EDIFICIO PIEDRA REAL – CONCEPCION, CHILE
CASE STUDY OF AN UPLIFT REINFORCEMENT PROJECT

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ABSTRACT

The rapid urban development that Chile has experienced since the early 2000s, has motivated real estate developers to conceive higher buildings and deeper excavation pits, in order to use the available spaces more efficiently.

Chile is among the countries with the highest seismicity in the world, therefore both the structural and the geotechnical design are confronted with complex and challenging conditions, and have to fulfill the safety requirements stablished by the national construction regulations, seeking the best technical and economical solutions.

The implementation of worldwide well known technologies and construction methods, such as micropiling, has enabled designers to come up with optimal solutions in order to satisfy the requirements of the urban development.

The following article presents the case study of the project "Piedra Real", located in Concepcion, the 2nd largest city of Chile. The border conditions of the project, as well as it´s specific geological and geotechnical parameters will be briefly described.

This paper will focus mainly on the geotechnical design of the permanent uplift reinforcement of the building, materialized by self-drilling grouted Ischebeck TITAN micropiles. The design considerations regarding the structural capacity and durability of the reinforcement system will be presented.

Furthermore, the direct interaction of the uplift reinforcement system with other relevant items of the project, such as the temporary shoring of the excavation pit and the temporary ground water lowering, will be discussed.

Finally, an overview of other potential application fields for the case study solution, such as road and railway infrastructure, will be presented.

KEYWORDS

Micropiling, geotechnical design, seismicity, uplift reinforcement, excavation shoring, urban development, infrastructure

1. INTRODUCTION

The project *Piedra Real (Las Heras)* is located in the downtown area of Concepcion (Bio-Bio Region), the second largest city in Chile (Figure 1).

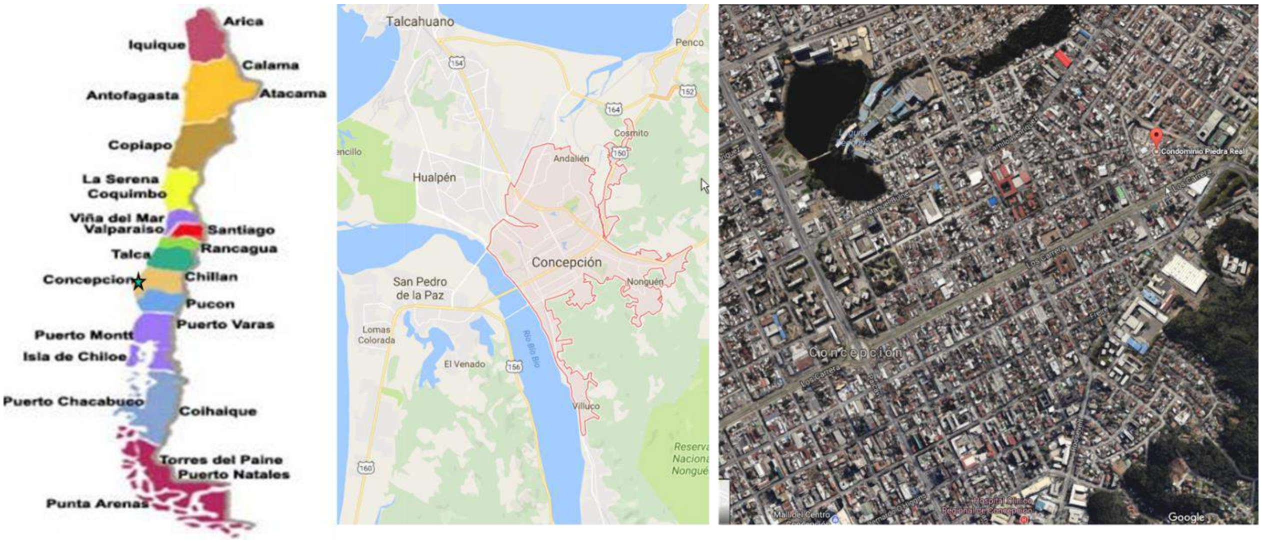


Figure 1 – Location of the Condo Piedra Real (Concepcion, Chile)

The condo consists in four 18-storey buildings, with two underground parking levels. The foundation of the complex was materialized by a reinforced concrete slab, with a surface of about 5680m^2. The projection of the towers occupies about 40% of the total building area (Figure 2).

Figure 2 – Overview Condo Piedra Real (Concepcion, Chile)

The architectural project required a free height of 7.0m for the underground parking levels (measured from the top of the foundation slab.

Due to the local groundwater conditions, the foundation slab is subjected to hydrostatic uplift, which – together with seismic actions – makes necessary a permanent reinforcement of the foundations to absorb the tension forces. The reinforcement was materialized by self-drilling grouted Ischebeck TITAN micropiles.

2. GEOLOGICAL DESCRIPTION

The city of Concepción is built on Tertiary sediments in a valley created by a graben, with metamorphic and granitic formations to the north, east, south, and beneath these Tertiary sediments. The sedimentary valley is depicted along with the 3 major faults that run through the area (Figure 3, [9]).

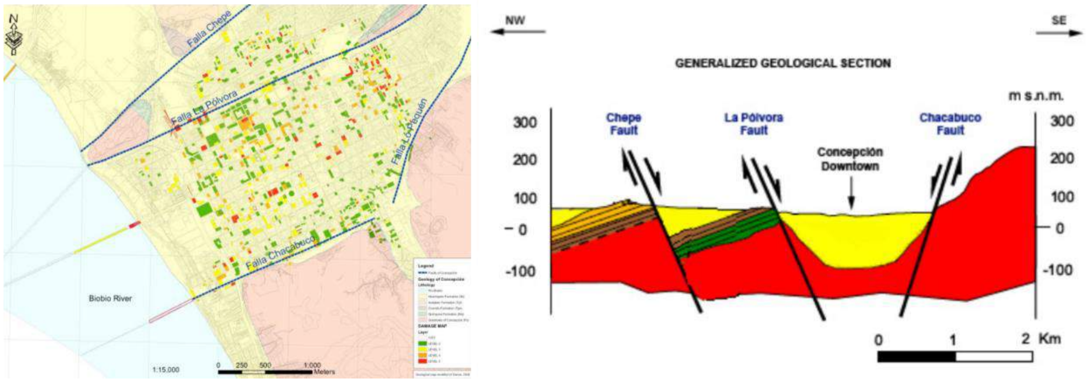


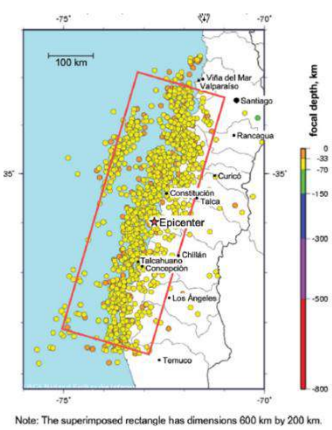
Figure 3 – Geology map and cross-section (NW-SE) of Concepcion [9]

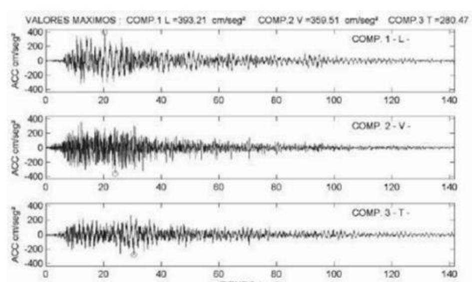
3. CHILEAN SEISMICITY – THE MAULE EARTHQUAKE

Chile is one of the countries with the highest seismic activity in the world. According to the USGS [19], 3 out of the 20 largest earthquakes, recorded worldwide since 1900, have occurred in Chile.

On Saturday 27 February 2010 (03:34 local time) an earthquake with a magnitude (M_w) 8.8 struck the central-south region of Chile, and had a deep impact in the public perception of the high vulnerability of the infrastructure to seismicity, since about 75% of the Chilean population was affected, and an estimated loss of about US$ 21 Billion was caused (approx. 70% of the total estimated economic damage) [4].

The earthquake-induced ground shaking had a total duration of about 140 s, with the strongest part lasting 40 -50 s [1]. In the region of strongest ground shaking, ground accelerations exceeded 0.05g for over 60 s in most of the records, and more than 120 s in the records corresponding to the Concepcion area [6].





Station	Maximum Horizontal Acceleration (g)	Maximum Vertical Acceleration (g)
Santiago Universidad de Chile	0.17	0.14
Santiago Elevated Train Station Mirador	0.24	0.13
Santiago CRS MAIPU	0.56	0.24
Santiago Hosp. Tisne	0.30	0.28
Santiago Hosp. Sotero de Río	0.27	0.13
Santiago Cerro Calán	0.23	0.11
Santiago Campus Antumapu	0.27	0.17
El Roble Hill	0.19	0.11
Viña del Mar (Marga Marga)	0.35	0.26
Viña del Mar (Downtown)	0.33	0.19
Curico Hospital	0.47	0.20
Concepción Colegio San Pedro	0.65	0.58
Valdivia Hospital	0.14	0.05

Figure 4 – Left: Main shocks and aftershocks of Mw ≥ 4 between 2/27/10 and 3/26/10 [1], Middle: Strong motion record of Downtown Concepcion [13], Right: Preliminary Processed Records Maximum Accelerations [6]

Despite the intensity of the earthquake, the majority of the engineered buildings behaved very well and the damages were mostly restricted to non-structural elements, mainly due to the use of appropriate design codes and construction´s practice regulations. However, according to official estimations, about 2.5% of the engineered buildings suffered severe damage, causing not only their structural failure (total or partial), but also human casualties [18].

Examples of severe damage were observed in Concepcion, mostly in its downtown area, where one building fully collapsed (Alto Rio Tower) and several others had to be demolished due to partial failure at localized floors (Figure 5).

Figure 5 – Alto Rio Tower (fully collapsed, left) and O´Higgins Tower (partially collapsed, right) [2]

Based on field observations of severe damage on mid-to-high rising buildings, it seems that vertical irregularities - due to set-backs in the upper floors or due to stiffness differences of vertical elements (reduced cross-sections of vertical shear walls at the parking levels) - might have triggered the failure of vertical elements, as a result of an induced concentration of stresses (compression and tension), compounded with flexural solicitations experienced during the earthquake [1]. The observed damage has been also associated with site and/or basin effects, ground failure (displacements caused by liquefaction) as well as with the soil classification, that led to define both the design spectra and the foundation type [9].

4. INFLUENCE ON THE DESIGN AND BUILDING REGULATIONS

Chile has experienced a high number of large magnitude earthquakes, and as a result, the relevant Chilean Design Codes: *NCh430.Of 2008* and *NCh433.Of96* have been subsequently updated, based on the lessons learned from past damaging events. The Maule Earthquake wasn´t an exception, and as a result of its occurrence, up to two amendments – in form of decree-laws - have been implemented to update and improve the building practice:

DS N° 60 (December 13, 2011): sets the new requirements for design of reinforced concrete structures, replacing the NCh430-Standard

DS N° 61 (December 13, 2011): sets the new requirements for seismic building design, modifying and/or complementing the NCh433-Standard, regarding the soil classification and the design spectra.

Also other national standards have been implemented, to set the requirements of special geotechnical works, such as excavations: *NCh3206.Of2010*

5. UPLIFT REINFORCEMENT – DESIGN AND IMPLEMENTATION

5.1 Geotechnical parameters of the subsoil

The geotechnical parameters for the construction site were defined by the geotechnical prospection as presented in Table 1. The groundwater table is located near to the surface, at depths between 1.5m and 3.5m.

Due to seasonal fluctuation, it was recommended to consider a design groundwater table at a depth of 0.5m below ground level [8].

Layer	Thickness (m)	γ / γ' (ton/m^3)	ϕ' (°)	c' (ton/m^2)	N_{SPT}
H1: sandy artificial fillings	1.9 - 3.0	-	-	-	-
H2: SM / SP-SM loosen to medium dense	2.7 – 4.5	1.7 / 9.5	28	0.0	10-30
H3: SP-SM / SM dense	Undefined (below 6.0m under ground level)	1.8 / 10.5	40	0.0	40-60

Table 1 – 20 Geotechnical Parameters [8]

The competent soil for the foundation of the structures corresponds to the layer H3, which was classified under category C according to the DS N°61.

5.2 Structural design

The structures were designed in accordance with the above mentioned Chilean Standards NCh433 and the decree-laws DS N°60 and DS N°61.

The towers were conceived with seismic dissipaters installed at the sides (Figure 6), in order to reduce the seismic effects both in the structural and non-structural elements [12].

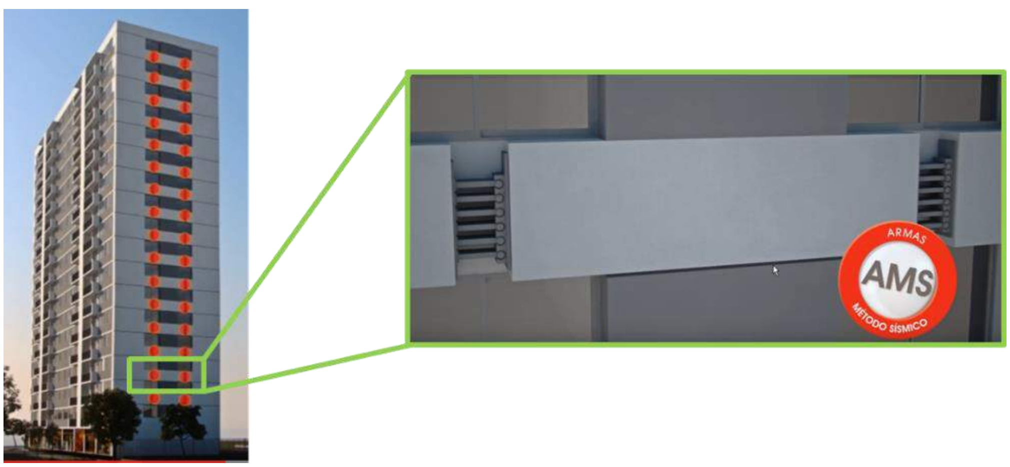


Figure 6 – Seismic dissipaters installed at the sides of the towers [12]

Focusing on the foundation design, the structural analysis resulted in the reinforcement requirement to absorb the following:

a) **Tension loads**, mainly caused by uplift in the areas outside the projection zones of the towers.
b) **Compression loads**, at certain locations under the towers, where the admissible load bearing capacity of the subsoil were exceeded. The reinforcement was materialized by 108 CFA-bored piles (Φ0.6m), however their design is outside of the scope of the present document and will not be further discussed.

The distribution of the required reinforcement is presented in Figure 7 (for half of the structure), and summarized Table 2 (for tension loads)

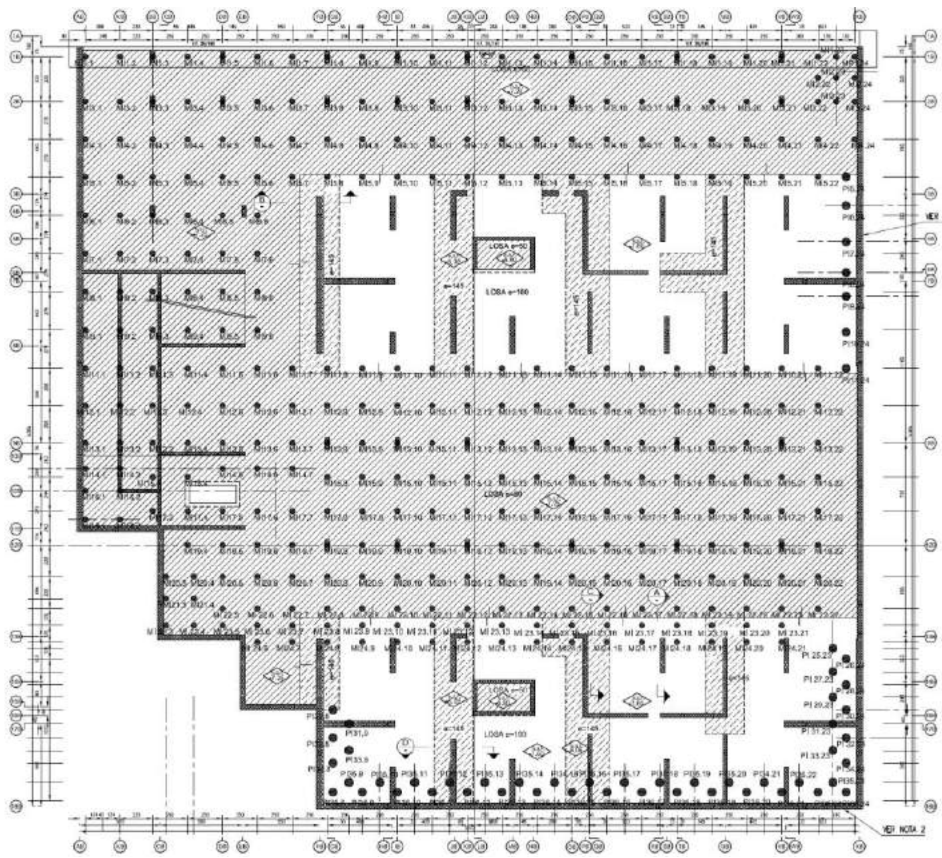


Figure 7 – Distribution of the required foundation reinforcement [16]

Total Quantity	**Design (ultimate) load [ton]**
516	40
104	53
32	71

Table 2 – Summary of the required reinforcement for tension loads [16]

5.3 Geotechnical design of the uplift reinforcement

There is no official Chilean norm for the design of special geotechnical works, thus the geotechnical design of the reinforcement elements was carried out in compliance with the German Standard DIN 1054:2010-12 and the EAP2007, considering the safety concept based on the *Partial Safety Factor Approach*, where the relationship:

design **E**ffect of actions ≤ **d**esign **R**esistance

$$E_d \leq R_d$$

has to be verified for all limit states (ultimate and serviceability). Figure 8 displays the geotechnical verifications required for the design of the uplift reinforcement and the results are summarized in Table 3.

The reinforcement was materialized with self-drilling Ischebeck TITAN grouted micropiles. Micropiles transfer the loads (tension and/or compression) coming from the structures to the foundation ground over skin friction.

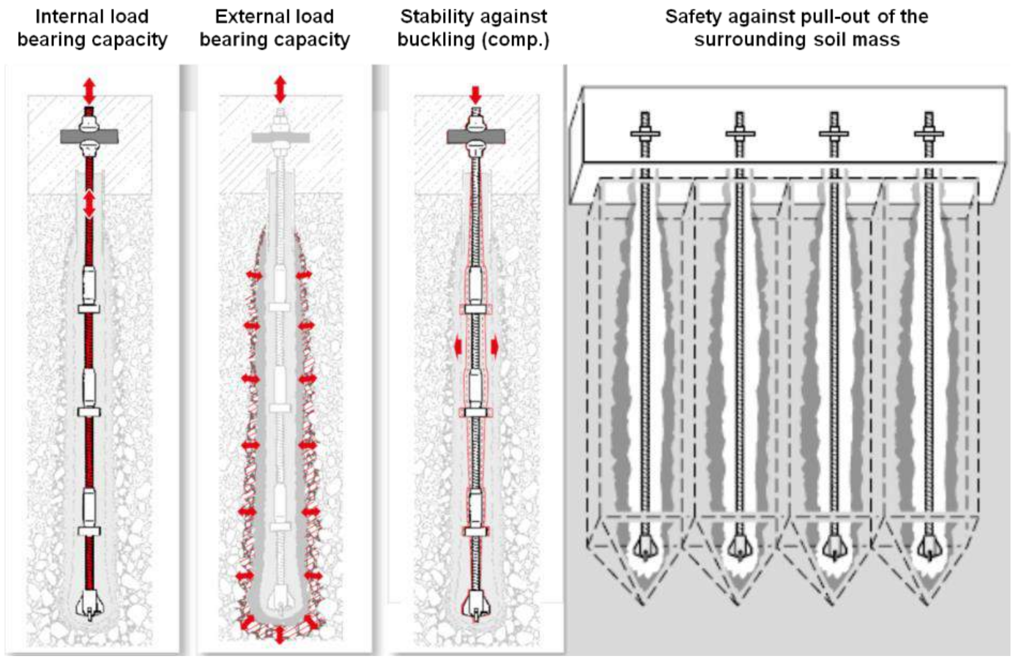


Figure 8 – Geotechnical verifications for the design of micropiles [7]

The TITAN micropiles consist of continuously threaded hollow bars, made out of seamless fine-grained steel pipes (S460 NH), installed via rotary percussive drilling. During the drilling process, the micropiles are continuously grouted (dynamic injection), building a rough interlocking at the interface grout-soil, increasing the skin friction. According to [10], the characteristic skin friction value of $\mathbf{q_{s,k} = 215kN/m^2}$ was adopted for the layer H3.

The components, the installation process and a typical cross-section of the grouted body are presented in Figure 9.

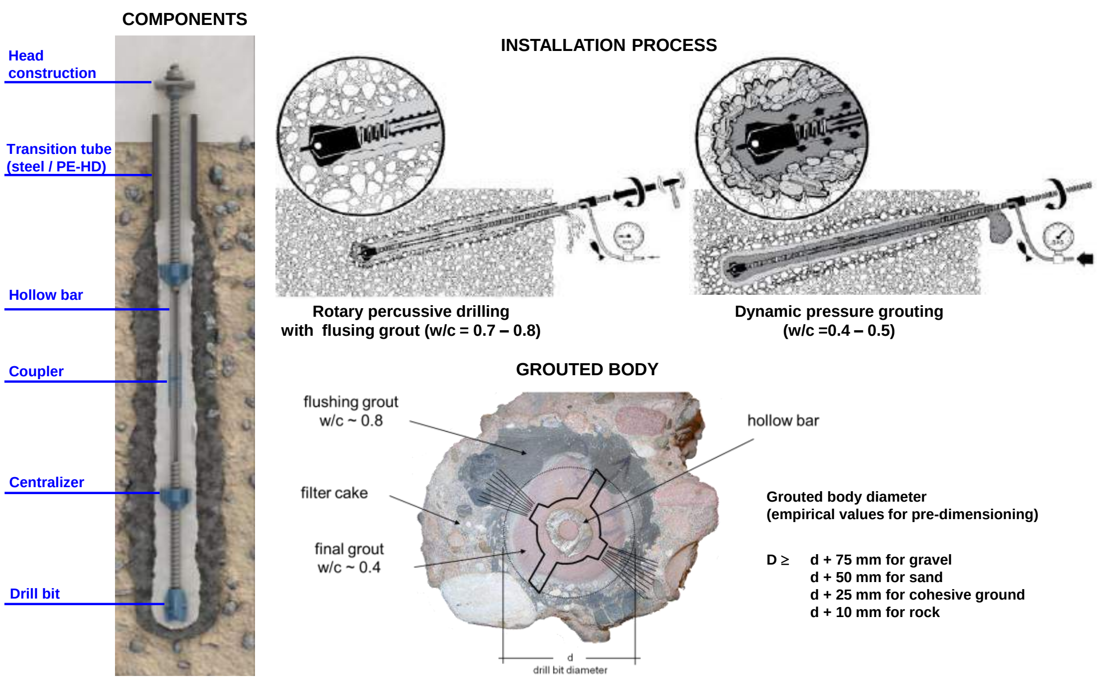


Figure 9 – TITAN System: components, installation process and grouted body [7]

Quantity	TITAN micropile	Design tension load (ultimate)	Load bearing capacity					Pull-out cone
			Internal	External				
		E_d [ton]	$R_{M,d}$ (1) [ton]	d [mm]	D (2) [mm]	L_{min} [m]	R_d (3) [ton]	L (4) [m]
516	40/16	40	41.5	90	140	7.0	44	7.0 – 14.0
104	52/26	53	56.5	130	180	10.0	57	10.0 – 21.0
32	73/53	71	77.3	130	180	10.0	80	10.0 – 13.0

(1) $\mathbf{R_{M,d} = R_{M,k} / \gamma_M}$: $R_{M,k}$ according to [11] for a minimal grout covering c = 40mm. γ_M = 1.15 according to [5]

(2) Diameter of the grouted body, with an extension of 50mm (sandy soils,

(3) Figure **9**)

(4) $\mathbf{R_d = \pi * D * q_{s,k} * L / \gamma_p}$: $q_{s,k}$ = 215 kN/m^2 according to [10]. γ_p = 1.5 according to [5]. Resistances associated to displacements ≤15mm

(5) Required length depending on the micropiles separation (s_x, s_y)

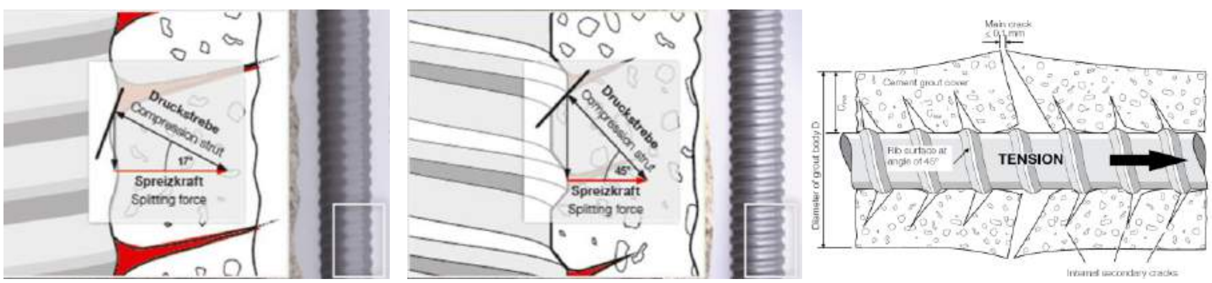
Table 3 – Uplift reinforcement – Design summary [16]

5.4 Durability

For permanent reinforcement systems, the design loadbearing capacity needs to be guaranteed during the serviceability of the planed structures. In the case of micropiles, it must be ensured that the steel load bearing elements are effectively protected against corrosion.

The permanent corrosion protection of 100 years of the TITAN micropiles is provided only by meanings of sufficient grout cover, as highlighted in the National Technical Approval Z.34.14-209, granted by the German Institute of Building Technology (DIBt) [11].

The steel quality and thread geometry of the TITAN hollow bars induce a regular cracking pattern in the grouted body, with crack widths smaller than 0.1mm, considered to be self-healing (Figure 10).



R-thread (acc. To ISO 10208) **TITAN-thread (acc. To EN 10080)** **TITAN-thread (crack width)**

Figure 10 – Cracking pattern and splitting forces on R-threaded hollow bars (left), on TITAN hollow bars (middle) and crack width limitation in the grouted body (right) [7]

5.5 Installation of the micropiles and load tests

The micropiles were installed from the bottom of the excavation pit (Figure 11). Up to three drilling machines were used for the installation: two Tamrock rigs and one Morath drifter (HB70), attached to a telescopic jib (Manitou). The achieved drilling performance was approx. 100m/day/equipment.

After installation, load tests were carried out in order to verify the adopted design considerations, especially regarding the skin friction of the layer H3. Three test micropiles were executed: 1x73/53, 1x 52/26 and 1x40/16. The micropiles were subjected to maximum test loads, equal to 90% of the yield force (at 0.2% elongation) for the correspondent micropile type, without reaching the ultimate limit state of the pull-out resistance (Figure 12). The required safety level and the adopted design considerations were validated. The registered displacements for the design loads were between 9 and 13mm.

Figure 11 – Installation of the micropiles (left) and load tests (right)

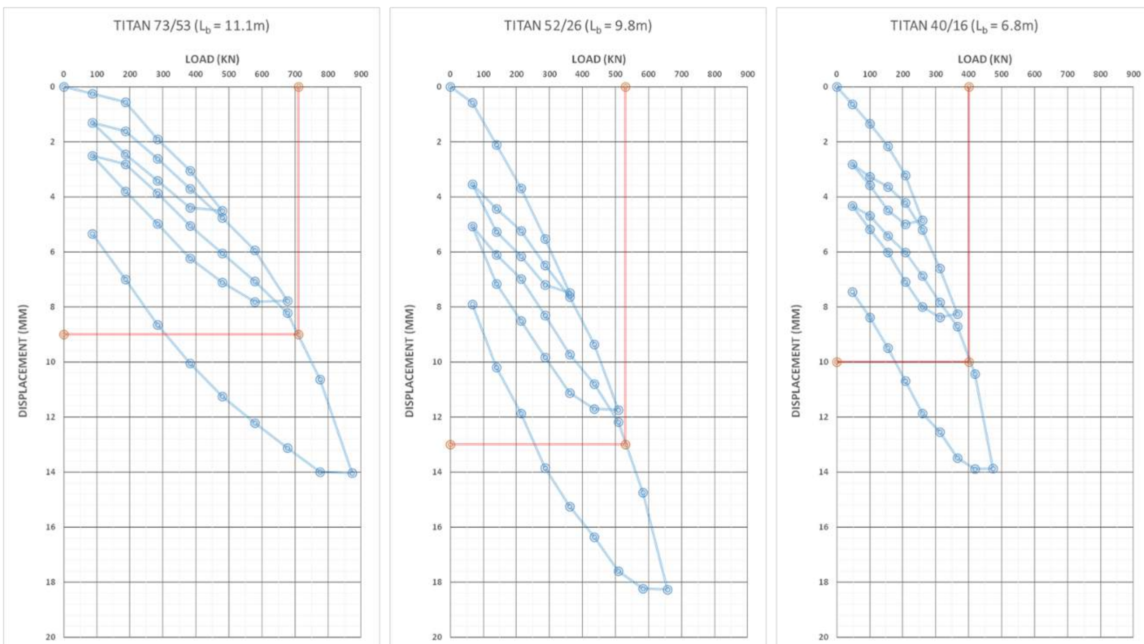


Figure 12 – Results of the executed load tests

According to [11] the required grout cover for the micropiles considered in the design was 40mm, in order to guarantee the permanent corrosion protection. The measurements at the micropiles necks showed grout covers of at least 45mm, fulfilling the requirements for the durability.

6. INTERACTION WITH OTHER RELEVANT PROJECT ITEMS

During the preliminary engineering approach, a 2.5m-thick bottom slab was consider to resist the uplift forces thus for the planned two underground parking levels, the project required an excavation pit with a free height of 9.5m. For this matter, a temporary excavation shoring consisting of an anchored soldier pile wall and a network of well-points to lower the groundwater had to be implemented. The corresponding requirement is schematically presented in Figure 13.

The implementation of the presented uplift reinforcement solution had also a positive effect in the temporary shoring and groundwater lowering, since the excavation depth was considerably reduced to 7.6m, making possible to optimize the design of the above mentioned items (Figure 14).

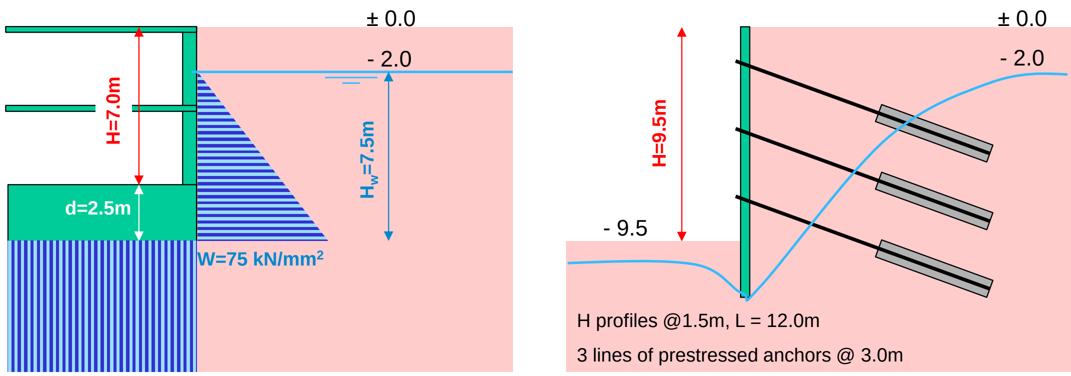


Figure 13 – Preliminary design approach (left) and constructive measures for the temporary excavation shoring (right)

The lateral support for the soldier pile wall was also materialized with TITAN tension piles (passive anchors). This solution was also proven to be more convenient than the originally considered use of strand anchors, since the higher installation speed of self-drilling anchors (>100meter/day) enabled to finish the excavation shoring faster. Furthermore, the installation of anchors and micropiles was carried out using the same equipment, simplifying the logistic at the construction site and reducing its costs (i.e. mobilization).

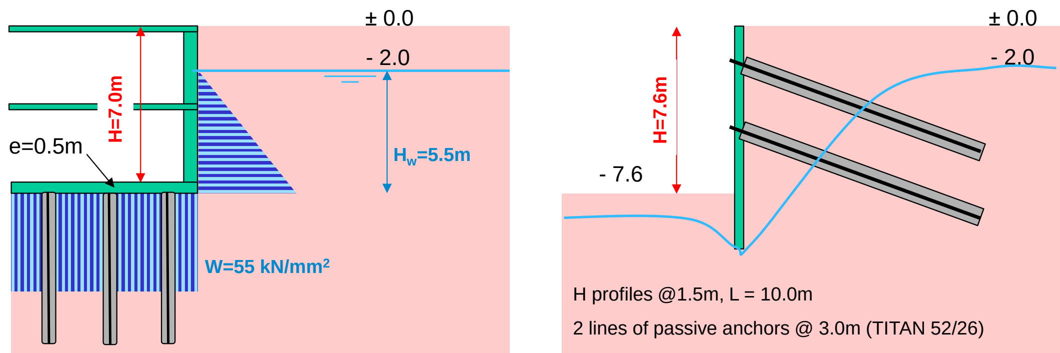


Figure 14 – Adopted design approach (left) and constructive measures for the temporary excavation shoring (right)

7. SUMMARY AND CONCLUSIONS

The present document described the implementation of an uplift reinforcement system for an 18-storey building located in the city of Concepcion-Chile, consisting of self-drilling grouted micropiles.

The difficult board conditions of the project, mainly related to the seismic activity of the region as well as the local geology, imply a high complexity for the design, in order to provide optimal solutions that fulfill the safety requirements stablished by the national construction regulations.

The presented solution highlights the technical benefits of micropiling, showing that its implementation can represent significant reductions on relevant items, such as the requirement of large amounts of reinforced concrete with the associated logistic involved in its time-consuming preparation and installation. Other relevant project items, such as the temporary shoring and groundwater lowering, necessary to materialize underground levels can also be optimized, having a favorable impact in the project as a whole, in terms of structural requirements, execution time and costs reduction.

The opportune interaction between structural and geotechnical designers is required, from the early stages of the planning process on, in order to facilitate and optimize the processes involved in the design.

It is evident that the use of micropiles as uplift reinforcement systems can be applied to other types of infrastructure, such as road and rail underpasses, caissons, tunnels, etc.

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