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Excavation support solutions for a large underground parking

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ABSTRACT: Geotechnical conditions of an underground parking project in the city centre of Concepción are described. The geotechnical study of the excavation support had to consider the avoidance of any damage of the Palace of Tribunals and surrounding buildings. The solution adopted was an anchored soldier pile wall driven into semi dense silty sand around the excavation. Two rows of anchors were designed, where the anchors installation and placement were studied in order to not disturb the different stages of construction. Tests were carried out to verify the anchor designs. It can be concluded that the excavation support solution adopted performed adequately, since no serious deformation in the Tribunals nor in the surrounding buildings nor in the services has occurred.

1. INTRODUCTION

Nowadays, it is practically impossible to avoid underground constructions in big cities due to lack of available land and due to the high prices. The construction of undergrounds not only is a challenge for the excavation stability, but also needs the avoidance of damage to neighbouring streets, monuments and buildings. There are different techniques to sustain an excavation depending on the type of soil and excavation height. The city of Concepción in Chile has had a considerable growth not only of flat and office buildings but also underground parking, shopping centres and transport infrastructure. To sustain excavations in these projects it has been widely used a technique known as Soldier Pile Wall (SPW). Anchored SPWs have the advantage of offering free movement within an excavation unlike the use of struts or other shoring methods.

A SPW is a continuous and temporal support, whose design considers the soil conditions and excavation geometry, especially depth and width. The technique consists in driving soldiers (steel H sections) into the soil before digging, with distances between them to be calculated. The range of distances is between 1.2 m and 3 m, 1.6 m being the most common in Concepción. Once the excavation starts, from the line formed by the soldier piles, 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 movements of the soil, particularly close to the surface. This is due to the high flexibility of this type of support system, even with relatively rigid H sections, they become slender because of their length. To solve this problem, which is not related to stability nor to the capacity to hold the excavation, anchors are incorporated in SPWs to reduce soil deformations potentially able to affect neighbouring structures.

Although spaces between timber laggings are very small, they are not tight enough to stop ground water to flow through them. If gaps between timbers do not let pass completely the water flow through the SPW, it is easy to install drains in the SPW. The idea is to avoid the build up of pore water pressure behind the SPW, which could add more lateral pressure and undesirable deformations. It 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.

The appropriate design of retaining structures depends significantly on the knowledge of the geotechnical properties of the soil. Therefore, it is paramount to carry out geotechnical studies as complete as possible, which can provide reliable values of the geotechnical properties of the soil to be dug, the soil below it and the soil to be sustained.

This article describes and analyses the current design practice of anchored SPWs, where relevant structural and geotechnical issues are considered. This analysis is later on applied to the complex project of underground parking under the Tribunals. This work arises as a way to contribute to the scarce number of available technical publications about temporal retaining structures in Concepción.

2. LOADING ON SPWs

A SPW is a flexible retaining structure even if the soil being retained is very dense or overconsolidated and with high stiffness. Therefore, the lateral earth pressure on a SPW has no chance to be at rest, not even under initial conditions, since soil deformations will 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 lateral displacement u_{hmax} on top of a 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 walls, the estimation of any lateral earth pressure will depend mainly on the anchor pre-stressed loads.

The active pressure applies from the top to the bottom of the excavation behind the wall and downwards the passive pressure applies in front of the wall from the bottom of the excavation to the end tip of the H section piles. 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 in heterogeneous and anisotropic soils and in flexible walls. As a consequence of the above, 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.

Dead and live loads, the latter can be constant or variable, are added in addition to the soil lateral pressure. The EAB (2008) recommendations consider a uniform distributed load over the surface of 10 kN/m^2, trying to represent the effect of pavements and streets plus their live loads. It is important to point out that the SPW calculation procedures are strongly linked to the construction sequence. For example, according to the calculation results a SPW without anchors will resist only a few metres, to keep digging the installation of a row of anchors at the bottom of the initial excavation will be necessary. Once these anchors are under tension, it is possible to continue with the excavation of the next 3 or 5 m for example and then performing the installation of a second row of anchors if the excavation continues another 3 or 5 m and so on. EAB (2008) suggests that if the height from the bottom of the future excavation to the anchors line is h, then the anchors 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).

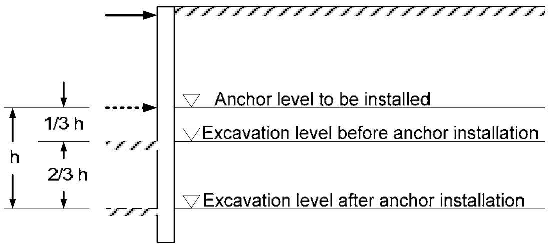

Figure 1: Excavation limit before installing anchors (EAB 2008)

It is suggested to avoid the presence of water pressures behind the walls, but if there is no way to eliminate it during heavy rain seasons or when the pumps of the well points system do not work, water pore pressures should be included as hydrostatic and hydrodynamic pressures in case of water flow. Pore water pressure for the latter case can be determined by means of flow net analysis.

2.1. Equilibrium of forces

In the force equilibrium analysis the soil and water lateral pressure, as well as dead loads of surrounding buildings, live loads of streets and possible earthquakes are included. The excavation support design using a SPW considers all the forces involved as part of horizontal forces equilibrium within the height of the excavation. The resistance offered by the soil and wall interaction has to be higher than the lateral pressures. Another analysis to be carried out corresponds to the determination of the embedding depth of the H section soldier piles, where in addition to the horizontal forces equilibrium, moment equilibrium is included (see Figures 2 and 3).

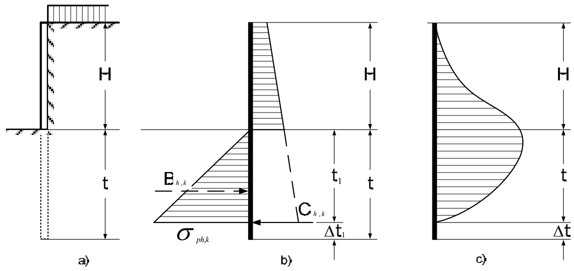

Figure 2: SPW without lateral support: (a) initial excavation, (b) lateral pressures and forces diagram and (c) bending moments diagram (EAB 2008)

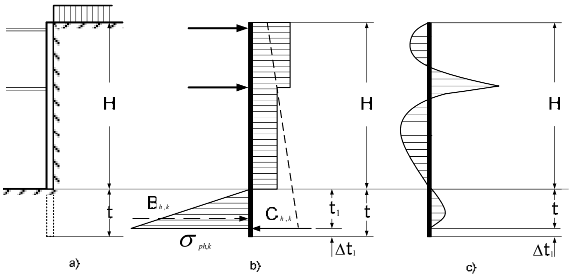

Figure 3: SPW with double lateral support: (a) final excavation, (b) lateral pressures and forces diagram and (c) bending moments diagram (EAB 2008)

3. THE TRIBUNAL RETAINING PROJECT

The project of underground parking next to the Justice Tribunals of Concepción was a great challenge to Geotechnical Engineering 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 (Figure 4). 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 (Figure 11).

3.1 *Soil Mechanics*

The soil encountered in the project area corresponds mainly to silty sands with no plastic fines whose geotechnical properties are shown in Table 1. The soil-wall interface angle of friction δ, was assumed as δ/ϕ = 2/3 for the active and passive side. The coefficient of permeability was estimated in the order of 10^{-5} m/s. The data shown in Table 1 was used as an input for the analyses presented later on.


Figure 4: View of the curved anchored SPW adjacent to the Tribunals

Table 1: Values of the soil parameters

Soil	h m	γ kN/m³	γ' kN/m³	G_s	ϕ'_{cr}
Fill	0-2	17.5	7.5	2.6	30
SM	2-7	17.5	7.5	2.8	33
SM	7-16	20.7	10.7	2.8	34

Soil	DR, %	ϕ'_{max}, °	*c*, kPa	$(N_1)_{60}$
Fill	45	30	0	15
SM	60	34	0	18
SM	82	37	0	36

Averaged values estimated from Soil Mechanics data

One not minor problem Geotechnical Engineers have to deal with is the quality and reliability of the parameter values obtained in situ and in the laboratory. The parameters related to the soil shear resistance are mostly based on SPT tests, which disregarding equipment and operator shortcomings, are affected by the intrinsic methodology of the test. The repetitive impacts or blows imposed to the soil until a sampler drops a standardized distance 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 have been generally determined for different soils and conditions. Furthermore, the design of retaining structures requires the geotechnical properties of shallow deposits. However, the Soil Mechanics studies focus mainly on the design of building foundations, hence concentrating on deeper soils, which are below the excavation or 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.

It is not yet clear whether the savings made when appropriate Soil Mechanics studies are not performed results finally in over designed retaining structures, 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.

3.2 *Design methodology*

The method of Kranz (1953) or also known as the method of blocks, allows the calculation of retaining structures with anchors. With this method it is possible the determination of the anchor length and hence the stability of the wall, soil and anchor system. The Kranz method was originally derived for walls with only one anchor, however, Ranke and Ostermayer (1968) extended the method for more than one anchor. Figure 5 shows that this method analyses the equilibrium of a trapezoidal soil prism in the form of forces in a free body diagram, which results in a polygon of force vectors. The block or trapezoid resistance against sliding, which is not possible to cover with the soil shear strength, is supplied by the anchor forces.

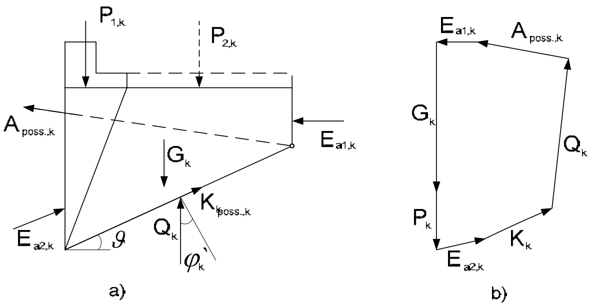

Figure 5: Force determination for the anchor A: (a) forces acting on the soil block sliding and (b) polygon of forces (EAB 2008)

In addition to the 10 kN/m^2 general street overburden at the surface, it was considered for edifications an overburden of 12 kN/m^2 per floor. For the whole Tribunals an overburden of 100 kN/m^2 was considered at the foundation level, *i.e.* at a depth of 3 m (Lancuyen, 2008).

The seismic forces were estimated by the expressions recommended by Okabe (1926) and Mononobe and Matsuo (1929). The values of horizontal seismic acceleration adopted are shown in Table 2.

Table 2: Horizontal accelerations used in the anchor design

Structure	a_h/g
Tribunals	0.18
General edification	0.15
Street	0.12

It is worth noting that vertical accelerations are not considered, when they could become as important as the horizontal ones (Villalobos, 2009). 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.

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.

3.3 Design of grouted postensioned anchors

The design of anchors was performed considering the results obtained in the stability analyses undertaken for the project as part of the GGU-RETAIN (2008) computing program outputs. From these results, 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 according to the stability analysis results. The free length has to respond the following requirements:

- Allowing the length of grouting outside the failure zone (Figure 6).
- 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 construction parameters defined from the perforation method and type of injection, which are not easy to evaluate theoretically and are determined from the drilling company experience. The values empirically determined are associated to different type of soils and predefined safety factors.

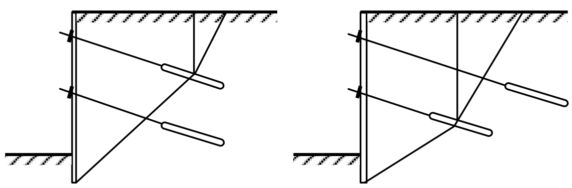

Figure 6: Consideration of anchors outside and inside of the failure zone (EAB 2008)

The method used in this project to determine the grouting length was proposed by Bustamante (1986). This method, very popular 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 following parameters:

- Perforation diameter
- Type of grouting
- 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,

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

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 were estimated where the grouting will be injected (~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 $\alpha = 1.2$ was used for an injection IGU.

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

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

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 used in the project.

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

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 5 were used to determined the necessary number of cables for each anchor.

Table 4: Allowable load versus the number of cables

N° of cables	Allowable load, kN
2	313
3	470
4	627
5	783
6	940

Anchor loading tests were carried out in the first and in the second row. The anchors had three steel cables, 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 7 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. 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 55 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).

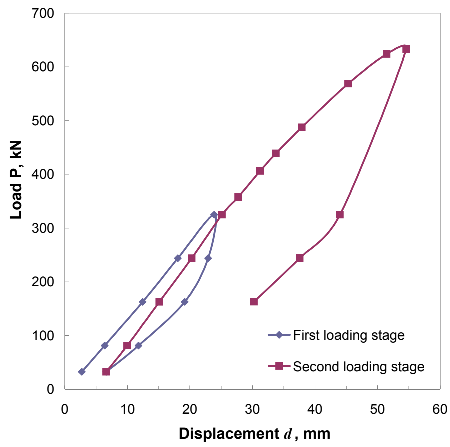

Figure 7: Anchor capacity measured in a loading test

The phenomenon of creep was not observed in any of the loading steps tested for displacements up to 55 mm and time up to 15 minutes.

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 (see Figures 6 and 12).

4. STABILITY ANALYSIS

Figures 8, 9 and 10 depict the excavation geometry, the soil deposits, the level of the water table, the foundation of the neighbour building and the resulting distributions of lateral pressure, moment, shear and axial load and the deformation. It is worth pointing out that Figures 8, 9 and 10 should be observed as a construction sequence, where Figure 8 represents 2 m excavation without anchors, Figure 9 includes the first row of anchors at 2 m for

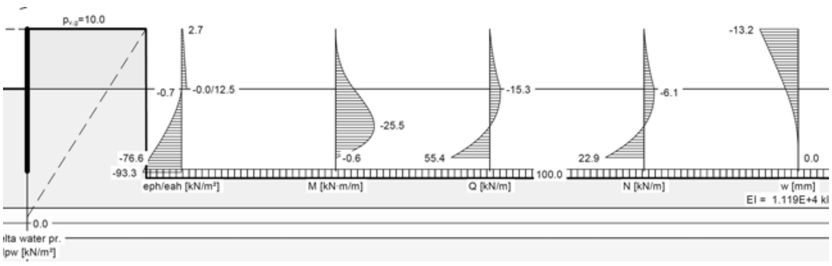

Figure 8: Example of excavation stability analysis without anchors next to Hites building

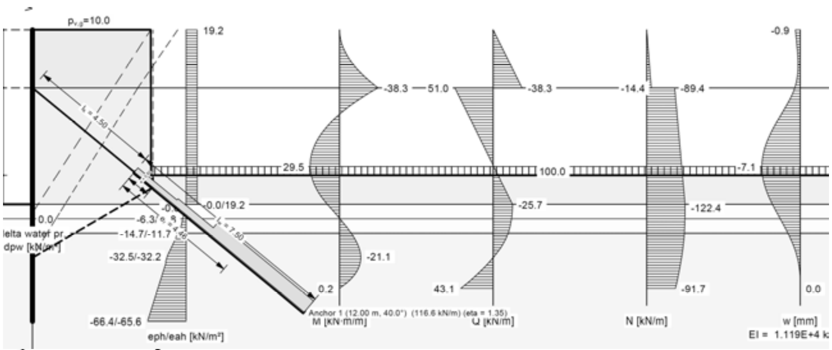

Figure 9: Example of excavation stability analysis with the first row of anchors next to Hites building

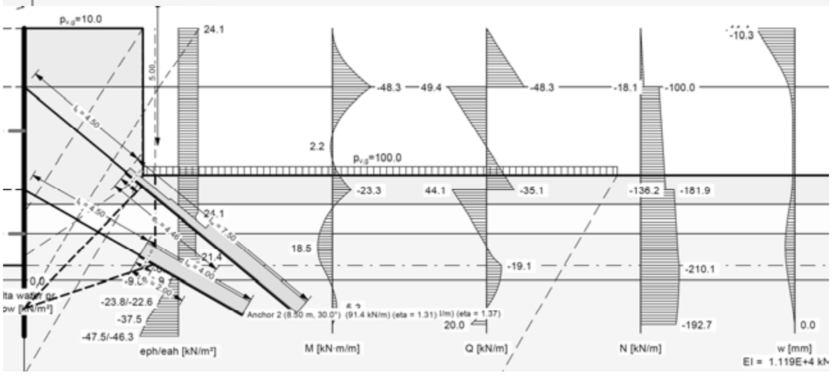

Figure 10: Example of excavation stability analysis with second row of anchors next to Hites building

a 6 m excavation and Figure 10 the final two rows of anchors at 5.5 m for a 8.1 m excavation.

The outputs shown in Figures 8, 9 and 10 have been obtained using the computational program GGU-RETAIN (2008). The use of this type of program eases enormously calculations, otherwise it would be very complicated to deal with so many variables and different stages of construction.

Table 5: Calculation basis for the SPW at Hites

Calculation	Fig. 8	Fig. 9	Fig. 10
Distribution	-	rectangle	rectangle
Active ep	DIN4085	DIN4085	DIN4085
ep k_{ah}	0.2	0.2	0.2
Passive ep	Streck	Streck	Streck
a_h, g	-	0.15	0.15
Excavation depth, m	2	6	8.1
Embedment depth, m	2.76	3.48	2.3
Required length, m	4.76	9.48	10.4

In Figures 8, 9 and 10 there are boxes with blurry information. The one on top right resume the soil deposit properties which are shown in Table 1. The box on the right at the bottom shows the plan view of the SPW with the distance between H sections centres of 1.6 m. The other two boxes on the left are reproduced in Tables 5, 6 and 7.

Table 6: Verification of soldier piles at Hites

Soldier pile	Fig.8	Fig. 9	Fig. 10
M_{max}, kNm	40.9	61.3	77.3
N_{max}, kNm	36.7	195.9	336.2
σ_{work}, MPa	81.7	151.2	208.5
σ_{allow}, MPa	250	250	250
Q_{max}, kN	88.7	91.6	79
τ_{work}, MPa	54.8	50.5	48.9
τ_{allow}, MPa	145	145	145
$\sigma_{v\ work}$, MPa	98.1	174.6	225
t, m	4.3	2.1	-2.1
M, kNm	9.3	61.3	77.3
Q, kN	88.7	81.6	79
N, kN	36.7	195.9	336.2
$\sigma_{v\ allow}$, MPa	275	275	275

The soldier pile adopted in the design was a W310x38.7 kg, with the following characteristics: b = 16.5 cm, E = 21 MN/cm^2, I = 8527 cm^4, h = 31 cm, A = 49.4 cm² and S/s = 527.4 cm². Working stress σ_{work} is determined by:

$$\sigma = \frac{N}{A} + \frac{Nw + M}{W} \qquad (3)$$

where N and M are the maximum axial load and moment, w is the maximum displacement, A is the cross sectional area and W is the section modulus.

Table 7: Verification of timber laggings

Timber	Fig.8	Fig.9	Fig. 10
Max eah, kPa	12.5	19.2	40.8
σ_{allow}, MPa	15	15	15
Thickness t , cm	4	5	7.2

It can be noted that in the results shown in Figures 8 and 9 the water table level is initially at -6.5 m on both sides of the SPW and in Figure 10, the water table level drops to -8.6 in the excavation due to dewatering. This water table lowering does not consider the possible effects of hydrodynamics pressures behind the SPW. 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 8: Anchor design from GGU (Lancuyen 2008)

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

Table 8 resumes the anchor design. Each row corresponds to a zone with these 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 11 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. The installation of these types of anchors has not only avoided touching the Tribunals foundations, but also has not touched other anchors.

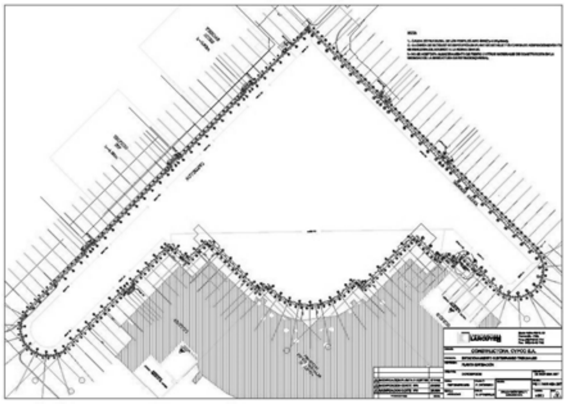

Figure 11: Plan view of the parking project showing position of anchors (Lancuyen 2008)

Figure 12 shows the SPW with two rows of anchors with the inclined walings. Also, it can be seen the well points at the toe of the SPW.

5. FINAL REMARKS

The parking 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 the transfer of loading from the retaining structure to the new and definitive structure.

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.

Figure 12: View of the excavation for the underground parking, showing SPW and well points

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