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The use of micropiles for the seismic reinforcement of a waterfront structure

F. Lopez
Friedr. Ischebeck GmbH, Ennepetal, Germany

A. Guzman & J.M. Fernandez
Pilotes Terratest S.A., Santiago de Chile, Chile

ABSTRACT: The following article presents the case study of the reinforcement of an approx. 300m-long quay wall (mole), located in Talcahuano, Chile. The mole was erected as a gravity wall around 1916 and is constituted by large concrete blocks.

After the Mw 8.8 earthquake of February 27th 2010, the structural assessment of the mole revealed the occurrence of large deformations, which increased the risk of internal and external stability losses that could lead to the structural collapse.

The reinforcement, materialized by self-drilling grouted Ischebeck TITAN micropiles, was designed to restore the functionality and to guarantee the stability of the mole in the face of future seismic events. The design considerations (i.e. loading conditions, structural capacities, etc.) for the definition of the required reinforcement will be discussed. The proposed installation sequence and the constructive solutions implemented during the installation process will be presented. Finally, the results of the performed load tests will be briefly reviewed.

1 INTRODUCTION

1.1 *Chilean seismicity – The Maule Earthquake*

Chile is one of the countries with the highest seismic activity in the world. According to the United States Geological Survey (USGS 2018), 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, with a following tsunami that hit the coastal areas (harbors, docks, shipyards, etc.).

This event reached a high intensity (Figure 1) and left 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 according to the official estimates, a loss of about US$ 30 Billion was caused.

The earthquake-induced ground shaking had a total duration of about 140 s, with the strongest part lasting 40 -50 s (Alarcon & Franco 2010). In the region of strongest ground shaking, ground accelerations exceeded 0.05g for over 120 s and a peak (horizontal) ground acceleration of 0.65g was registered in the records corresponding to the Concepcion bay area (EERI 2010).

The earthquake and posterior tsunami caused significant damages in the coastal areas, affecting strongly the existing infrastructure (roads, lifeline systems, etc.) as well as ports, harbors, and industrial facilities along the shoreline (Figure 2).

1.2 *The "Molo 500 Sur" quay wall in Talcahuano*

Some of the strongest tsunami-related damage was recorded in the city of Talcahuano. Among other facilities, the naval port of Talcahuano was strongly affected by tsunami waves.

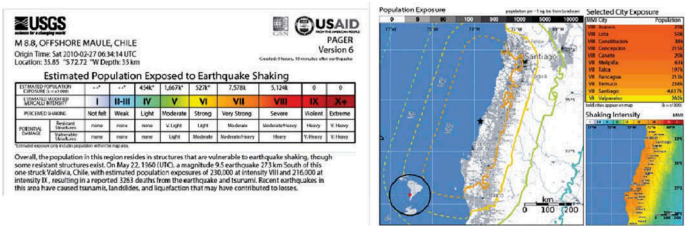


Figure 1. Intensity contours and estimated population exposed to the earthquake shaking for the Maule Earthquake of February 27, 2010 (after USGS 2018)

Figure 2. Left: damaged dock in San Antonio. Middle: Lateral spread at the Port of Coronel. Right: Tsunami damages near Navy facilities in Talcahuano (after GEER 2010)

The following document describes the reinforcement of the quay wall (mole) 500 (*Molo 500 Sur*), located in the navy area.

The mole is an approx. 300m-long gravity wall, which was erected around 1916 and is constituted by large concrete blocks, founded on a rockfill. After the Maule earthquake and posterior tsunami, the structural assessment of the mole revealed the occurrence of large deformations:

- rotations of −5.9° to +4.1°
- differential horizontal displacements as a result of lateral spreads
- differential settlements along the quay wall

The assessment of the quay wall stablished that the first 90m from the south (Sector A, Figure 3) had a good behavior after the earthquake, therefore no further constructive measures were required. On the other hand, the estimated rotations towards the sea (negative rotations) in the last 200m to the north (Sectors B and C, Figure 3) increased the risk of internal and external stability losses that could lead to localized structural collapse, in the case of a future seismic event.

1.3 *Description of the geotechnical context*

The geotechnical parameters for the construction site were defined by the geotechnical prospection. The layering is schematically presented in Figure 4 (left) and the parameters are summarized in Table 1.

The ground water table at the back of the quay wall is defined to be at a level of +0.6m (3.4m below the ground surface). According to the Chilean Seismic Zoning, defined in the Chilean Code NCh 433.Of96, the project site is located in the Zone 3, with an effective seismic acceleration Ao = 0.4g (Figure 4, right).

2 PROPOSED REINFORCEMENT

In order to guarantee the stability and serviceability of the quay wall in the unstable sector (Figure 3, sectors B and C) in the face of future seismic events, a reinforcement system, materialized by a group of self-drilling grouted Ischebeck TITAN micropiles, was proposed. The layout of the reinforcement is presented in Figure 5.

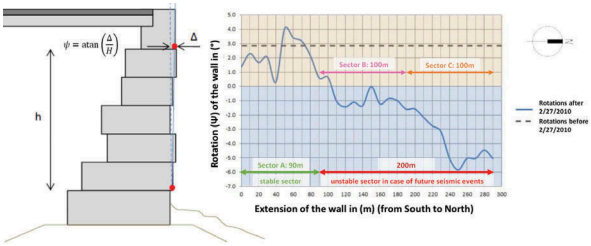


Figure 3. Estimated rotations along the quay wall, based on submarine measurements (modified after PRDW 2010)

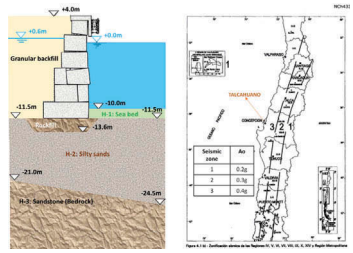


Figure 4. Left: Geotechnical conditions. Right: Seismic zoning of the site (modified after NCh433 1996)

Table 1. Geotechnical parameters according to the Geotechnical Prospection (PETRUS 2012).

Unit	Description	Friction angle φ' (°)	Cohesion c' (kN/m^2)	Bulk weight γ/γ' (kN/m^3)	N_{spt} (blows)	Unconf. comp. strength q_u (kN/m^2)
H-1	Sea bed (mud)	0	0	16/6	0-2	-
H-2	Silty sands	37	3	18/10	18-50	-
H-3	Sandstone	-	-	26/16	-	5000
Backfill	Granular material	36	0	16/8	-	-
Rockfill	Rock blocks	32	0	22/12	>50	-

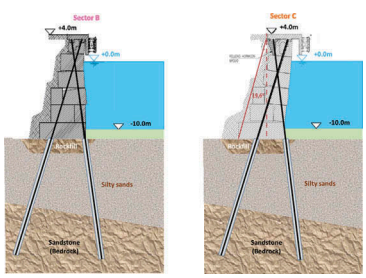


Figure 5. Proposed reinforcement with a group of micropiles for the unstable sectors B and C (PT 2014a)

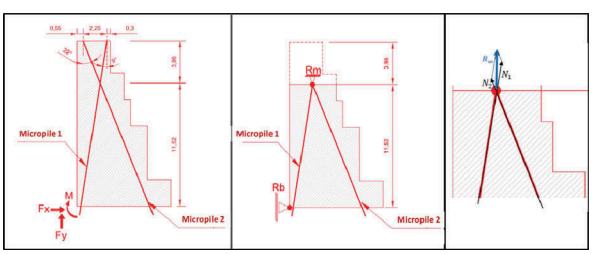


Figure 6. Proposed reinforcement with a group of micropiles for the unstable sectors B and C (PT 2014a)

3 DESIGN CONSIDERATIONS

3.1 *Structural model*

The analysis was carried out, based on a simplified model as shown in Figure 6. In structural terms, the micropiles "sew up" the concrete blocks, so that the quay wall is considered as a monolithic, rigid element that is anchored to the underground again through the micropiles, which are conceptualized as hinged rods, able to transfer axial forces only.

For the equilibrium analysis, all the forces acting on the wall are transferred to the toe, to obtain the reactions Fx, Fy and M (Figure 6, left).

The reaction (Rm) provided by the micropiles, and the horizontal (base) reaction (Rb) provided by the friction between the wall and the rockfill (δ_b = 32°, PETRUS 2012) can be obtained from the statically determined supported element (Figure 6, middle). Finally, the axial forces acting on the micropiles (N_1 and N_2) can be obtained from the resulting reaction (Rm) (Figure 6, right).

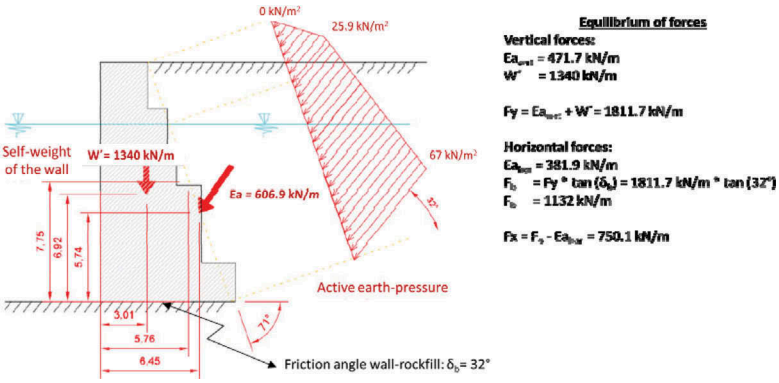


Figure 7. Equilibrium analysis of the quay wall under static load conditions (modified after PT 2014a)

The structural assessment showed that both sectors (B and C) were stable under static loading conditions, mainly influenced by active earth-pressure of a granular backfill acting on the wall as well as its self-weight. Figure 7 presents the estimated static loading condition of the quay wall, from which it can be inferred that in order to verify the equilibrium of forces, the base reaction (Rb) cannot exceed the magnitude of the horizontal reaction (Fx). For the subsequent verifications of the safety against sliding, the admissible base (horizontal) reaction of the quay wall is conservatively limited to the value of Rb_{adm} = Fx = 750.1 kN/m.

3.2 *Loading*

Since it was accepted that the quay wall is stable under static load conditions, the reinforcement with micropiles was conceived to resist the effects of:

- operational loads: traffic, mooring loads on the bollards (designed for 300kN and 1000kN) and fender forces (berthing)
- hydrostatic water pressure and hydrodynamic actions of surge
- loading situations coming from seismic actions: inertial forces for horizontal and vertical accelerations, hydrodynamic water pressure and the hydrostatic pressure as a result of a tsunami

The loading conditions, relevant for the design of the reinforcement are presented schematically in the following Figures:

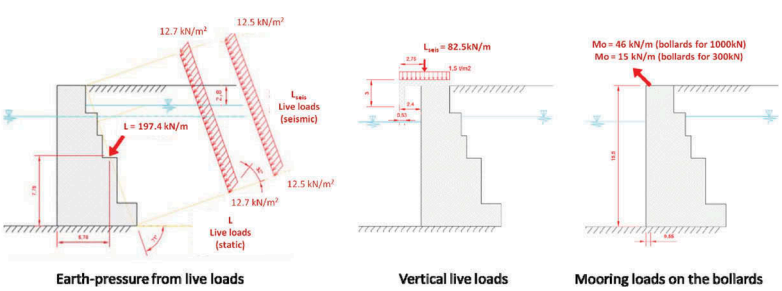


Figure 8. Effects (static and seismic) of live loads (modified after PT 2014a)

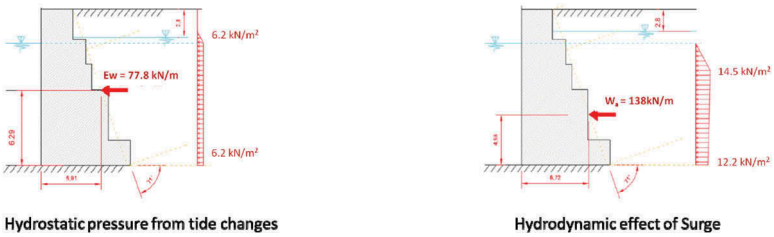


Figure 9. Hydrostatic water pressure from tide changes and hydrodynamic effect of surge (modified after PT 2014a)

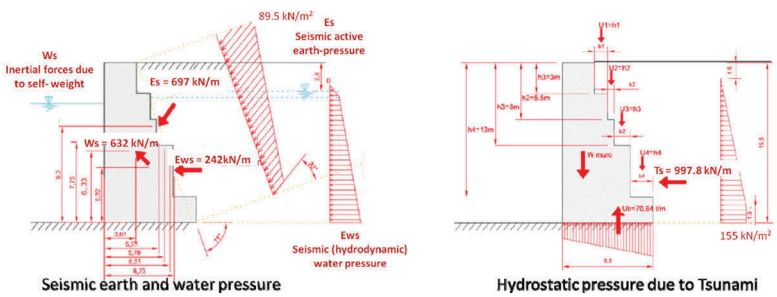


Figure 10. Loading situations from seismic actions (modified after PT 2014a)

3.3 *Design of the micropile group*

The reinforcement was materialized with self-drilling Ischebeck TITAN grouted micropiles, which consist of continuously threaded hollow bars, made out of seamless fine-grained steel pipes (Grade 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 adherence or skin friction (Lopez & Fernandez 2017). The components, the installation process and a typical cross-section of the grouted body are presented in Figure 11.

The inclination angles and the separation between micropiles were iteratively obtained, in order to verify the internal capacity (structural resistance) of the load bearing elements. For the project, micropiles from type TITAN 103/51 were used. The admissible internal load bearing capacity is obtained from the formula:

$$R_{M,adm} = R_{0.2}/\eta_M \tag{1}$$

where $R_{M,adm}$= admissible internal capacity; $R_{0.2}$ = Yield force at 0.2% elongation; and η_M = safety factor for the materials.

The disposition of the micropiles for the sectors B and C (Figure 5) is schematically presented in Figure 12.

Once that the loads acting on the micropiles were obtained, the bonded length of the micropiles was determined using the following formulae:

$$R_{ult} = \pi * d * \sum(\alpha_i * L_i * q_{s,i}) \tag{2}$$

$$R_{S,adm} = R_{ult}/\eta_S \tag{3}$$

where R_{ult} = ultimate external capacity; $R_{S,adm}$ = admissible external capacity; d = diameter of the drill bit; α_i = expansion factor for the i-stratum; L_i = Length of the i-stratum; $q_{s,i}$ = skin friction (adherence) of the i-stratum; and η_S = safety factor for the skin friction (adherence).

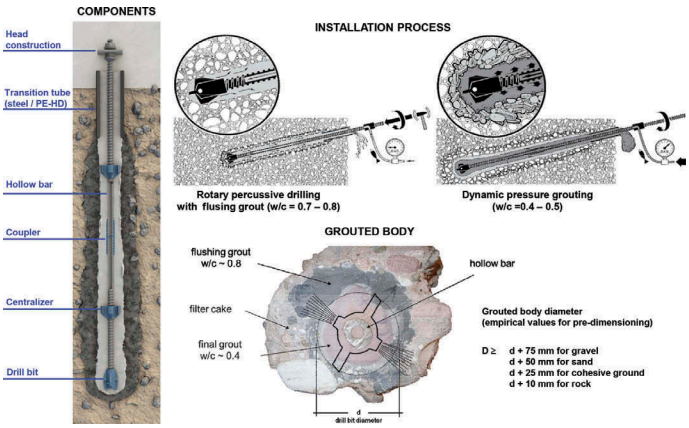
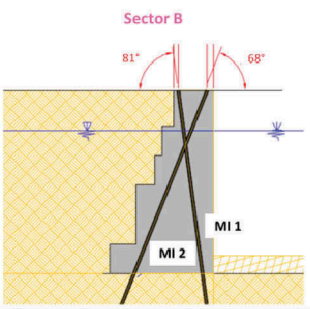
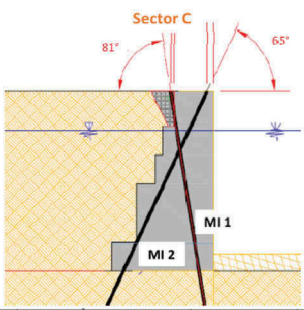


Figure 11. Self-drilling micropiles: components, installation and grouted body (Lopez & Fernandez 2017)





Bollard	Micropile	Inclination angle (from horizontal) α_h (°)	Inclination angle (from vertical) α_v (°)	Horizontal separation S_h (m)
300kN	MI 1	-81	9	1.38
	MI 2	68	-22	1.94
1000kN	MI 1	-81	9	1.40
	MI 2	68	-22	1.86

Bollard	Micropile	Inclination angle (from horizontal) α_h (°)	Inclination angle (from vertical) α_v (°)	Horizontal separation S_h (m)
300kN	MI 1	-81	9	1.34
	MI 2	65	-25	1.78
1000kN	MI 1	-81	9	1.32
	MI 2	65	-25	1.71

Figure 12. Disposition of the micropiles. Left: Sector B and Right: Sector C (modified after PT 2014a)

Based on the experience in the site conditions, the skin friction (adherence) values (q_s) listed on Table 2 were adopted for the different geotechnical units, disregarding the bond at the interface grout - concrete block walls. The results of the analysis are summarized in Table 3.

4 EXECUTION AND TESTING

4.1 *Execution of the works*

The micropiles were executed during 2014. The installation was carried out with the following sequence:

- Pre-drill (with casing) through the concrete blocks of the quay wall and the undelaying rockfill.
- Pre-Injection of the boreholes. The boreholes were pre-injected with a bentonite-grout mixture (unconfined compressive resistance of 10N/mm^2). Since larger cavities inside the wall

Table 2. Skin friction values adopted for the design (PT 2014a)

Unit	Description	Available length L_i (m)		Drill bit d (mm)	Expansion[(1)] α_i (-)	Skin friction q_s (kN/m^2)
		MI 1	MI 2			
Quay wall	Concrete blocks	15.0	15.0	175	1.0	-
Rockfill	Rock blocks	2.0	2.2	175	1.0	300
H-2	Silty sands	16.0	21.0	175	1.5	150
H-3	Sandstone	>5.0	>5.0	175	1.0	750

(1) Borehole expansion based on experience values (Ischebeck 2009)

Table 3. Reinforcement system – Design summary (PT 2014a)

Sector	Bollard	Max. Service Load (kN)		Adm. load bearing capacity (kN)		Max. Length[(3)] (m)
		MI 1	MI 2	Int. $R_{M,adm}$[(1)]	Ext. $R_{S,adm}$[(2)]	
B	300kN	1739	-2082	2115	1843/2040	28.0/32.0
	1000kN	1803	-2066	2115	2182/2111	33.0/33.0
C	300kN	1801	-1938	2115	2196/2270	33.0/35.0
	1000kN	1371	-1126	2115	1885/2232	36.0/35.0

(1) With a safety factor for materials η_M = 1.3 (for seismic loading conditions)
(2) MI 1/MI 2. With a safety factor for skin friction η_S = 1.75 (for seismic loading conditions)
(3) MI 1/MI 2

were detected, this step was carried out to minimize the grout consumption, confining the grout during the actual installation of the micropiles.

- Installation of the self-drilling TITAN micropiles from the top of the quay wall and through the pre-injected boreholes.
- Execution of the micropiles´ head connections with the top slab. The mechanical connection is obtained using a washer plate, fixed to the load bearing elements with two collar nuts.

Figure 13. Installation of the micropiles (modified after PT 2015)

4.2 *Testing*

Early 2014, two suitability load tests (under tension) were carried out to validate the adopted design considerations, especially regarding the skin friction (adherence) values for the natural subsoil (silty sands and sandstone). The installation protocols of the testing micropiles showed a very good agreement between the recorded grout consumption and the expected (average) borehole diameters. A maximum load test of P_p=2340kN was applied to the testing micropiles, with displacements ranging from 25mm to 35mm (Figure 14). The observed load bearing behavior verified satisfactory the structural requirements.

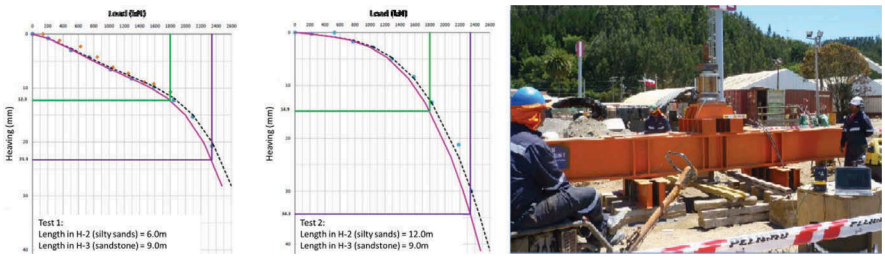


Figure 14. Load tests. Left: Load-displacement curves. Right: Testing structure (modified after PT 2014b)

5 SUMMARY

The structural assessment of an approx. 300m-long quay wall (*Molo 500 Sur*), located at the naval port of Talcahuano, stablished that large deformation took place after the Maule earthquake of February 27, 2018. Although the quay wall was stable under static loading conditions, the observed rotations towards the sea (negative rotations) in the last 200m to the north increased the risk of internal and external stability losses that could lead to localized structural collapse, in the case of a future seismic event.

A reinforcement system materialized by self-drilling grouted Ischebeck TITAN micropiles, was designed and implemented to restore the functionality and to guarantee the stability of the quay wall in the face of future seismic events.

After performing two suitability load tests on the project site to validate the adopted design considerations, especially regarding the external load bearing capacity of the micropiles (governed by the skin friction of the subsoil), nearly 9800 lineal meters of self-drilling micropiles TITAN 103/51 were installed during 2014.

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