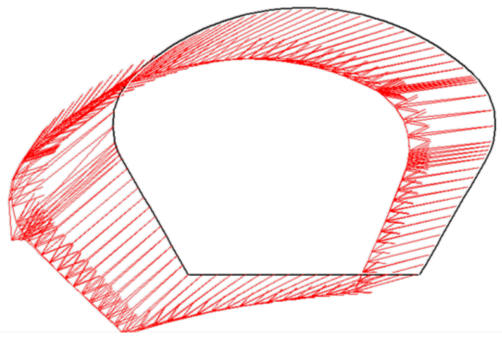

Figure 7 Metro tunnel displacements.

4.2. Structural modelling with values from MASW test

Before developing a model using the values obtained by the MASW tests, it's necessary to interpret the data. The MASW tests study the small deformations providing soil shear velocities, however the uncertainty increases with the depth. To study the soil behaviour two approaches were evaluated and the final soil profile was a combination of the both approaches.

The 1st approach only studies the MASW method, and allows studying the soil until a depth of 12m, with short uncertainty. Deeper layers showed an increase of the uncertainty making difficult to characterize this layer. In the first 12m it was possible to identify 3 layers (Figure 8), the first with the lower value of Vs of 200 m/s can be defined as landfill and the second extends until the 12m with a value of 400m /s, which is a reasonable value for a clay layer. The value of the next layer shows the uncertainty problem by defining a layer with a value of Vs of 1200m/s, because the SPT tests do not corroborate this with it.

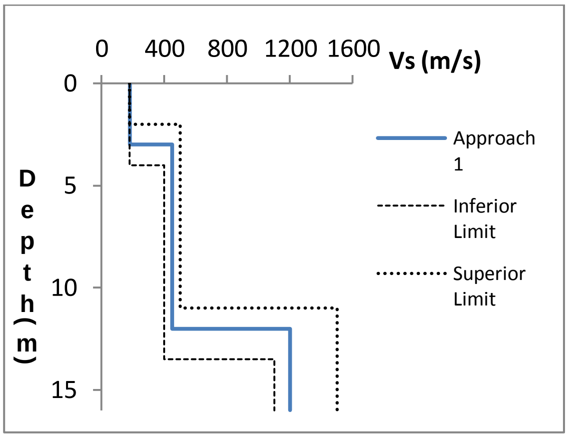

Figure 8 Vs-Depth graphic from Approach 1.

To correct the overestimation of Vs values at the deeper layers, it was study another approach that conjoins the MASW tests with the study of Rayleigh wave's elipticity. The use of Rayleigh waves at low frequencies improve the observation field and decrease the number of profiles adjustable to the dispersion curve. The depth of this approach increases to 80m, though after the 25m the uncertainty grows again.

The defined profile shows 5 substrates (Figure 9) between the ground surface level and the 80m depth. This approach also has a landfill layer with a thickness of approximately 3m and a value of Vs around 180 m/s. The second layer extents to 22m of depth with a Vs value of 400 m/s, after the third layer the value is more dispersed but it was possible to divided in 2 different layers with Vs values of 550m/s and 650m/s. Lastly the fifth layer lengthens between 52m and 80m with Vs similar to rock of 1200m/s.

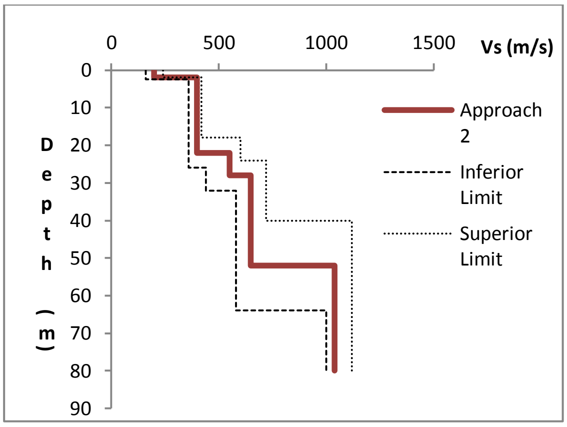

Figure 9 Vs-Depth graphic from Approach 2.

The final profile is a comparison of both approaches. Simplified to just 3 layers at a maximum depth of 30, when compared with the profiles obtained by the values from the SPT, it's possible to back up and accept these values, shown in the Figure 10.

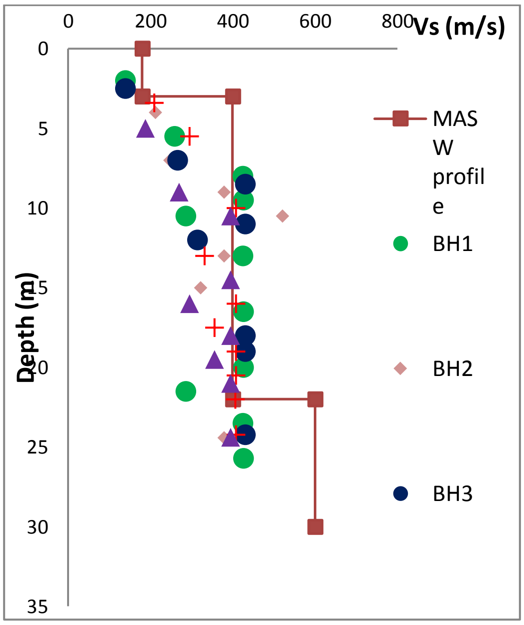

Figure 10 Vs-Depth of boreholes and MASW profile.

With the Vs of the layers it's possible to obtain the Young's modulus value, thought the following calculations.

$$E = 2G(1 + \upsilon), \qquad (1)$$

$$G = \rho Vs^2 \qquad (2)$$

Table 4 Summary of Young's modulus obtained.

Layer	Depth (m)	Vs (m/s)	G (MPa)	E (MPa)	Emáx (MPa)
1	2.5	180	52,9	137,5	10
2	22	400	310,2	806,5	60
3	30	600	734,7	1910,2	120

The values obtained were rather high, because they were obtained from small deformation state so that was necessary to define the maximum values of young modulus deployable in the soils, depending of the decompression state. Besides the Young modulus of the soil layers the wall, tunnel and slab bands characteristics remain the same used in the previous numerical model.

The ground and wall displacements obtained were smaller than the ones obtained through the previous modelling, Figure 11. The behaviour of the soil was similar and the place where the biggest horizontal displacement of 37,71mm was the same, below the 2nd slab band. The progress of walls displacement were the foreseen with a maximum value of 33mm towards the centre of excavation, the vertical displacements slowly increase in the depth until a maximum 2,8mm.

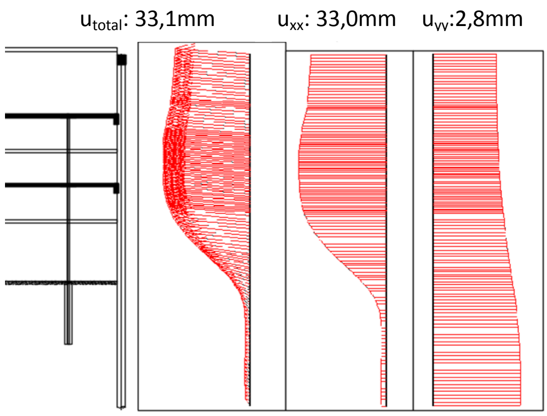

Figure 11 Wall displacements, approach 2.

The metro tunnel displayed displacements of 8,3mm, slightly bigger than the ones defined in the alert criteria. However it's important to notice that in structures such as the metro tunnel demand a more accurate modelling than the one used in this paper. Nevertheless these values are acceptable as just one value surpasses the alert criteria and it is assumed that the displacements don't increase.

4.3. Comparative analysis

Comparing the displacements obtained in the Structural model with values assessed from SPT tests and MASW test, it was predictable that the former were bigger, proving however that the difference between values weren't as big as supposed, probably because in the model using MASW test values the used Young modulus values were smaller (closer to reality) than those obtained from the calculations. It's also important to refer that both tests lead to values smaller than the alert criteria. If the progress of the construction site were made according to the project, the displacements measured at site would have been smaller than the predicted by the numerical model.

The displacements obtained closer to the surface ground level were bigger than expected, this may have happen because the retaining structure with a had a bigger waiting time than expected without support, leading to a soil decompression without any kind of system that help to minimize those displacements.

Figure 12 shows the displacements measure in the inclinometers at different days, as well as, the displacements calculated by both numerical models. Figure it shows a maximum depth of 5m, correspondent to its the current depth of the excavation.

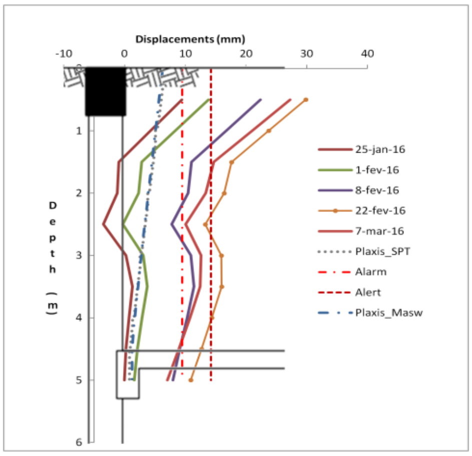

Figure 12 Comparison of the different displacements.

5. Main Conclusions

The targets proposed for this paper were mainly achieved, just missing the monitoring the construction of a slab band, as well as the excavation to the final depth.

The numerical modelling concluded that the displacements obtained through values assessed from the SPT tests were higher than the values obtained by MASW tests. However, it should be noticed that the version of Plaxis software does not perform a 3D analysis thus creating limitations in the study of the slab bands 3D effects.

The studies of the monitoring plan results were very important and it must never be neglected because as proven by the case study thus helps to detect excessive displacements that made.

The site visits helped to better understand the construction techniques described in the theory and realise the need to readjust the main solutions according to the new information achieved.

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