Soil-structure interaction of a pile wall at the toe of a natural slope

Interacción terreno-estructura en una pantalla de pilotes al pie de un talud

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RESUMEN

Para la construcción de una nave industrial se ha realizado una excavación de unos 10 metros de profundidad al pie de una ladera de pendiente elevada y con signos de inestabilidad. Esto llevó a adoptar una solución mediante excavación con una pared de pilotes con anclajes al terreno a varios niveles. Por otro lado, se ha mejorado la estabilidad de la ladera con elementos hincados y con un sistema de drenaje interno. También se han estabilizado las zonas superficiales del talud que tenían más pendiente con mallas de acero bulonadas. Para el diseño se llevaron a cabo análisis de equilibrio límite y por elementos finitos. Asimismo, se han instrumentado la excavación y talud mediante inclinómetros, dianas y células de carga.

ABSTRACT

The construction of an industrial unit required the execution of a 10-meter excavation at the toe of a slope with high inclination and signs of instability. This led to solution of excavation with a soldier pile wall with several anchorage levels. On the other hand, the stability of the slope has been improved with driven steel piles (rail segments), as well as an internal drainage system. It was also necessary to stabilize the superficial areas with a bolted steel mesh. For the design of the adopted solutions, limit equilibrium and finite element analyses were performed. Furthermore, field monitoring systems were installed, such as inclinometers, load cells on the anchors and conventional topographical targets.

PALABRAS CLAVE: inestabilidad de talud, pantalla de pilotes, análisis por elementos finitos, instrumentación.
KEYWORDS: slope instability, soldier pile wall, finite element analysis, field monitoring.

1. Introduction

The construction of a 280 m by 90 m industrial unit required a 10 meter-deep excavation at the toe of a natural slope. This slope shows signs of previous instabilities, which made it necessary to design and construct a series of retaining structures, as well as an intensive monitoring of the slope and the surrounding structures.

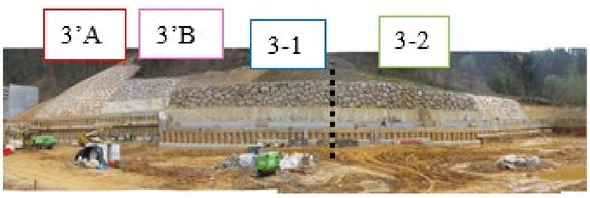

Figure 1. General view of the pile wall. Critical zone in April 2014, before reaching the maximum level of excavation.

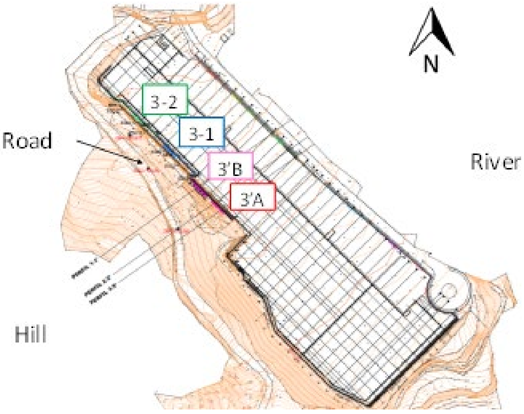

Figure 2: Top view of the industrial unit and the different zones of the pile wall under study.

The industrial unit is located between a river and a slope with signs of previous instabilities. The unit faces the NW-SE direction and has a slight positive inclination towards the SE side. On the NW side of the unit, a building with a basement was constructed, therefore, that area needed a deeper excavation. In order to execute the excavation, a soldier pile wall with variable height and composition was built. This wall has different section profiles (Figs. 1 and 2). Zones 3 and 3' have an 8.5 m maximum excavation depth and piles of 1 m of diameter, spaced centre-to-centre 1.2 and 1.1 m.

In Section 2, the signs of slope instability observed before and during the construction process are depicted. The geotechnical characterization of the area is presented in Section 3. Section 4 shows the limit state analyses and corrective measures adopted. The use of a soil-wall interaction model is presented in Section 5. Finite element calculations are explained in Section 6. Last of all, Section 7 describes the monitoring results and presents some conclusions.

2. Previous signs of slope instability

Before the construction of the industrial unit started, a series of levelling works were performed. As part of these, a riprap wall was constructed, on the northeast side of the plot (zones 3-1 and 3-2 of Figs. 1 and 2). On December 2010 and March 2013, two major landslides occurred near these locations. The first one was fixed building a riprap lining and, after a geotechnical study, an anchored concrete gravity wall was added at the lower part (zone 3'-B). The second landslide occurred after a heavy rainy period. The adopted correcting measures were to i) clean up the debris, ii) set back the slope 2-3 meters and iii) place a riprap lining above the anchored wall to refill the area (Fig. 1).

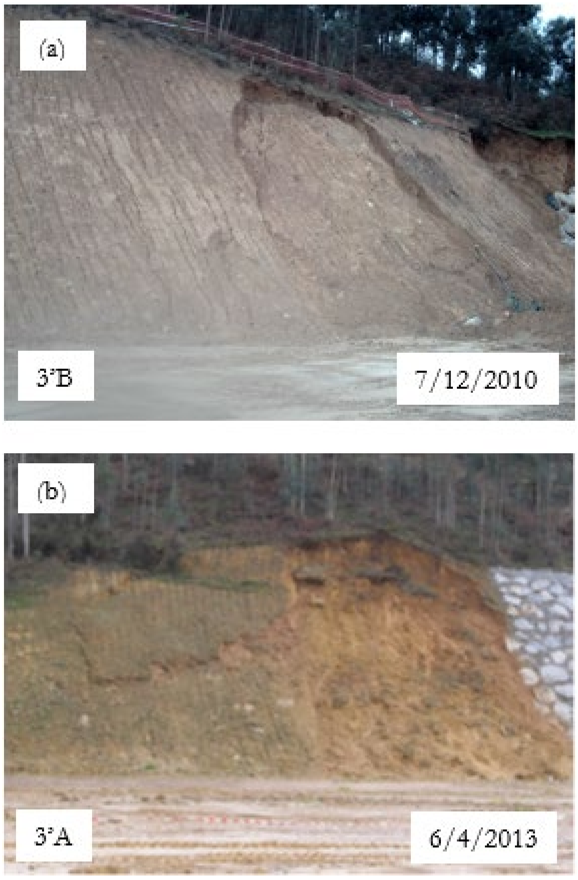

Figure 3: Landslides prior to the construction of the industrial unit. Top photo was taken on December 2010 and bottom one on April 2013.

Due to these instabilities, monitoring and control systems were set in place. In September 2011, after the first landslide, topographic tracking targets were placed on top of the riprap wall in zones 3-1 and 3-2. At the beginning of the construction in April 2013, movements of 2 cm were registered. This, added to the visible instability signs, led to the installation of inclinometer pipes at the backside of the riprap wall.

The excavation started as the construction of the bored pile wall finished, on December 2013. During the excavation in the winter of 2014, the monitoring devices recorded significant movements in front of the soldier pile wall. Therefore, the initially proposed solution was reconsidered.

3. Geotechnical characterization

According to the geotechnical studies, the natural slope behind the pile wall presents unfavourable geotechnical conditions (environmental and topographical), particularly the 3'A zone.

The geological map indicates two issues with the unit: the influence of the Laredo – La Peña fault, which has a NW-SE orientation, just like the industrial unit; and the composition of the ground, consistent of a clay and silt matrix with rock boulders, possibly due to Keuper diapiric phenomena. The presence of these blocks explain the problems when defining the location of the bedrock. They also help to explain the discontinuities found in the limestone when placing ground anchors and during the construction of the retaining wall. The different exploration boreholes performed detected old landslides. The site is also affected by a river, running in a NW-SE orientation, same direction as the fault and the building area. On both sides of the lot, two tributary streams flow to the main river.

The following site investigation and geotechnical parameters are based on: i) the results of three boreholes performed in the slope, and ii) information gathered during the construction of the anchors and piles. Two of the boreholes were performed in the beginning as part of a general site investigation, BH1 and BH2. The third one, BH3, was done later in May 2014 and it was located in the most critical area (3'A zone).

The slope is mainly composed by material from previous slips, described as sandy silt or silty clay with heavily altered sandstone gravel. At variable depths, fractured limestone blocks or actual bedrock can be found. In particular, borehole BH3 reached the bedrock at 10 meters. However, since perforations close to this borehole did not find bedrock at the same depth, extrapolating a profile has proven difficult.

Table 1. Soil parameters from BH1 and BH2.

Depth (m)	**c (kPa)**	**ϕ (º)**	**LL (%)**	**PI (%)**	**w (%)**
4.0-4.6*	40	23.0	32	12	15
9.0-9.6*	10	25.8	29	11	23
7.1-7.4**	25	24.5	52	24	25

* Triaxial CU Test
** Direct Shear Test

Samples were retrieved from the first two boreholes, BH1 and BH2, and they were used to perform several tests, from a series of common characterization tests such as SPT, particle size and Atterberg limits, to five pressuremeter tests, two C-U triaxial tests and one C-D reversal direct shear test. Table 1 presents soil parameters derived from these laboratory tests. The residual strength measured in the reversal direct shear test was 18º.

4. Limit State Analyses

4.1 Calculation assumptions and parameters

In order to perform a stability analysis, limit state methods by slices, assuming circular failure surfaces, were used. The Morgenstern-Price and the commercial code Slope/W [1] were applied, with the hypothesis of a semi-sinusoidal contact stress distribution between slices. Strength representative parameters were obtained from

Table 2. Material parameters for the limit state analyses.

Name	γ/γsat (kN/m3)	c (kPa)	ϕ (º)
Bedrock*	27	2000	-
Silt (shallow)**	20	5	26
Silt (deep)**	20	20	26
Riprap**	19	5	47
Wall (concrete)*	25	1000	-

Material model considered for the numerical code (Slope/W):

* Undrained

* Mohr-Coulomb

the results of the triaxial and direct shear tests (Table 1). Since the failure surface considered involves the whole soil mass, the value of the residual parameters was not considered. These could be representative of a possible ancient slip surface, but not of the whole soil mass.

On the other hand, it is common for the soil mass to have a more altered and less resistant superficial area, which generally affects the cohesive parameter. To define the depth of said layer and its resistance, back-analyses of recent slides of the slope were stablished. The following section depicts them further, but Table 2 summarizes the different geotechnical units and the calculation parameters.

4.2 Retrospective analysis of previous landslides

The objective of these analyses was to validate and to have a better knowledge of the geotechnical parameters and calculation hypothesis employed. Two previous landslides were studied: the one that took place on March 2013 on the area with a wall profile 3A (Fig. 3b) and one of the seasonal slides that ocurr on the excavated slopes on the edge of the unpaved path of the hillside (Fig. 2).

The analysis of the March 2013 slide allowed checking a critical slope stability situation (S.F. =1.00). A high ground water table (GWT) was considered, corresponding with the

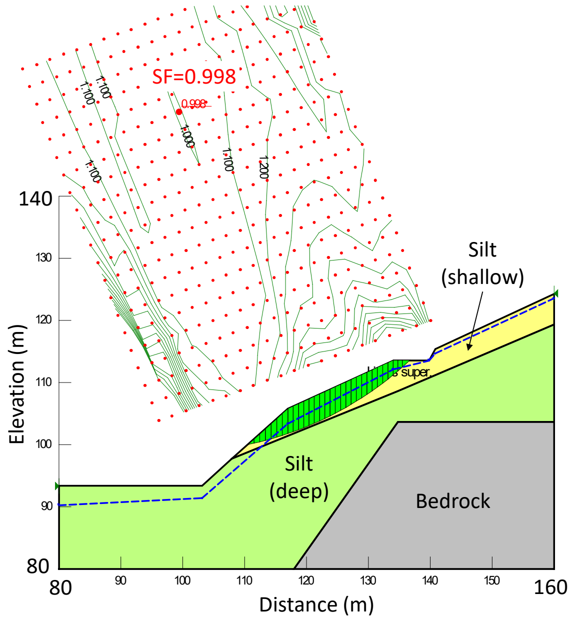

Figure 4. Back-analysis of March 2013 landslide.

heavy rain situation of the period. The failure surface was observed to be 3-4 meters deep, same as a preferential water flow surface sub-parallel to the slope. The material of the slope was considered to be more altered at the surface and, therefore, in the limit equilibrium analyses, two different materials were used, one for the first 5 meters with a lower cohesion (5 kPa) and the remaining soil mass with a higher cohesion (20 kPa) (Table 2). Both materials have the same friction angle. This distinction allowed taking into account that, on the lower part of the slope, while the levelling works were under way, the superficial layer of soil was removed. Thus, the existing soil should have the characteristics of the deeper material. This explains how the slide did not affect the toe of the slope, as it was observed during its stabilization.

On the side of the unpaved road that runs halfway up the slope seasonal instability phenomena appear on the excavated slope. These are reactivated on the following winter during heavy rain season. The slope has between a 2H:3V and 1H:2V gradient. The retrospective analyses confirm that this is a critical stability situation (S.F. =1.19). Nevertheless, this was an approximate calculation due to the uncertainties

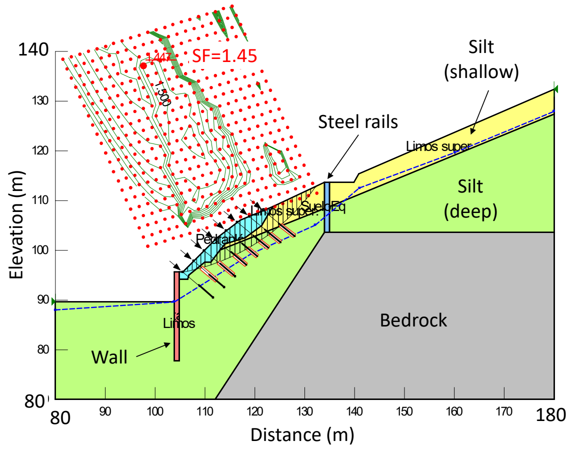

Figure 5. Stability analysis of the slope with corrective measures (horizontal drains, steel rails and bolted mesh of steel cables). Shallow slip circle.

of the slope profile, the depth of the GWT, the small size of the instabilities…

4.3 Slope stability analysis during the excavation

As it will be seen further into the article, on April 2014 during the excavation of the basement of the northwest side of the building, the monitoring results indicate worrying movements on the slope. This critical stability situation was confirmed by the limit equilibrium analysis (S.F. =1.12). The 3'A wall section was taken as critical for the entire analysis (Figs. 1 and 2). Given the geometry of the slope and the location of the bedrock, it is expected that the failure surface will neither go under the wall or through it, for the high resistance of the concrete (Φ 1000 piles, spaced 1.10 to 1.20 m). Therefore, the length of the retaining wall and the ground anchors are not relevant for the analysis and so, the wall was modelled considering only concrete resistance (Fig. 5).

4.4 Corrective measures

In order to improve the stability conditions of the slope, two corrective measures were proposed:

- Lowering of the groundwater table with sub-horizontal drains with an approximate length of 25 m in two areas: on the higher path and on the top of the wall.
- Executing a wall of driven rails braced by the top with an anchored concrete beam. This solution was proposed because the path is an easy access point for trucks. Moreover, a few cracks were registered on the path, consequence of the landslides, and it was a quick way to protect it.

Table 3. Summary of the factors of safety for the two major corrective measures and their combination.

Analysis	Safety factor
April 2014 (high GWT)	1.12
Lowering GWT (horizontal drains)	1.33
With driven rails	1.21
Sub-horizontal drains and driven rails	1.44

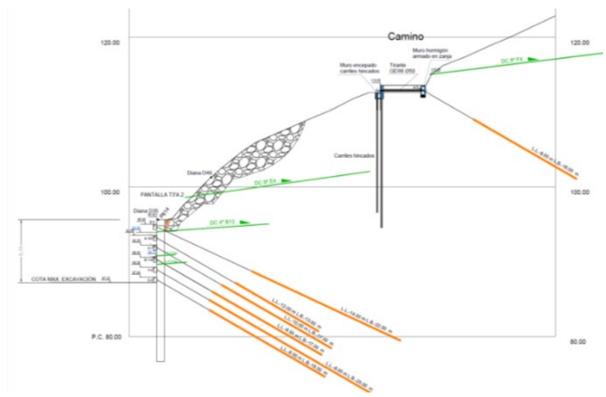

Figure 6. Sketch of corrective measures (long bolts and mesh of steel cables not shown).

The driven rails were arranged in two staggered rows spaced 0.6 m and 1 m between rails. The rails were 54 kg heavy and 5.8 m long. The planned length of each rail was 10 m but refusal was achieved between 7 and 30 m due to the irregularity of the bedrock location. The rails were place on the side of the path. Given the impossibility of executing the anchors in that area, they were done on the inside (superior) margin of the path (Fig. 6). Seven anchors of 40 T, spaced 3.5 m, were disposed. Their inclination was 30º, with a free length of 8 m and a bonded length of 16 m. A concrete beam braced the heads of the anchors and this beam was then

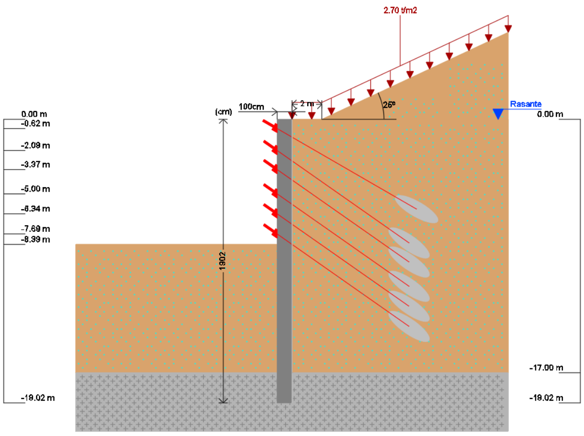

Figure 7. Soil-wall interaction model.

attached to the rail wall with Gewi ϕ50 rods, prestressed at 18 T.

In order to model the driven rails with the Slope/W code, an equivalent material was used. It had a shear resistance of 300 kPa, equivalent to the sum of the 10 T/ml reaction forces on the head of the rails and the 20 T/ml of passive resistance at the contact between the rails and the bedrock, caused by the embedment.

Table 3 summarizes the safety factors obtained for the different combinations of the corrective measures. Altogether, with the two proposed measures, the safety factor reaches an acceptable value of 1.44. In this analysis the considered surfaces have an effect on the whole slope around the excavation (up to 50 to 60 m behind it), and the shallow slip circles that affect the riprap lining have been discarded. For these shallow slip circles, the safety factor is still below 1.25. They are local and do not correspond to a generalized failure, but they can mobilize a considerable soil volume which fall could affect the warehouse in the future.

The low safety factor value for these shallow slip circles is consistent with the registered movements on the targets that where located on this lining zone. For all that, it was considered necessary to set a steel mesh fixed by long bolts over the riprap. This solution permits an increase in the slip safety factor of the shallow circles up to similar values of those of the deep circles and close to 1.5 (Fig. 5).

Table 4. Soil parameters for the finite element analyses.

	Bedrock	**Silt (shallow)**	**Silt (deep)**	**Rock fill**
Model	Elastic	HS**	HS**	M-C*
γ/γ_{sat} (kN/m³)	27	20	20	19
p'_{ref} (kPa)	-	100	100	-
m	-	0.5	0.5	-
E_{refoed} (MPa)	40000	20	20	30
E_{ref50} (MPa)	-	20	20	-
E_{refur} (MPa)	-	60	60	-
ν_{ur}	0.3	0.2	0.2	0.3
c (kPa)	-	20	30	5
ϕ (º)	-	26	26	47
ψ (º)	-	0	0	5
OCR	-	1	1	-

* "Mohr-Coulomb" model

** Hardening Soil model

An additional advantage of this solution is that the steel mesh can also prevent the movement and fall of some riprap blocks or boulders that could impact with the building at the toe of the slope. Throughout its useful life, it will be necessary to carry out some supervision and maintenance tasks of this mesh as well as a possible elimination of loose blocks that have been contained.

5. Soil-wall interaction analysis

To study the performance of the wall, a soil-wall interaction analysis was considered using the code CYPE [2]. Using this approach, the wall is modelled as a Winkler beam and the soil as spring elements using a subgrade reaction modulus. CYPE is a software amply used in the civil engineering industry for its simplicity to

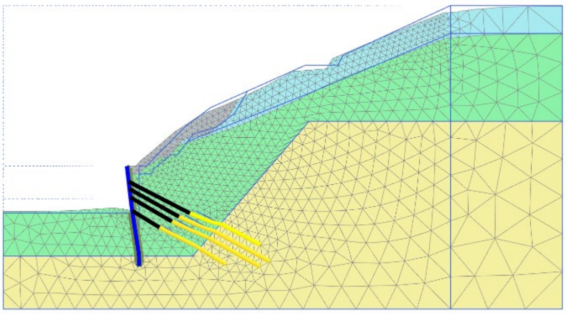

Figure 8. Finite element model and mesh. Predicted displacements (amplified by a scale factor of 20) in April 2014 (before adopting corrective measures).

Figure 9. Execution of the top row of ground anchors with temporary pile wall protection and backfilling of the excavation.

create different construction phases. However, in this case, the filling executed to install the superior row of ground anchors was not easy to reflect in the software, since it does not allow filling, only excavation.

Another inconvenient is that CYPE cannot model the specific soil profile behind the wall because of the inclined layering and the irregular ground surface, which led to an oversimplification of the ground. As a first approximation, a simplified model was considered consisting in one silt layer, a horizontal bedrock and a superficial load behind the pile wall for the riprap lining (Fig. 7).

In this case, a finite element analysis (described in the following section) was considered more suitable because of the importance in considering the slope at the back of the wall and its layering (Fig. 8).

6. Finite Element Analysis

The precarious stability situation of the slope during the first excavation works and pile wall anchoring provoked considerable movements on it. These movements led to important wall deflections and then to the increase of the anchor loads, above the initial prestressed loads. This caused its failure safety factor to lower notably and, in the case of the first row, came close to depleting.

It was considered appropriate to do a stress-strain model of the problem with the finite element code Plaxis 2D 2012 [3] to analyse this situation and the interaction between the slope, the wall and the anchors in order to assess and compare the movements of the wall and of the slope with the measured ones. This analysis helped as well to plan corrective measures to maintain adequate safe conditions on the anchors. An example of the model and finite element mesh used in the analysis can be seen in Figure 8. The analysis may be considered a type B prediction [4] because it was done during construction; available data of anchor loads and wall deflections were used to calibrate the model and the numerical simulations were used to propose corrective measures and predict future anchor loads, wall deflections and bending moments. Table 4 summarizes the parameters used in the simulations. The Hardening Soil (HS) model [5] was used to model de behaviour of the soil, both at shallow and deep levels. The cohesion values have been slightly increased from the limit equilibrium values to avoid local numerical problems.

The finite element analyses helped to prove the precarious stability situations of the slope and the increase in anchor loads. In the design stage four rows of anchors (named An1, An2, An3a and An3b) were planned and based on the performed analysis it was decided to add two additional rows (In1 and TopR). The first additional row, the intermediate In1, was

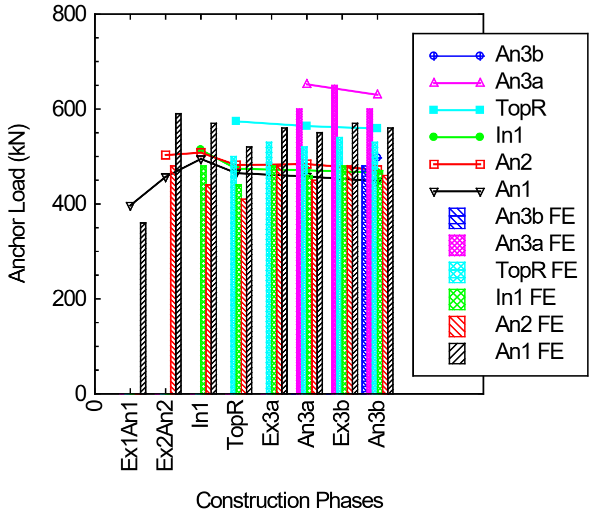

Figure 10. Anchor loads during construction process. Comparison between finite element simulations and measured values.

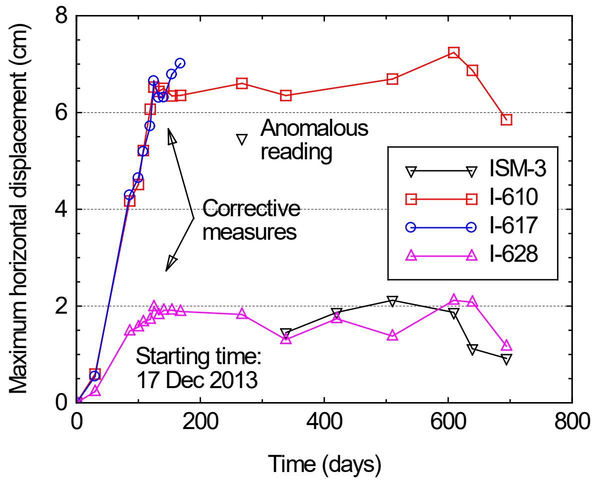

Figure 11. Evolution of maximum horizontal displacements measured at the inclinometer pipes.

installed at the level of the excavation on April 2014. However, the following analyses show the need for an additional row at the head of the wall (TopR) to discharge the initial anchors, which loads had increased notably. The execution of this superior row required the use of plastic and geotextiles for protection, as well as backfilling up to the top of the wall (Fig. 9). This was a satisfactory solution, as the load measures at the anchor heads lowered. Furthermore, the coincidence between the measurements and the predictions given by the numeric model has been adequate throughout the different phases of excavation and anchor execution (Fig. 10).

7. Field monitoring

Given the issues existing during the execution of the works, different measurement devices were installed, which allowed to perform an exhaustive monitoring of the slope along the 3 and 3' areas. In summary, the following devices were installed:

- Load cells in ground anchors (one per row) in Zones 3 and 3'.
- Inclinometers pipes on the back of the pile wall, four on Zone 3' and six on Zone 3.
- Topographical control targets, spread on the riprap lining and on the small anchored wall in zone 3 (Fig. 2).

The movements measured with the inclinometer pipes and the targets were notable until May 2014, with horizontal displacement rates up to 0.5 mm/day in the inclinometer pipes I-610 and I-617 (Fig. 11). This rate is in agreement with the safety factor calculated by limit equilibrium (S.F. =1.12) [6]. The measurements of horizontal displacements at the I-610 and I-617 inclinometer pipes showed maximum values at the top and null at the level of the bottom of the pile wall. This indicated a correct embedment of the pile wall and slope stability issues at its back.

Around the summer of 2014 the main corrective measure proposals were carried out (long sub-horizontal drain pipes, additional anchor rows, and driven rail wall in the slope), which helped to stabilize the movements since May 2014. From that date, the registered movements are practically constant (Fig. 11), but for slight oscillations. These could be attributable to seasonal movements, the accuracy and precision of the measurements or some additional works, such as the installation of the steel mesh fixed by long bolts in the summer of 2015 or the execution of the basement and floor slabs of the building.

8. Conclusions

The construction of an industrial unit required a 10-meter deep excavation. Close to it, a natural slope showed signs of previous instabilities. The proposed solution was to execute a soldier pile wall with several levels of ground anchors. During the excavation, notable horizontal movements were registered by field instrumentation and, therefore, corrective measures were adopted, such as long sub-horizontal drain pipes, additional ground anchor rows and a driven rail wall. The superficial stability was secured with a bolted steel mesh.

For the design of the adopted solutions, limit equilibrium and numerical analyses were performed. Finite element analyses were used to study the wall because, in comparison with the Winkler beam approach provided by CYPE, fewer assumptions needed to be made to model the soil mass behind the soldier pile wall. Besides, the soil behaviour was more realistically represented by the HS model. The results provided by the finite element analyses coincided with the field measurements.

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