Retaining Structures with maintenance of Existing Walls

– Case Study of Solar de Santana 60-65, in Lisbon –

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Abstract: This project development is based in the construction of retaining walls located in high populated areas as the city centres. The growing of need useful areas lead to the increasing of buildings volume, which are expressed in the expansion to heights below the existing structural bases. The referenced cases have special issue centred in the reforcement and reuse of existing vertical elements that imply the adjustment of the different projects (existing walls containment, retaining wall and building final structure) to be harmonized. The need of special attention on this kind of works falls on the precarious characteristics of the materials to mantain, and the surrounding environment where that buildings are usually located. Generally, that leads to high sensitivity situations where there are almost zero tolerance to movements promoted by the progress of the works. In addition to theorical context about the aplyed methods and their bases, there was a permanent attendance in the construction that led to an active control of it. In this subject, these constant field data collections and the monitoring of the works, allowed to justify the employed methods and the changes to the original project.

Key Words: Urban excavation; Berlin-type retaining walls; Wall strengthening techniques; Monitoring

1. FRAMEWORK

The EC7 predict the existence of three types of structures with characteristics that meet the needs, being these ones divided by the performance methodology in: Gravity Structures, Flexible structures, and the ones that are a mix of the two before. The gravitic ones depends actively of their weight, being useless in city applications caused by their high volume. The mechanism formed are a compact rigid block where the ground forces applied can be calculated by simple and classic methods. (e.g.: Gabion walls, concrete or masonry elements). In opposition there are reduced thickness solutions with insignificant weight, known as flexible structures or walls, that functionally mostly with bending moments. These ones generate a new problem due to the way they act, a soil/structure interaction issue. Therefore, are needed a compatibilization with the service deformation of the curtain and the capacity of absorption or accompaniment of the movement by the ground around it. (e.g.: Slurry Walls, Sheet Pile Walls, Berliner Walls (temporary or definitive).

1.1. EARTH PRESSURES

Known the main objective of retaining the surrounding earth pressure, Coulomb present a simple way of calculus. However, it has a more

adjustable application to elements operated by gravity, since it preconized a plastic behaviour of the material with movements of transaction or rotation by the base.

That method even has served to base for EC7, whose considered three main states of tension. Two in the limit zone (active and passive) and one zero state of balance, which simulates a situation with no movements.

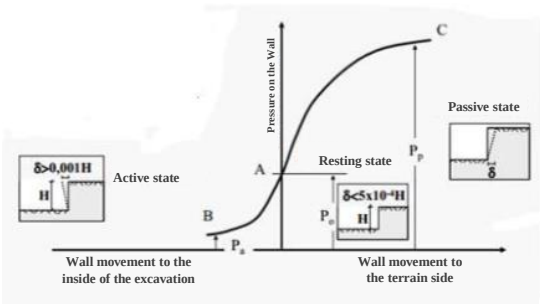


Figure 1 - State mobilization diagram.

In the graph presented above, are notable that for passive mobilization, represented by the smashing of the ground in the back of the curtain, the required movement is higher than for an active state, where the retaining wall tend to the interior of the excavation area. (magnitude can vary up to ten times). Being the earth pressure value generally:

$$I = \frac{1}{2} K \gamma H^2$$

That expression will depend of volumic weigh of soil (γ), the height digged (H) and the soil frition angle ($\emptyset'$), beeing the last one withheld in the expression of earth pressure coefficient (K), which depends on type of terrain mobilization.

Table 1 - Earth pressure

Active state	Passive state	Neutral State
$K_a = \frac{1 - \sin \emptyset'}{1 + \sin \emptyset'}$	$K_p = \frac{1 + \sin \emptyset'}{1 - \sin \emptyset'}$	$K_0 = 1 - \sin \emptyset'$

There are even mechanisms based in the plasticity theories which presume failure of the soil in general or through a breaking surface. Those involve collapse loads and them evaluation can be divided in limit analysis (TRI and TRS) and balance analysis (EL).

In flexible structures exist either a change in pressure diagram and induced forces promoted by the cambering deformation of the structure.

The camber effect of flexible structures promotes non-uniform deformations in the soil mass, leaving the triangular diagrams to be disabled by the changes percuted in pressure distribution and induced forces.

Stimulated by the inexistent way of calculating the forces acting on this type of structure, led Terzaghi and Peck to perform an experimental methodology, based in the measuring of the effort installed on locking devices who hind the movement of the walls. This study, that has resists until now, allow the rating of pressures by apparent diagrams which are preponderant in pre-dimensioning situations and leads to acceptable values.

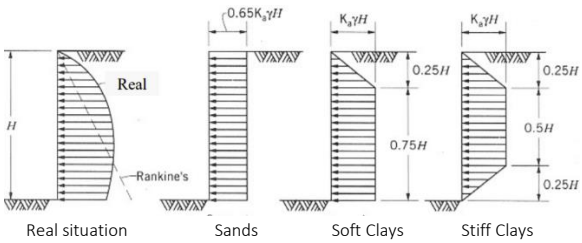


Figure 2 - Apparent pressure diagram.

1.2. GROUND ANCHORS DESIGN

In general, that kind of thin reinforced concrete structures don't have enough strength by himself to sustain the earth pressures installed, being reinforced by locking devices as anchors or embedment's of passive type.

Both methods presume a temporary use, being dismantled when the construction have enough self-supporting capacity.

The embedments of passive type are formed by timberings which, despite its economic character, have a limited use derived to the high space consumption. In alternative, are used ground anchors which, although release space in the construction side, it invades the neighbour's terrain changing it in their crossing.

According to Hanna in Metalana, the behaviour of the anchor is connected by its own morphology, being the occurrence of minimum movements associated with the non-inclined (horizontal execution). However, the lack of strenght soil on the surface restricts their design to an incline projection. In this field is demanded a balance between the inclination and length, to perform well fixed and functional devices, considering the economic factor as well. In a later study, Hanna and Abu Taleb also concluded that an 30º inclined anchor behaves similarly to the horizontal ones (with slope of 0 °).

1.3. GROUND ANCHORS BEHAVIOR

The functioning of an anchor acts by the change of tension state imposed on crossed ground. In the next graph are compared the impact of using or not that methodology in the execution of multiple level retaining panels.

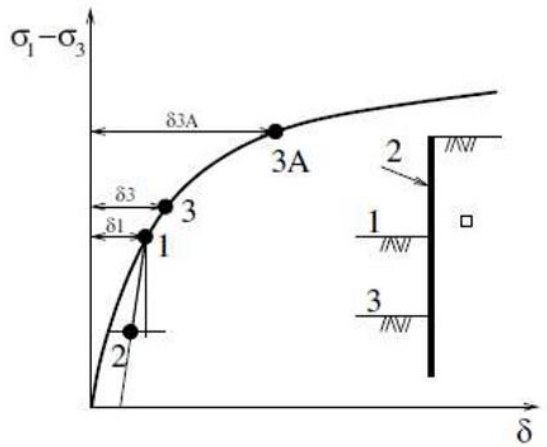


Figure 3 - Deformation/displacement tension graph by anchors

The difference in mass movement is remarkable. In the case of non-use of the anchor after level 3 excavation, the movement of the contained soil is much more evident ($\delta 3A$). On the other hand, if, after excavation of level 1, an anchor execution is considered in step 2, the movement caused by the opening of level 3 will be controlled with a smaller impact on the retained terrain ($\delta 3$).

Having all that into count, are predicted in the standard NP-EN:1997-1(2010) specific factors to be concerned in the base of a geotechnical project, being them also verified to limits states of failure.

As special elements that can define the level of security and impact of a construction, their execution requires compliance of many quality standards (performance and composition).

Established by EN 1537 (2013), all anchors are checked for resistance to deformation. Here, they are subjected to loading and unloading cycles and are requested to load increments until the maximum test load is reached.

The composition of these is also tested for fluidity, exudation, volume variation and compressive strength, according to NP-EN 445 (2008).

Existing also boundaries for his components, allowing by the norm EN447 (2008) a variation of 2% in the added cement and 1% in the added water. The inclusion of additives is still contemplated in EN1537 (2013), if they improve the initial characteristics of the blend.

1.4. IMPACT IN NEIGHBORHOOD

In general, the decompression results in the modification of the tension state of the soil. Although little displacements or integral rigid rotations don't compromise the structural security

neither daily routines, major damages can happen if a distortional action take place from differential settlements. His source can be horizontal, inducing compressions and extensions, or vertical, inducing flexion and sectioning. This kind of motions take place in the structure's bases, creating cracks and crevices that manifest themselves in less structurally contained areas, such as windows.

Conservatively, an indirect control of the influence in the neighbourhood is taken by limitation of the movement of the retaining structure, since its propagation is an iterative process of difficult conception. In dimensioning's thematic, being assume that the displacements are completely transmitted to the adjacent constructions, is necessary to limit the apparent stiffness of the dimensioned structures (through safety coefficients), as well as to invest in their monitoring.

1.5. WALL REINFORCEMENT

The normative imposition on maintenance of the public interest constructions transformed the reality of Portuguese market, following the transformation of existing buildings into new and more contemporary uses.

The rehabilitation of this building heritage, along with the underground works, entails the execution and design of unique solutions with highly specialized labours. The modifications inflicted, generally imply the change of the conditions of support with the growth of floors and overloads, reflected in the redistribution of tensions. Taking this into account, it is necessary to ensure the structural integrity of the elements held, either during or after intervention. In this way several methodologies are applied which, not only concerned about the state of conservation of the original materials but also the compatibility of execution with the technique in question.

1.6. METHODOLOGY REINFORCEMENT

Technics like grout injection and sewing nails are used to increase the strength, rigidity and monolithism of the elements. This kind of solution are based in the injection of grout or fluid cements in pre-drilled holes, with the addiction of reinforced bars in case of nails. That one could also contain a metal plates on the face to transversal confinement, being known as connector if it crossed-over totally the walls. All this method is intrusive and irreversible, requiring yet a careful consideration in terms of compatibility of the characteristics of the grout with the existing materials.

Another way to improving the original characteristics of the walls is to apply a new layer to the face. That can be achieved by armed plasters or sheets of reinforced concrete. Both cases increase the seismic and cohesive capacity in the elements as well as limit excessive lateral deformation of them. Although the concrete solution has a greater resistance capacity than the plastering, it implies significative increase in the structure's weight, rising the vertical and seismic efforts. This technics of adding an external layer are irreversible too, once their removal would result in loss of material and section.

For general application of immediate strength, can be use global strapping that increase the three-dimensional response of the structure, minimizing their risk of collapse. The methodology is based on the fixing of metal beams in the masonry, placed on the both sides of the Wall and linked by connectors. Its main advantage is the ease of application and possibility of removal when the building acquires self-supporting capacity.

In summary, the type of facade to be contained may present an enumerated variety of pathologies, existing or percuted, mainly due to the demolition of internal structures. Realizing that there is not an optimal solution, the current executions are due to the conjuration of the several methods presented, depending on the application of the characteristics of the building or even of the specific zone where it will be applied.

2. CASE STUDY

The main basis of this study is related to the developed solutions proposed for a palatial building dating back to the 17th century. This is located at the top of the garden of Campo Mártires da Pátria, privileged place on the highest point of one of the seven hills of Lisbon.

The works involved are of the facadism type, where the vertical elements of the structures are maintained, and the interior demolished for later requalification. However, this proved not to be an ordinary job due to the antiquity of the building and the maintenance of a large part of the original walls, which forces them to be reinforced and re-supported, in order to create a buried floor for parking.

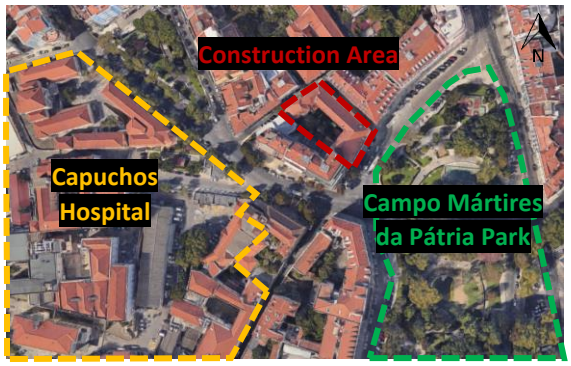


Figure 4 - Aerial view (google maps, 2018)

2.1. RESTRAINS

The works in questions implies a wide variety of technical constraints, imposed by dense environmental building surrounding and unconverted structures ti maintain.

Structurally, the building requires an active reinforcement of the walls to be maintained, through the installation of a global metallic shoring in order to guarantee their stability. In terms of architecture, the keeping of external look like implied a re-support of the wall's bases, by raising them in the air by micropilles which sustained them during the execution of the underground floor. Knowing the impact of the works in the normal function of then neighbourhood should be limited, in the case of affecting the domestic service networks, these should be diverted.

Also, must be in count unpredicted events linked to the tasks progress as over-excavation, work-delays, unplanned shakes or overloads, insufficient support and the sequence of the constructive methods.

2.2. SOLUTION PROPOSED

The solution predicted involves on reinforcement and re-support of the existing façades to be maintained. Through a sewing mechanism that confine them between re-support beams, as possible to transfer the loads form original wall bases to micropilles to be executed.

Than the digging of the parking floor take place with a maximum depth of 3.5 meters, being the peripheric terrains sustained by Berliner solution, locked by one level of ground anchor and corner shoring. The referred anchors will be composed by 4 cords of 0.6'' and executed with sealing bulb of 7 meters (by an IRS system) and a slope of 30º, to reach the consistent soils.

The retaining wall will be concreted against the terrain as usual, starting with the execution of

primary panels with maintenance of buttresses to mobilize an arc effect, being than proceeded by larger secondary panels that close the containment. In case of possible excavation of the back in slope, the retaining structure will be executed as a normal reinforced concrete wall with posterior waterproofing asphalt screens installation and embankment of that zone.

Since the design phase was foreseen an instrumental an observational plan to manage and prevent the risks and keep the work in the safety and financial boundaries, too.

This constant monitoring allows a real-time control of the variance of the admitted assumptions, quantifying and qualifying the main associated risks.

In those campaign is predicted topographic targets and rules to measure the horizontal and verticals displacements, inclinometers to measure the vertical moves of the keeped facades, and load cells that not only measure the tension installed in the anchor, but also allow an indirect control of horizontal displacements of the contained soil mass. Following the displacements, the next criterias:

Table 2 – Displacement criteria

	Horizontal	Vertical
Alert	15 mm	10 mm
Alarm	30 m	20 m

2.3. GEOTECNICAL INVESTIGATION

To define the main characteristics of the ground was executed five geotechnical trials (S_1 to S_5) using the Hollow Steam Auger method, which allows the execution of dynamic penetration tests (SPT), being collected soil samples to further chemical and granular test. In some drills was also include a piezometer of open circuit to check the possible water levels changes.

Adding to that, was carried out a campaign of standardized penetration test (SPT), to characterize "in situ" the level of compaction (granular soils) and of consistency (cohesive soils).

After topographic levelling, was verified a landfill layer (A_t) characterized by a very compact soil ($8 \leq N_{SPT} \leq 34$), having either some higher records justified by the stony character of that landfill.

Bellow this layer establish the miocene substract represented by the local unit of "Areolas da Estefânia" (M_{ES}). That take form by the presence of sandy silts with abundant level of very resistance rocks. Globally classified as hard soils (NSpt≥60), although interleaved with occasional layers of decompressed miocene device (30≤NSPT≤46).

Table 3 – Stratigraphic Column

Stratigraphy	Symbol	Formation	Litology
Contemporary	At	Landfill	Landfill deposit, a bit stony
Miocene	M_{ES}	Areolas da Estefânia	Sandy-Silt, thin, with rocky layers (brownish)
			Sandy-Silt, thin (brownish)
			Silty-Clays (grayish)

Although the hydrogeological device proves to be positive, the water has no interaction with the construction, being well below the excavational bottom level.

The surface layer is then composed of deformable material and of low resistance capacity which, despite the stony sandy-silt formation, did not serve to the presupposes of the project. On the other hand, the miocene device, although somewhat decompressed, present a mechanical behaviour appropriated to support of the load transmitted by the structure bases. In this way, it must be ensured that all foundations clearly exceed the landfill layer to the contact stress be mobilized in the miocenic layer.

2.4. WORKING SEQUENCE

Executed after general strutting of the building by the introduction of metal profiles that confine them transversely, the demolition operation began by removing of the inner and outer plasters to be possible a reinforcement of the facings. This was followed by the complete demolition of the annexes and the internal structures of the three blocks. The second one was executed from top to bottom, beginning with the dismantling and removal of the cover, followed by demolition of the interior walls and floors, sequentially, floor to floor, removed by sliced sections with the help of the crane in order to avoid collapse of them. In parallel to the demolition operation, reinforcement of the façades was carried out in such a way as to maintain the conditions of security and balance of the same.

Subsequently, work began on the execution of micropilles, starting with those that would serve as new support for the façade. The harmonized behaviour of these are warranted by beams concreted against the walls, and later fixed by a sewing mechanism composed by pre-stressed bars from GEWI type. These cross walls and both beams in order to confine them and thus ensure monolithic behaviour of the solution now based on the executed micropilles.

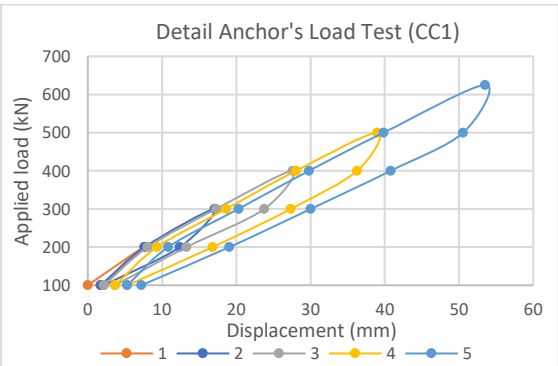
The sequence of works pertaining to the execution of micropilles was divided into drilling, introduction of reinforcement frame, sealing and injection. The drilling was performed by the short auger method where, after verification of the angle of attack helical sections would be introduced until be reached the desired dimension. Then, the main frame was introduced, consisted by high strength steel tubes (N80) also introduced by sections where the connections are guaranteed by threaded rings. Subsequently, the introduced frame was sealed against the ground, requiring a drying from 12h to 24h, which preceded the injection, executed in IRS regime and with a maximum stipulated pressure of 30bar. It uses a double shutter and non-return valves in which, the execution level by level allows the selection of the tube zone to be injected in order to achieve a suitable sealing bulb which fixes the element to the ground.

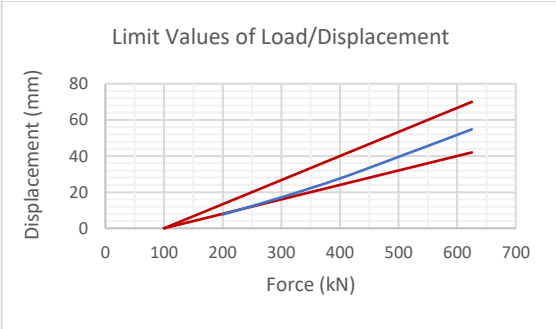
In relation to the cement mix used for sealing and injection, a type I 42.5R cement with an A / C (water/cement ratio) of 0.4 is recommended, being removed 4 cubes per type of blend. These will be subject to an RCU test in a properly accredited laboratory and must present a resistance of 27MPa at 27 days.

With the development of the process of excavation of the underground floor, the work of peripheral containment was initiated, starting by the opening of the primary Berlin-type containment panels to be applied in block B. However, these would not be executed in a traditional way since the ground found during the excavation presents better skills than those considered in the project design. Thus, in a general way, the constructive project goes through the opening of the total height of the panel to be executed followed by execution of Shallow foundations where micropilles heads are prepared to be within those. Next, the panel are armed, with fixings to elements who will support the vertical charges and concreted against the terrain leaving some free height corresponding to the crown beam. These one could be executed latter with the top floor of parking level to promote a better and effective connection between retaining wall and the new structure, once the containment was executed from the bottom up.

After the execution of the panels, the anchors were applied to those for which these types of work would be foreseen being left a negative one the Berliner executed. The drilling was performed with a slope of 30 °, through the phased introduction of bit rods with a water circulation system installed to cleaning the hole. Subsequently, the reinforcement was introduced, consisting of four cables of 139 mm^2 each, followed by the sealing until the fluid pops out of the tube mouth without any trace of soil or sludge. After a drying from 12h to 24h, the sealing bulb is executed, this also through an IRS system. Once the device has been executed, a hold time of about 7 days is required before the anchorage gains enough skills to receive the set pre-stress load. The application of the charge is printed on the head of the anchor by a hydraulic jack (T4 / 8) to which a given pressure regime is associated with a force applied.

During this process a receive test is performed in which the displacement inherent to the applied charge cycle is recorded. In the case observed, several loading and unloading cycles were applied, reaching a maximum value of 625 kN, a value about 25% higher than the one that is intended to be installed. On these, a continuous data collection is running where a quantitatively and qualitatively evaluation is keeped according to referenced boundaries.





After a greater confinement of the area was possible to proceed with the excavation and remove the original bases of the pillars that support the arches.

At those time, perform an unplanned work related to the seismic reinforcement of the stone pillars. Initially designed with a reinforcement through a layer of armed concrete, was during the demolition that it was faced with the high aesthetic and patrimonial value of the blocks of white stone that formed the pillars of the arcades. It was then decided that his aesthetics would have to prevail to the reinforcement method to be applied, being proposed, by Zircom, a solution corresponding to an inverted drilling and subsequent nailing using the elastic sleeve. The solution consists of a structural element made of stainless steel inserted in an elastic sleeve. It has the function of containing the injection grout, allowing it to take the morphology of the voids caused by the drilling, but preventing the migration of the fluid to the support that could contaminate the masonry or the plasters. The total length of the reinforcement also predicts a free length below the base of the pillar where a metal plate will be placed to later insertion in the slab. Such detail ensures the occurrence of a monolithic behaviour of the slab/pillar system.

With the growth of the top floor of the parking area, the monolithic behaviour of the structure was gradually increasing, acquiring a self-supporting capacity with the conclusion of the said work. As a

result of that, is the structure capability of absorbing the external earth pressures imposed being possible the deactivation of some support devices. Here, the cutting of anchorages and metal shoring's was carried out as well as the demolition of the beams that supported the walls maintained, during the excavation phase.

2.5. DISPLACEMENT ANALYSIS

In order to obtain a reliable source to serve as a basis in the decision process, a monitoring campaign was developed, playing an active role in the control of the construction phase. Those provides essential information for the control and evaluation of structural integrity, measuring in real time its behaviour evolution. Through this, it was possible to associate some events that may have some impact on the variation of the measured measures, due to their importance or magnitude.

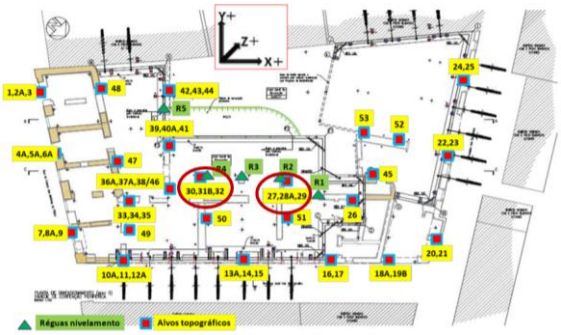


Figure 5- Location of topographic targets and levelling slides.

In general, the structure showed a partially uniform movement at XX ' since the internal demolition phase, until a reversal of the trend began in early March. This succeed also for the YY movement, especially in targets 27 and 30, where it passes the alert criteria (15mm), but not meeting or exceeding the alarm criteria (30mm).

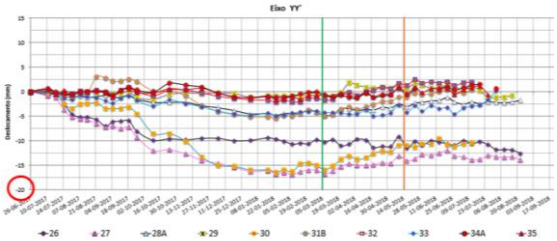


Figure 6 - Displacement YY from topographic targets nº26 tonº35 (mm).

The target 19 also showed a highlighting role since it was the one that presented values closer to the alert criteria. Its movement was initiated during the excavation operation, reaching a maximum exponent in January 2018, date from which it stabilized, keeping his variation inactive until the last reading made.

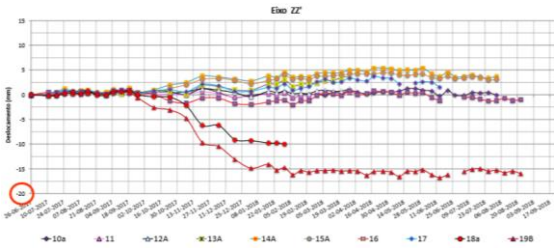


Figure 7 - Displacement ZZ from topographic targets nº10 tonº19 (mm).

After the evaluation of the preponderant results readings, described once they showed greater absolute variations, it can be concluded that the work is in perfect conditions of structural safety, also following good practices regarding the impacts in the neighbourhood. This assertion is further reinforced by the reading of the inclinometer and load cells, that not presented relevant values for the soil displacements. Regarding the load cells, they only showed residual variations during the execution of the work whose magnitude did not exceed 10%, indicative value for a well dimensioned device.

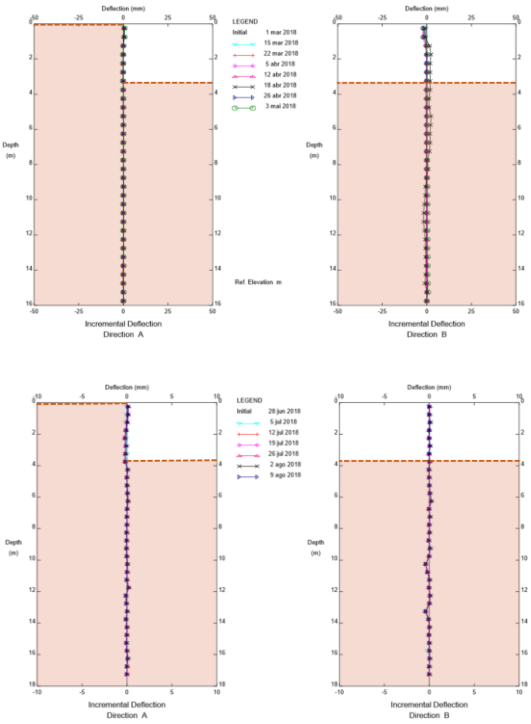


Figure 8 - Incremental graph of inclinometers (mm).

As a commentary on this chapter and after the evaluation of the monitoring data's, is possible to conclude that the solution predicted of Berliner walls is over dimensioned. Although nowadays there are better technic and economic solutions, those could not be applied here, derivate to the space restrains. That way, the executed solution was the one that better adapts to the constrains of this specific build but can be optimized.

3. MAIN CONCLUSIONS

The consideration of problems of geotechnical character has taken special attention at the present time, there being several cases that depend on their consideration. Its difficulty of forecasting and dimensioning leads to the need to create innovative proposals that are favourable in economic, environmental and safety terms. In order to optimize these factors and minimize the errors associated to the design, it is necessary to invest not only in detailed preliminary studies to characterize the base system, but also invest in an efficient and effective instrumentation and observation system whose configuration allows for confirmation of the considerations taken and also, if necessary, to proceed preventively in the revision or alteration of the project within the pre-established safety and monetary parameters. The boundaries control mentioned execution phase also has a decisive role in the financial stability of the work since the increasing competitiveness of the global markets, allows few flexibilities of timings and budgets in the tender phase, entailing the responsibility of occurrence of profit or loss for the execution phase.

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