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Diseño y construcción de muro Berlinés para estacionamiento subterráneo de grandes dimensiones

Design and construction of an anchored soldier pile wall for a large underground car park

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Abstract

The geotechnical conditions of the soil and the construction conditions of a retaining wall for an underground two-level car park project are described. The project is located in Concepción's downtown, south of Chile. The excavation support had to prevent any damage for the Courts of Justice historic building and surrounding buildings. These are buildings between four and nine floors and the Hites building is also classified as a historical building. Also, any cutoff of water, sewage, gas and electricity had to be avoided. The solution adopted was an anchored soldier pile wall of 8 m depth around the excavation, where the soldier piles (H section steel piles) were driven into semi dense silty sand. Two lines of anchors were designed vertically and horizontally separated by 3 m and 3.2 m respectively. The design loads for the anchors ranged between 300 and 560 kN respectively. The installation and placement of anchors was studied in order not to disturb the different stages of construction. The project included 3596 m^2 of anchored soldier pile wall with 314 post-stressed anchors and 300 soldier piles totalling 3200 m. It is concluded that the temporary excavation support solution adopted performed properly to the high demands set, since no important deformations were noted in the building of Courts of Justice, in the surrounding buildings or in the services of drinking water, power, gas or electricity. The city of Concepción was severely struck by an 8.8 moment magnitude earthquake on the 27th February 2010. The construction of the underground car park was just ready when the earthquake occurred. No damage was observed in the car park due to the seismic event. It is believed that the buried soldier pile wall reduced the seismic loads acting on the underground car park structure since no evidence of damage exists after the very big earthquake.

Keywords: Anchored soldier pile wall, silty sandy soil, excavation support construction sequence.

Resumen

Se describen las condiciones geotécnicas del suelo y las condiciones constructivas de una estructura de contención para un proyecto de estacionamientos subterráneos de dos niveles. El proyecto está ubicado en el centro de la ciudad de Concepción, en el sur de Chile. La entibación tuvo que impedir deformaciones perjudiciales para el edificio histórico de los Tribunales de Justicia y también de otros edificios circundantes. Estos edificios tienen alturas que varían entre cuatro y nueve pisos y el edificio Hites también está clasificado como edificio histórico. Se debe evitar cualquier interrupción de los servicios de agua potable, alcantarillado, gas y electricidad. Se adoptó como solución de contención un muro tipo Berlinés anclado de 8 m de profundidad, compuesto por perfiles H hincados en arena limosa semidensa. Se utilizaron dos líneas de anclajes separados horizontalmente 3.2 m y verticalmente 3.0 m, cuya carga varió entre 300 y 560 kN, respectivamente. El emplazamiento de los anclajes se estudió con la finalidad de facilitar la ejecución de las diferentes etapas constructivas del proyecto. Este contempló 3.596 m^2 de muro Berlinés anclado con 314 anclajes postensados y 300 perfiles H totalizando 3.200 m lineales. Se concluye que la solución de entibación temporal adoptada respondió adecuadamente a las altas exigencias impuestas, dado que no se observaron durante la construcción deformaciones importantes ni en los Tribunales ni en los edificios circundantes ni en los servicios de agua, luz, gas ni electricidad. La ciudad de Concepción fue severamente golpeada por un terremoto de magnitud de momento de 8.8 el 27 de febrero de 2010. La construcción del estacionamiento subterráneo estaba justo concluida cuando el terremoto ocurrió. No se observaron daños en el estacionamiento debido al sismo. Es posible que la entibación ya enterrada pueda haber reducido las solicitaciones sísmicas que actuaron sobre la estructura de los estacionamientos subterráneos dado que no hay evidencia de existencia de daños después de este gran terremoto.

Palabras clave: *muro Berlinés anclado, suelo areno limoso, secuencia constructiva de entibaciones.*

Introduction

Big cities are suffering worldwide the lack of space. Therefore, it results logical the use of basements since land prices are high and architectural as well as engineering conditions may lead to the use of underground space. Construction of large undergrounds is not trivial, on the contrary, they become a great challenge for the excavation stability and consequently for the stability of the surrounding constructions. This is more relevant under the presence of busy streets, important monuments and buildings. There are different construction techniques to hold excavation safely. These techniques depend mostly on the type of soil, the excavation height and adjacent structures.

This paper analyses a particular type of excavation support used in a project of an underground car park in the centre of Concepción. The city of Concepción is the capital of the Bío Bío region and is located in the south of Chile. During the last decade Concepción has had a considerable growth in the construction of housing developments, office buildings but also underground car parks, shopping centres and transport infrastructure. To sustain excavations in these projects it has been widely used a technique known as Soldier Pile Wall (SPW). SPWs are anchored because of the advantage of allowing free movement within an excavation unlike the use of struts or other shoring methods, which can take significant space inside the construction area.

An anchored SPW is a continuous and temporal support, whose design considers the geotechnical soil conditions, adjacent structures and excavation geometry, namely depth and width. The construction technique consists in driving steel H section profiles (soldier piles) into the soil before digging, with distances between them to be calculated. The H sections are also calculated with and without anchors, according to the excavation depth (construction sequence), to resist the lateral earth pressures and to control horizontal displacements. The distance between soldier piles ranges usually from 1.2 m and 3 m, 1.6 m being the most common in Concepción. This distance is also part of the excavation support calculations and the above values are a standard range of distances normally used.

Once the soldier piles are driven into the ground, down to the designed depth, forming a line or a curve, the excavation starts and timber laggings are inserted horizontally between the flanges of the H section soldier piles. In an excavation, for example 10 m wide and 3 m deep, it is highly likely that deformation calculations result in large horizontal movements of the soil, particularly close to the surface. This is due to the inherent flexibility of this type of support system, even with relatively rigid H sections, they are not stiff enough to control horizontal displacements. Anchors can be incorporated in SPWs to solve this problem, which is not related to stability nor to the capacity to hold the excavation, but to reduce soil deformations. Soil deformations are caused by a change of stresses owing to the excavation. The stresses are applied towards the excavation which may result in significant horizontal displacements if the excavation is not protected by an adequate support. As a consequence, any structure adjacent to the excavation can suffer damage such as fissures, cracks or more serious such a dislocation due to relative settlements leading to shear and moment failure of structural elements.

Anchored SPW are not recommended when the water table is high (EAB 2008). Ideally the water table should be below the SPW. However, it is possible to accept the presence of certain level of ground water when it is withdrawn with for example well points, but controlling possible transport of sediments. Attention should be paid in case of uncontrolled lowering of the water table, since it can induce undesirable relative settlements in the adjacent structures under the presence of soil layers which can suffer consolidation, such as soft clays or highly plastic soft silts or transport of fine particles which can also induce settlements.

SPWs provide enough space between timber laggings to allow the flow of ground water. In case gaps between timbers do not let pass easily the ground water, drains should be installed perforating holes in the timbers. The idea is to avoid any build up of pore water pressure behind the SPW, which could add hydrostatic or hydrodynamic lateral pressure and as a result undesirable deformations. In Concepción is customary to use well points to lower the water table in case of seepage behind the SPW. This avoids flooding and the transport of soil to the excavation.

An appropriate design of retaining structures depends significantly on the knowledge of the local geology and the geotechnical properties of the ground. Therefore, it is a key point to perform geotechnical studies as complete as possible, which can provide reliable values of the geotechnical properties of the soils to be dug, the soils below the excavation and the soils to be sustained.

This article describes and analyses the current design and construction practice of anchored SPWs, where relevant structural, geotechnical and construction issues are considered. This analysis is later on applied to the complex project of underground car park under the Tribunals. The underground car park was open only a few days before the earthquake on 27th February 2010. This earthquake of 8.8 moment magnitude was an enormous test for the project. This work is pertinent and necessary since it covers aspects of temporal retaining structures, which are scarce in technical publications.

Static loading conditions on SPW

SPWs are considered flexible retaining structures, even if the soils being retained are very dense or overconsolidated or with high stiffness. Consequently, the lateral earth pressure on a SPW has very little chance to be at rest, since soil deformations are highly likely to occur, which obviously means that the soil is not at rest. A mobilised condition should be assumed between the at rest condition and the active lateral earth pressure condition. Sowers (1979) proposed that an active lateral earth pressure develops when the maximum horizontal displacement u_{hmax} on top of a rigid wall of height h is $u_{hmax} \geq 0.002h$ in loose granular soils and $u_{hmax} \geq 0.0005h$ in dense granular soils. In the case of anchored flexible walls, the estimation of any lateral earth pressure will depend strongly on the anchor pre-stressed loads.

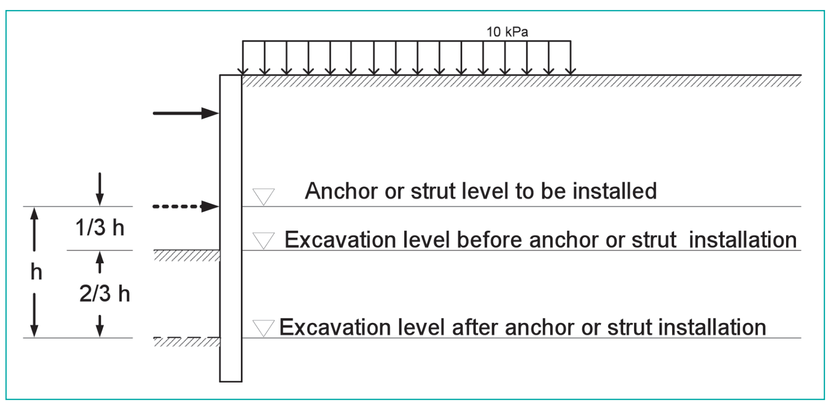
Active pressures apply behind the wall from the top to the bottom level of the excavation. Below the excavation level passive pressures apply in front of the wall from the bottom level of the excavation to the end tip of the H section piles. Passive pressures develop for maximum horizontal displacements u_{hmax} an order of magnitude less than that for active pressures; $u_{hmax} \geq 0.01h$ in loose granular soils and $u_{hmax} \geq 0.005h$ in dense granular soils (Sowers 1979). The active and passive lateral earth pressures can be calculated using the theories of Rankine and Coulomb. Both theories of plastic equilibrium assume a homogeneous soils and a Coulomb failure criterion, which is not always applicable to heterogeneous and anisotropic soils, let alone to flexible walls. Norms and codes based mostly on results from laboratory and field investigations of strain and stress measurements around walls for different soils, recommend parabolic, triangular and rectangular pressure distributions or a combination of them.

In addition to earth lateral pressures, dead and live loads can act as constant or variable loads. The EAB (2008) recommendations consider a uniform distributed load over the surface of 10 kPa, representing the effect of live loads on the pavement and street (Figure 1). It is important to highlight that the SPW calculation procedure follows the construction sequence. From the calculation results a SPW without anchors will be safe only a few metres. Figure 2 shows an example of an excavation construction sequence where it can be seen that to keep digging, the installation of a row of anchors or struts at the bottom of the initial excavation will be necessary. Once these anchors or struts are under tension, it is possible to continue with the excavation the next calculated couple of metres and then performing the installation of a second row of anchors or struts if the excavation continues another couple of metres and so on. EAB (2008) suggests that if the height from the bottom of the future excavation to the support line is h, then the anchors or struts should be installed at $h/3$ from the bottom of the current excavation, leaving obviously a distance of $2h/3$ between the current and future excavation (see Figure 1).

Force equilibrium analysis

In the force equilibrium analysis all the loads which can act on the retaining structure are included, namely earth and water pressure, as well as dead loads of surrounding buildings, live loads of streets and possible earthquakes. Horizontal forces equilibrium within the height of the excavation is considered for the excavation support

Figure 1 *Excavation limit before installing anchors or struts (EAB 2008)*

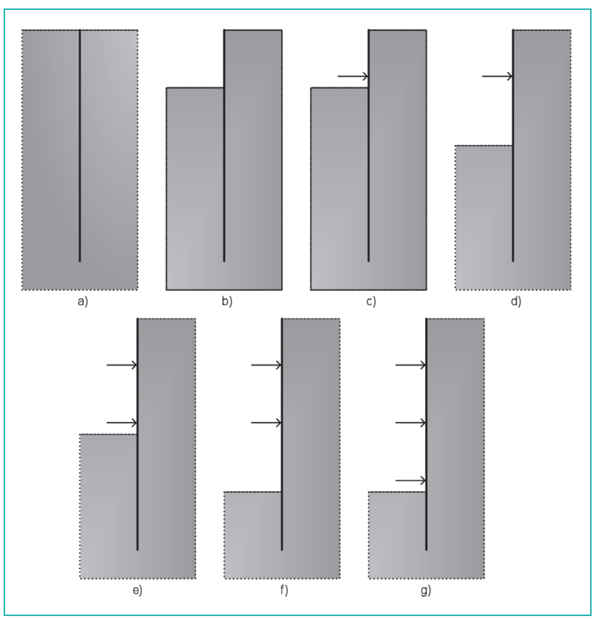


design using a SPW. The interaction between the soil and the wall (soil under passive pressure) has to resist the active pressures coming through the soil behind. As a result of the horizontal force equilibrium analysis plus moment equilibrium analysis, the embedding depth of the H section soldier piles is to be determined too (see Figures 3 and 4). In the following sections it will be indicated that the force equilibrium analysis is performed using a software due to the complexity of the problem.

The tribunals excavation support project

The car park project under the Justice Tribunals of Concepción was a great challenge not only for the large excavation and following construction, but also because of the central location, in the middle of the city. The Tribunals architecture and location are emblematic, the building has a quarter circle shape and is a reinforced concrete structure with masonry confined shear walls which also can be considered as structural elements (Figure 5). Moreover, buildings of 4 to 6 floors and one of 12 floors (fortunately on the corner) are situated along two perpendicular streets close to the Tribunals and on the edge of the parking limits (Figures 11 and 12).

Figure 2 *The use of supports in the construction sequence of an excavation*



Soil mechanics data

The soil encountered in the project area corresponds mainly to silty sands SM with no plastic fines. The geotechnical properties assumed in the project are shown in Table 1, where *h* represent the depth range for each layer, γ and γ' are the humid and submerged unit weight, G_s is the solid particle specific gravity, ϕ'_{cr} and ϕ'_{max} are the constant volume (or critical state) and maximum effective angles of internal friction, DR is the relative density, *c* is the cohesion and $(N_1)_{60}$ is standard penetration test blows number. The effective soil-wall interface angle of friction δ', was assumed as $\delta'/\phi' = 2/3$ for the active and passive side. The coefficient of permeability was estimated in the order of 10^{-5} m/s using the Hazen expression. The data shown in Table 1 was the input for the analyses presented later on.

Figure 3 *Retaining structure without lateral support: (a) initial excavation, (b) forces and distribution of active and passive lateral pressures and (c) bending moments diagram (EAB 2008)*

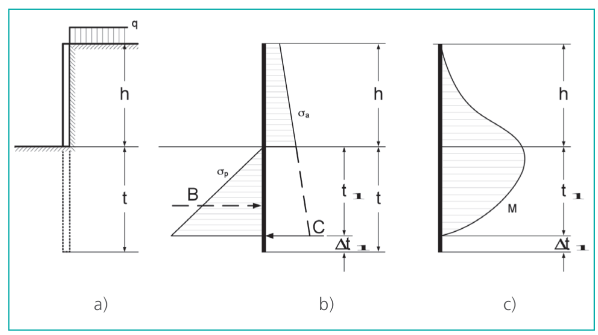


Figure 4 *Retaining structure with double lateral support: (a) final excavation, (b) forces and active and passive lateral pressures and (c) bending moments diagram (EAB 2008)*

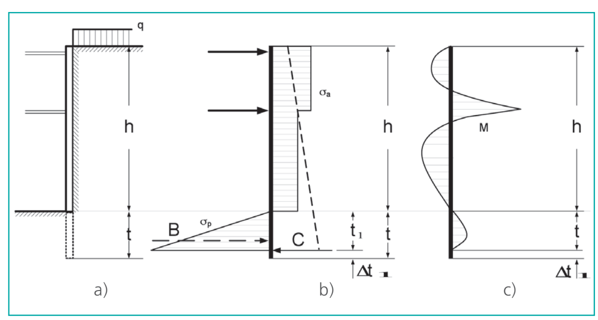


Figure 5 *Curved anchored soldier pile wall supporting the Tribunals (December 2008)*

Table 1 *Values of the soil parameters*

Soil	*h* m	γ kN/m³	γ' kN/m³	G_s	ϕ'_{cr}	ϕ'_{max}	DR %	ckPa	$(N_1)_{60}$
Fill	0-2	17.5	7.5	2.6	30	30	45	0	15
SM	2-7	17.5	7.5	2.8	33	34	60	0	18
SM	7-16	20.7	10.7	2.8	34	37	82	0	36

In practice the parameters related to the soil shear resistance are based on laboratory tests of samples taken from boreholes, but in Chile is quite often to estimate soil geomechanical properties from results of Standard Penetration Test (SPT). The SPT is not actually a standard test since the energy applied during the test varies depending on the equipment. An automatic SPT loses less energy and gives more consistent results than a manual operated SPT (Reading *et al.*, 2010). Although in Chile manual equipments are mostly used, there are also other effects related to the intrinsic methodology of the test. The repetitive impacts or blows imposed to the soil until a sampler drops 450 mm obviously perturb and change the initial soil properties. Moreover, the angle of friction ϕ' is estimated from correlations involving the number of blows $(N_1)_{60}$, which are averaged results with large scatter generally determined for different soil conditions. Furthermore, the design of retaining structures requires the geotechnical properties of shallow deposits. However, soil mechanics studies focus mainly on the design of building foundations, hence concentrating on deeper soils, which are below the excavation or the retaining structure. To improve the quality and reliability of the input parameters in excavation support analyses it is necessary to include from the beginning of the project appropriate laboratory and field studies.

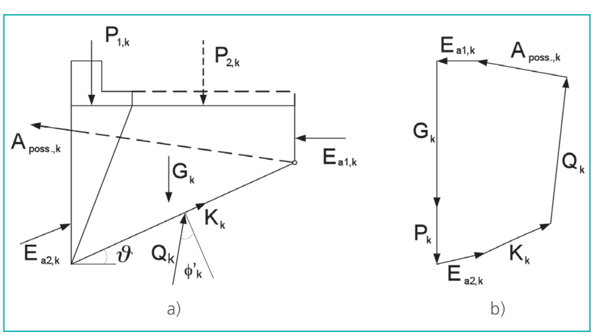
On the one hand, it is not yet clear whether the savings made when not complete or inappropriate soil mechanics studies are performed results finally in over designed retaining structures. This leads to spending more resources than the money supposedly was initially saved. On the other hand, under designed retaining structures can lead to the risk of failures.

Design procedure

The method of blocks proposed by Kranz (1953), allows the calculation of retaining structures with anchors. The block method is used for the determination of the anchor length which results in the stability of the wall, soil and anchor system. The block method originally derived by Kranz (1953) for walls with only one anchor, was later extended by Ranke and Ostermayer (1968) for the case of more than one anchor. Figure 6a shows that this method considers the static equilibrium of a trapezoidal soil prism in the form of forces in a free body diagram, which results in a polygon of force vectors as shown in Figure 6b. The block or trapezoid resistance against sliding, which is not possible to cover with the soil shear strength, is supplied by the anchor forces.

In addition to the 10 kPa general street overburden at the surface, it was considered for edifications an overburden of 12 kPa per floor. For the whole Tribunals an overburden of 100 kPa was considered at the foundation level, *i.e.* at a depth of 3 m (Lancuyen, 2008).

Figure 6 *Force determination for the anchor A: (a) forces acting on the soil block sliding and (b) polygon of forces (EAB 2008)*



The seismic forces were estimated by the expressions proposed by Okabe (1926) and Mononobe and Matsuo (1929). The values of horizontal seismic acceleration adopted are shown in Table 2. In the global stability designs it was verified that in the static case the factor of safety FS ≥ 1.5 and in the seismic case FS ≥ 1.1.

It is worth mentioning that vertical accelerations are not considered, when they could become as important as the horizontal ones (Villalobos, 2009). Evidence of this was again observed in acceleration records of the 27F 2010 earthquake. Moreover, the values of a_h are higher than the normally adopted, this responds to the importance of the buildings involved and their crowded location as well as the longer exposure time of the buildings (6 months compared with 1 month in a smaller project). The seismic accelerations were incorporated in the design of each construction sequence, *i.e.* during excavation and anchor distressing. Fortunately, the 27F 2010 earthquake occurred when the car park was finished. Otherwise, it is clear that the values shown in Table 2 are below at least three times the acceleration values recorded in the city of Concepción. After this enormous seismic event the engineering and construction community should rethink whether this excavation support technique under this design procedures are plausible for excavations surrounded by large buildings in the middle of the centre of a big city.

It is believed that the soldier pile walls resting buried between the soil and the car park walls may have reduced the accelerations and hence the displacements of the underground reinforced concrete structure. This hypothesis requires further research. There was no evidence of damage inside the car park and not serious damage in any of the buildings around the car park.

Design of anchors

Two rows of anchors were considered instead of struts. The design of grouted postensioned anchors follows the results obtained in the stability analyses undertaken for

Table 2 *Horizontal accelerations used in the soldier pile wall design*

Structure	a_h/g
Tribunals	0.18
General edification	0.15
Street	0.12

the project as part of the GGU-RETAIN (2008) computing program outputs. From the GGU-RETAIN outputs, anchor loads and the necessary anchor free length to guarantee the SPW stability were obtained, as well as the length of grouting and the number of cables in the anchor.

The anchor free length was determined following to the stability analysis results. The free length has to comply with the following requirements:

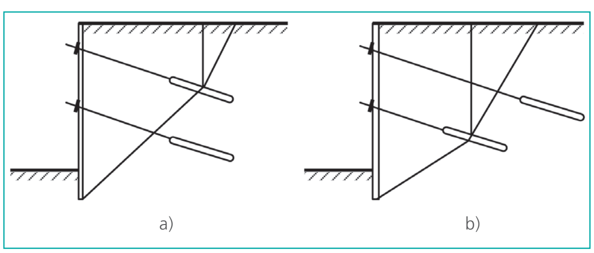
- Allowing the length of grouting outside the assumed slip plane (Figure 7).
- In the presence of rock, it should be avoided to have one part of the grouting length in the soil and the other in the rock.
- The minimum length considered from the bearing plate is 4.5 m for cable anchors.

The grouting length calculation is based on limit equilibrium methods (EAB 2008). These methods require parameters which are obtained from construction companies specialised in grouting injections. These parameters are defined from the perforation method and type of injection, which are not easy to evaluate theoretically and are determined from the drilling company records. The values empirically determined are associated to different type of soils and predefined safety factors.

The method used in this project to determine the grouting length was proposed by Bustamante (1986). This method, regularly used in Chile, consists of correlating the number of blows N in the SPT test with the friction capacity of the analysed soil. The length of grouting depends on the perforation diameter, the type of grouting and the grouting injection method. Assuming that the above variables are defined by the specialised company, the following expression can be used to estimate the limit tension of the anchor T_u,

$$T_u = \pi\, D_s\, L_s\, q_s \qquad (1)$$

Figure 7 *Anchors outside and inside of the slip plane (EAB 2008)*



where D_s is the mean diameter of the grouting length section, L_s is the grouting length and q_s is limit unit lateral friction acting along the grouted surface. To determine the allowable loads a factor of safety equal to 1.8 was used. From characteristic SPT values q_s values are estimated for the injected grouting (~300 kPa). The mean diameter D_s can be determined multiplying the perforation diameter D_d (0.15 m) by the injection coefficient α, *i.e.* $D_s = \alpha D_d$. The coefficient α depends on the type of injection, being IGU an Injection Global and Unique and IRS an Injection Repetitive and Selective. A value of α = 1.2 was used for an injection IGU.

The anchor allowable load T_a was determined using the following expression,

$$T_a = n\, A_c\, f_y / FS \qquad (2)$$

where n is the number of cables, A_c is the area of each cable, f_y is the cable yield stress and the factor of safety FS = 1.5. Table 3 resumes the cable technical characteristics for the post-stressed anchors installed in the project.

The resulting anchor allowable load as a function of the number of cables is shown in Table 4. Table 4 and the values of T_o in Table 8 were used to determined the necessary number of cables for each anchor.

To verify the design loads taken by the anchors, loading tests were carried out in the first and in the second row. The anchors had three steel cables as shown in Figure 8, and the properties shown in Table 3. The maximum capacity was defined as the 90% of the steel yielding load, resulting then in 635 kN. Figure 9 shows the results of a test in the second row for an anchor with a grouting length of 2.5 m. Initially increments were applied until half of the maximum capacity (first loading stage). A linear response is clearly observed and during unloading there is an important recovery of the displacements.

Table 3 *Anchor cable properties (ASTM 416, GRADE 270)*

Parameter	Value
Cable diameter D, mm	15.2
Cable area A_c, mm^2	140
Yield stress f_y, MPa	1670
Characteristic ultimate load T, kN	250
Characteristic yield load T_y, kN	235

Table 4 *Allowable load versus the number of cables*

Nº of cables	Allowable load, kN
2	313
3	470
4	627
5	783
6	940

A second loading stage or reloading is then applied until the previous maximum load of around 325 kN is reached. The response is again linear although slightly stiffer. However, passing the 325 kN load this trend changes smoothly towards a less stiff response and the loading is halted when the stiffness suffers a clear reduction for a deformation of 54 mm. A clear failure condition was not possible to measure since a cable failure would have occurred before mobilising the strength of the grouting length. Assuming the value of 635 kN as the anchor maximum capacity, corresponds to a dense sand according to the curves of Ostermayer (1974).

The phenomenon of creep was not observed in any of the loading steps tested for displacements up to 54 mm and time up to 15 minutes (Figure 10).

It is customary the use of metallic channels to transfer loads directly from the anchor to the H section soldier piles. These pieces, known as walings, form a beam made from a pair of back to back C sections with spacing for the anchor cables. This beam is turned perpendicular to the inclination angle of the anchor as can be seen in Figure 8.

Stability Analysis Following the Construction Sequence

Figures 11, 12 and 13 depict the excavation depth, the soil layers, the water table level, the foundation of the neighbour Tribunals building and the resulting distributions of lateral pressure, moment, shear and axial load and deformation. To diminish initial top deformations of the SPW a slope of 1 m high and 45º inclination was considered. Two horizontal dashed and dotted lines in front of the SPW represent the positions of the reinforced concrete slabs of the final car park. It is worth pointing out that Figures 11, 12 and 13 should be observed as a construction sequence, where Figure 11 represents 3 m excavation without anchors, Figure 12 includes the first row of anchors at 3 m for a 6 m excavation and Figure 13 the final two rows of anchors at 5.5 m for a 8 m excavation.

Figure 8 *Loading test set up in the second row of anchors (December 2008)*

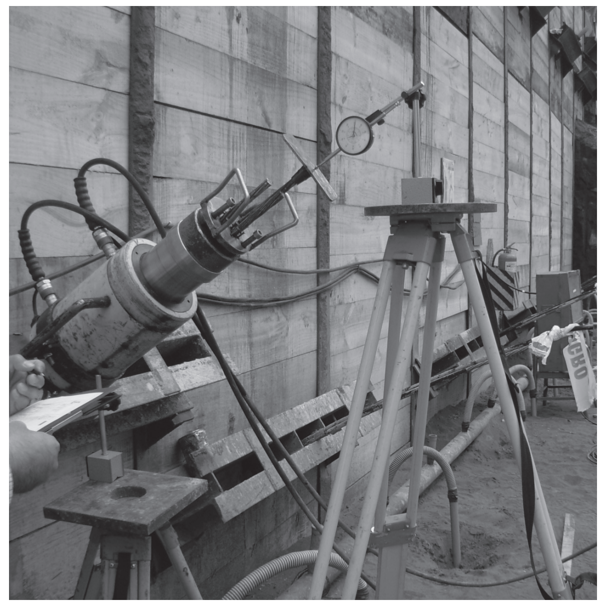


Figure 9 *Load displacement curve determined in an anchor loading test*

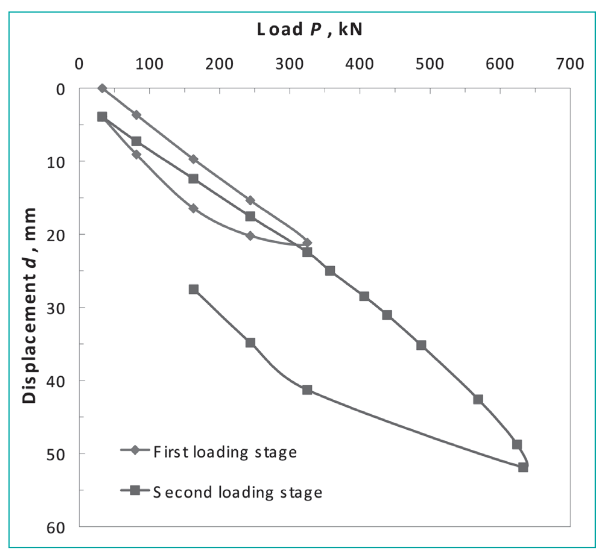


Figure 10 *Results from creep tests for second loading stage*

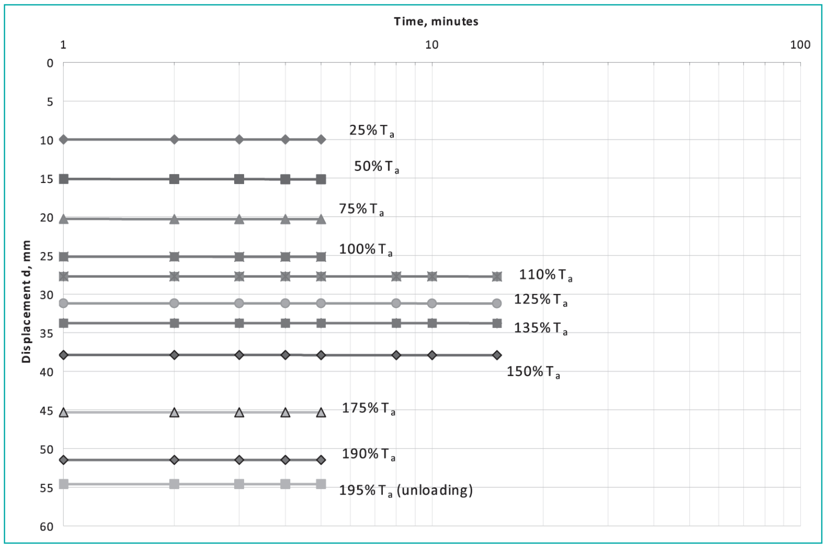


Figure 11 *Excavation stability analysis without anchors next to the Tribunals*

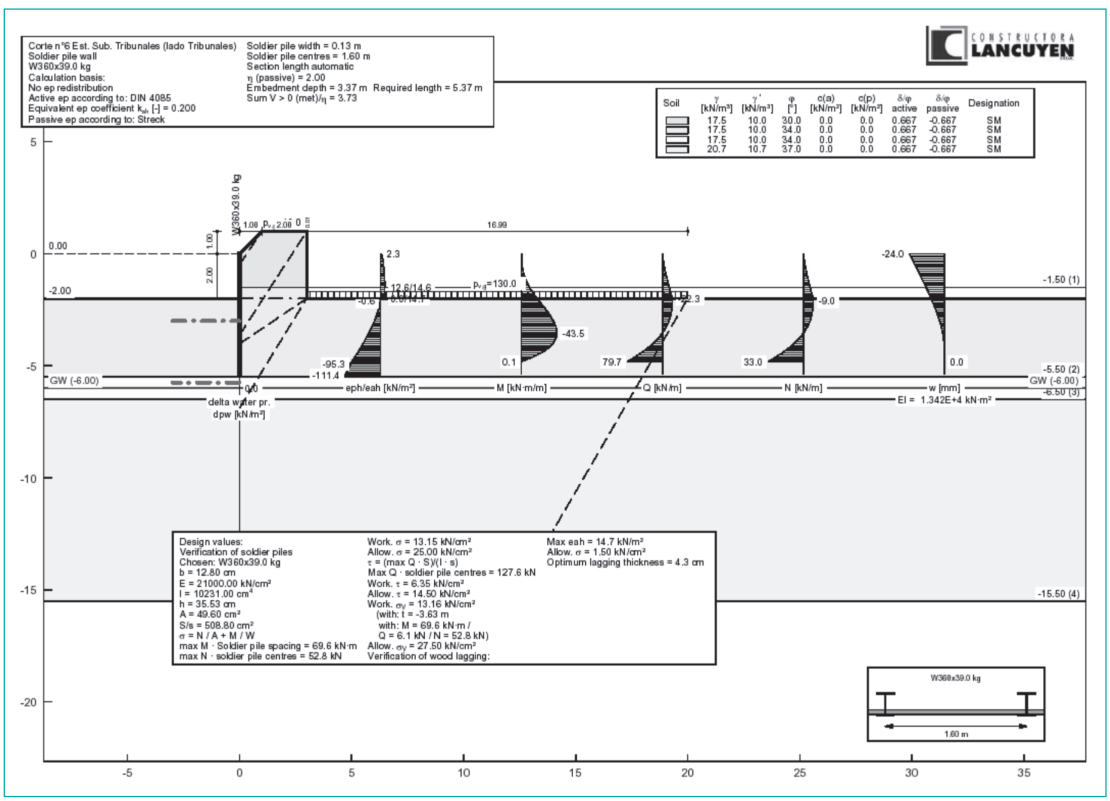


Figure 12 *Excavation stability analysis with the first row of anchors next to the Tribunals*

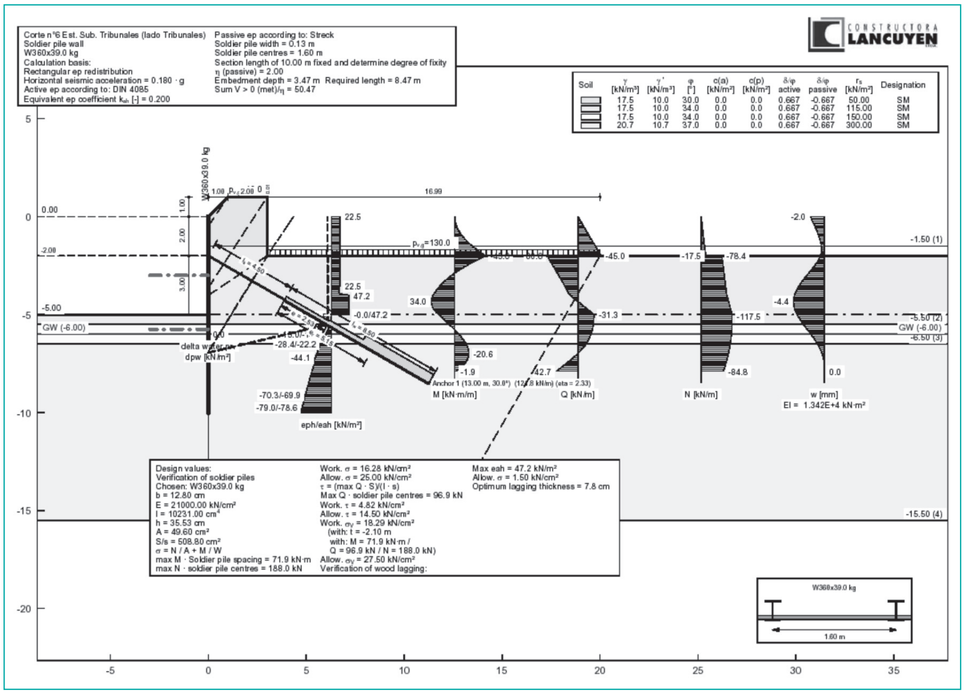
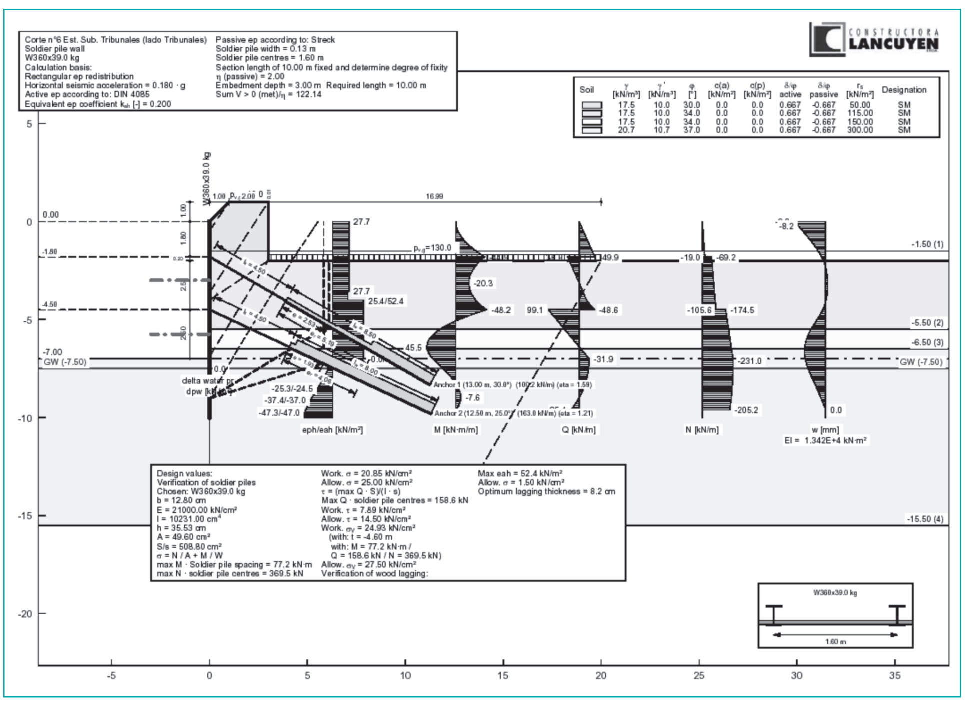


Figure 13 *Excavation stability analysis with second row of anchors next to the Tribunals*



The outputs shown in Figures 11, 12 and 13 have been obtained using the computational program GGU-RETAIN (2008). The use of this type of program eases enormously calculations and hence analyses, otherwise it would be very complicated to deal with so many variables and different stages of construction.

The soldier pile adopted in the design in front of the Tribunals was a W360x39 kg steel profile, with the following characteristics: b = 12.8 cm, E = 21 MN/cm^2, I = 10231 cm^4, h = 35.53 cm and A = 49.6 cm^2, where E is the steel young modulus, I is the inertial moment and b, h and A are the section width, height and area, respectively. For each different loading condition, *i.e.* in front of each building and street, a similar analysis was performed.

It can be noted that in the results shown in Figures 11 and 12 the water table level is initially at -6 m on both sides of the SPW and in Figure 13, the water table level drops to -7.5 in the excavation due to dewatering on both sides. This water table lowering does not consider the possible effects of hydrodynamics pressures behind the SPW caused by transient flow. It is recommended to study further this effect since it is not clear whether this simplification may have consequences or not on the stability of the SPW tip.

Table 5 resumes the project anchor design. Each row in the table corresponds to a zone with these two anchors, T_o is the anchor resistance obtained from GGU-RETAIN program multiplied by the horizontal distance between anchors (3.2 m) resulting in the allowable load of the anchor, L is the total anchor length, β is the anchor angle of inclination respect to the horizontal axis and D_f is the building foundation depth next to the anchored SPW. The free length adopted for all the anchors was 4.5 m. Figure 14 shows the plan view of the SPW and the location of the anchors.

There is a higher density of anchors under certain zones of the Tribunals and under other buildings. In some areas under the Tribunals there are anchors passing under other anchors. Construction of these types of anchors has not only avoided touching the Tribunals foundations, but also has not touched other anchors. Figure 15 shows the SPW with two rows of anchors with the inclined walings. Also, it can be seen the well point system at the toe of the SPW.

Table 5 *Anchor design using program GGU-RETAIN (Lancuyen 2008)*

T_o kN	L m	L_s m	β °	buildings	D_f m
350 280	12.5 8.5	8 4	30 25	Fiscalía, Tucapel St	0
410 300	12 8.5	7.5 4	40 30	Hites	5
370 480	12.5 11	8 6.5	30 25	Entrances INP	1.5
450 325	11.5 9	7 4.5	45 35	INP	5
350 330	12.5 9	8 4.5	30 25	Tribunals	3
330 520	13 12.5	8.5 8	30 25	Tribunals	3
400 300	12.5 8.5	8 4	35 25	Tribunals	5.5
370 480	12.5 11	8 8.5	30 25	Barros Arana St	1.5

Discussions and Conclusions

Since no important disturbance was observed in terms of cracks or damage of neighbouring structures, it is concluded that the anchored soldier pile wall offered an adequate solution for the support excavation required for the construction of a large underground car park.

However, it is believed that during the car park construction the anchored soldier pile wall may have not been able to resist adequately the inertial forces imposed by the earthquake on 27th February 2010. As a consequence, large displacements of the soldier pile wall may have occurred owing to strong seismic lateral and vertical earth pressures, inducing serious damage to the buildings been supported. It is suggested that a better solution is to build a diaphragm wall instead of a soldier pile wall. Diaphragm walls are much stiffer, becoming the final walls of the structure, and therefore can offer a better response under strong seismic loads.

The car park project contemplated 3596 m^2 of anchored SPW with 314 postensioned anchors totalling 3784 m under loads between 300 kN and 560 kN and 300 H section soldier piles totalling 3200 m. Once the definitive parking foundations, walls and slabs are built and can resist the lateral pressures, anchors are distressed and the SPW lies buried with the H section piles and the timber laggings, except the walings which can be recovered. The final reinforced concrete walls and slabs stay in contact with the H piles of the SPW, assuring

Figure 14 *Plan view of the parking project showing position of anchors (Lancuyen 2008)*

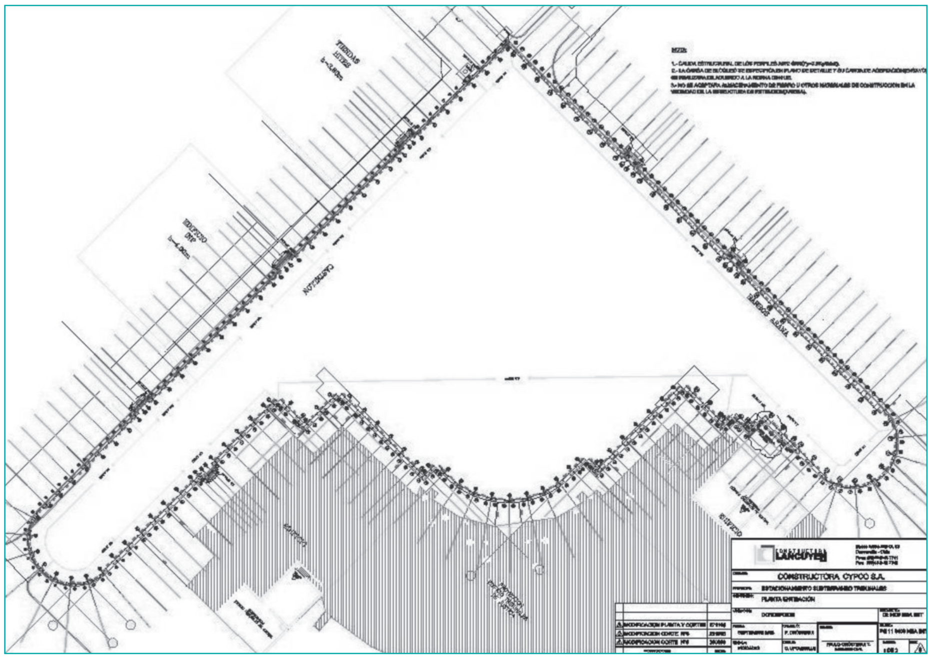


Figure 15 *View of the excavation for the underground parking, showing SPW and well points (December 2008).*

the transfer of loading from the retaining structure to the new and definitive structure. It is believed that the buried SPW may reduce the seismic loads, since no major fissures or crack were observed after an 8.8 moment magnitude earthquake. To verify this hypothesis further research contemplating acceleration and displacement monitoring is needed.

However, some questions may arise in terms of the integrity of the timber laggings and steel H piles with time. Above the water table it might be possible the decomposition of the wood and rusting of the steel, which could induce future soil displacements with associated settlements. Therefore, it is suggested the continuous study by monitoring any soil displacement that may occur behind the timbers and possible settlements of neighbouring buildings.

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