DESIGN OF A PILE REINFORCEMNT OF A MOTORWAY SLOPE IN SOFT COHESIVE SOILS

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Abstract

Costanera Norte is a major concessionaire of urban motorways in Santiago de Chile. Among others they exploit the Radial NW Motorway to Santiago. Between Km 5+000 y 5+500, a series of instabilities were registered since August 2013.

At this sector, the slope height is 30 m with two benches, the lower bench has an inclination of 56,3° while the upper one is 45º. Between both benches a 4 m wide berm was disposed. The natural hillslope has a relatively step inclination and it is constituted of cohesive clay with boulders, typical from colluvial deposits. The water table is located at 1/3 of the slope height. In December 2006, a major slope collapse took place in the sector 5+180-5+240 and a rockfill wall was for this reason was constructed. In the winter of 2014 this collapse was reactivated coinciding with a hard rain period.

For this reason, a back analysis of the failure was carried out, using FLAC 2D code. As a result of this calculus a precise knowledge of the ground and the strength parameters was obtained. With these values a reinforcement consists in Californian drainages, and two pile-reinforcement walls were designed.

The construction of all the reinforcements was carried out at the beginning of 2015 with a very successful and satisfactory results.

Introduction

Costanera Norte is a major concessionaire of urban motorways in Santiago de Chile. Among others they exploit the Radial NW Motorway to Santiago.

The construction of this motorway was carried out between 2005 and 2006. As it crosses one of the pre-Andean reliefs, several tunnels and high slopes were necessary to be excavated. Basically, the road slopes have been excavated in rock materials, but in a section between Km 5+500 and 5+000, the slopes were constructed affecting clayey colluvial deposits.

At this sector, the slope height is 30 m with two benches, the lower bench has an inclination of 56,3° while the upper one is 45º. Between both benches a 4-m wide berm was disposed. The natural hillslope has a relatively step inclination and it is constituted of cohesive clay with boulders, typical from colluvial deposits. The water table is located at 1/3 of the slope height.

Since the finalization of the motorway construction, a succession of minor events was recorded between Km 5+240 and 5+140, from December 2006 to September 2008. For this reason, in the Northern part of the slope (right hand side in Figure 1), between Km 5+300 y 5+000, it was necessary to modify the inclination of the slope (down to 50º) and to reinforce the toe of the slope w0ith a rockfill wall. The result of these stabilization measures can be observed in **Figure 1**.

Five years later and in winter time, August 2013, some minor failures of the slope took place, making necessary to carry out partial re-excavation of the slope, in its Southern part, between 5+180 to 5+020 (left hand side of Figure 1). The problems continued and for this reason at the beginning of 2014 it was decided to make a global reparation of the slope in its section from 5+200 to 5+000.

Back analysis

In order to make an accurate design of the slope reinforcement measures firstly a back analysis of the failure was carried out.

Firstly, a sensitivity analysis of the shear strength values of the colluvial clays was done. For this purpose, a 2D limit state analysis approach was used. The methodology was to reduce the values of cohesion and friction angle, until reaching a Factor of Safety equal to one. This analysis was carried out using two stages. In a first stage, all the possible failure circle solutions for a given slope profile, are calculated. As a result of this first stage several couples of cohesion/friction are obtained. In a second stage, a given failure surface for the specific profile analysed is chosen This failure surface chosen is supposed to be the real one observed on site. Also for this specific failure circle it is possible to obtain couples cohesion/friction satisfying a Factor of safety equal to one.

Figure 3 shows the result of these analyses. In blue colour the first stage is represented, while the second stage corresponding to the observed site failure has been represented in red colour.

The intersection of both curves, give us the values of eth shear strength of the soil. In this case the following values are obtained:

- Cohesion: 10 kPa
- Able of friction: 17º

Once a first approach of the soft cohesive shear strength was done, a complete a detailed back analysis of the slope was done. This back analysis was done using FLAC 2D code (Itasca, 2008). As a result of this calculus a precise knowledge of the soil parameters necessary to reproduce the slope failure was achieved.

This methodology has been extensively used and several discussions about its use can be find in Lorig *et al.* (2000) and specifically for soft soils at Duncan *et al.* (2005)

The following constitutive models were considered:

- Elastoplastic Mohr.Coulomb for the clay rock substratum and rockfill wall.
- Elastoplastic Mohr-Coulomb with strain softening behaviour for the clayey colluvial deposits.

A value of Kkx = Kkz = 0,5 was considered for the natural stresses in the boundary conditions.

The water table is located at 1/3 of the slope height.

The use of a strain-softening constitutive model for the soft colluvial clays is highlighted, as this use can reproduce the progressive failure of the slope observed between 2006 and 2014. In this model it is possible to define the cohesion, friction angle and dilation as piecewise-linear functions of softening parameters measuring the plastic shear strain (Itasca, 2008), shown in **Figure 4**.

At each excavation phase, the safety factor was calculated reducing the shear strength (Dawson et al., 1998). And a safety factor of 1 was considered to reproduce the slope failure and consequently to calibrate the shear and deformability parameters of the ground.

As a conclusion of the back-analysis calculation, the strength and deformability values shown at the **Table I**, were derived.

Slope reinforcement design

Once an accurate knowledge of the ground parameters was achieved, the slope reinforcement measures can be designed in a rigorously way.

The slope reinforcement consists in a combination of draining measures and a pile-reinforcement. In detail the measures proposed were:

- Three rows of Californian drainages, in order to reduce pore pressure at the slope face
- Down load of the slope head, respecting the eminent domain as well as two existing electric poles
- Two retaining walls with reinforced piles (Ø 85 cm; 2.5 m spacing). A first one located at slope toe (l=4 m) and a second located at the inner slope berm (l=11 m)
- Concreted rock fill wall at the foot of the slope

Figure 6 shows a section of the slope with all the reinforcements.

In **Figure 7** these slope reinforcement measure are shown in a plan view of the motorway.

These measures were modelized using the same FLACE2D code, as can be shown at **Figure 8**.

In **Figure 9** the results of the calculation are included. As it can be observed the maximum moment is 2.2 mt/m while the maximum axial force at the structure is 5.8 t/m.

The resulting global Factor of Safety is 1.18, while this value decreases to 1.1 if the seismic action of 0.2g, according to the Chilean seismic regulations, is considered.

Construction

The construction of all the reinforcements was carried out at the beginning of 2015 with a very successful and satisfactory results.

Figures 10 and **11** shows respectively the retaining wall with reinforced piles located at the foot of the slope and the execution of the concreted rockfill wall. A detail of the grouting of the rockfill can be observed at **Figure 12**.

Finally **Figure 13** show the slope after the finalization of the stabilization construction measurements.

Conclusions

According to the construction experiences, the use of a back-analysis techniques in order to calibrate the ground shear and deformability parameters of a soft cohesive soil, is highly recommended.

References

Itasca Consulting Group, Inc., 2008. “FLAC2D Fast Lagrangian Analysis of continua in 2 Dimensions". Version 6.0. Minneapolis: s.n., (2008)

Lorig, L. and Varona, P. 2000. Practical Slope-Stability Analysis Using Finite-Difference Codes, Slope Stability in Surface Mining, Society for Mining, Metallurgy and Exploration (SME), Hustrulid, W.A., McCarter, M.K. & van Zyl, D.J.A. (eds.).

Duncan, J.M. y Wright, S.G., 2005. “Soil Strength and Slope Stability”, John Wiley & Sons Inc., Hoboken NJ, USA.

Dawson, E. M., Roth, W. H., and Drescher, A., 1998. “Slope stability analysis by strength reduction”, Geotechnique, Vol. 49, No. 6, pp.835-840.

Figure 1. General view of the stabilization measures constructed in 2009 (laying down of the slope inclination and rockfill wall reinforcement)

Figure 2. Slope failure registered in August 2013

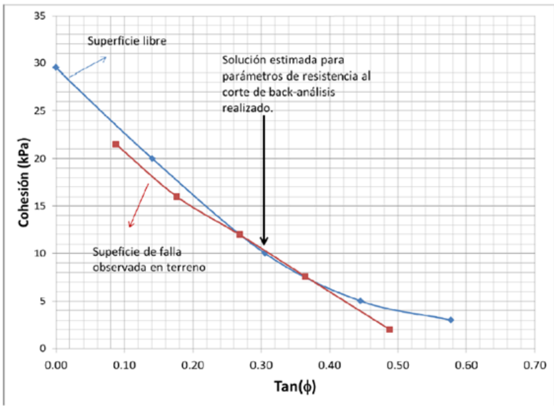

Figure 3. Sensitivity analysis cohesion/angle of friction to obtain FS=1

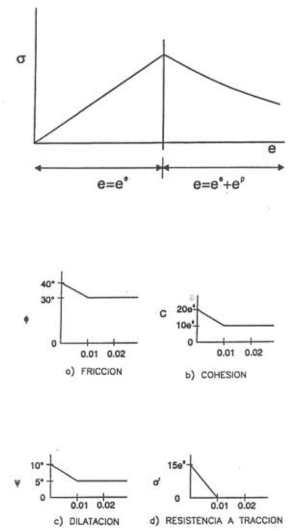

Figure 4. Strain-softening behaviour (Itasca, 2008)

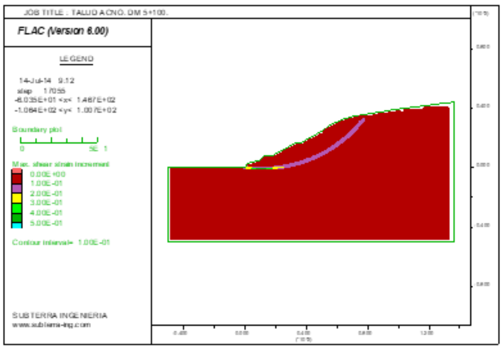

Figure 5. Back analysis. Shear strain distribution at the slope failure

Table I. Strength and deformability values calibrated in the back analysis.

Unit	Density (kN/m^3)	Cohesion (peak) (kPa)	Angle of friction (peak) (º)	Cohesion (residual) (kPa)	Angle of friction (peak) (º)	E (MPa)	ν
Coluvial clays	20	33	19	15	17	225	0,30
Subtratum	20	300	25	-	-	700	0,30
Rockfill	22	30	45			1.000	0,20

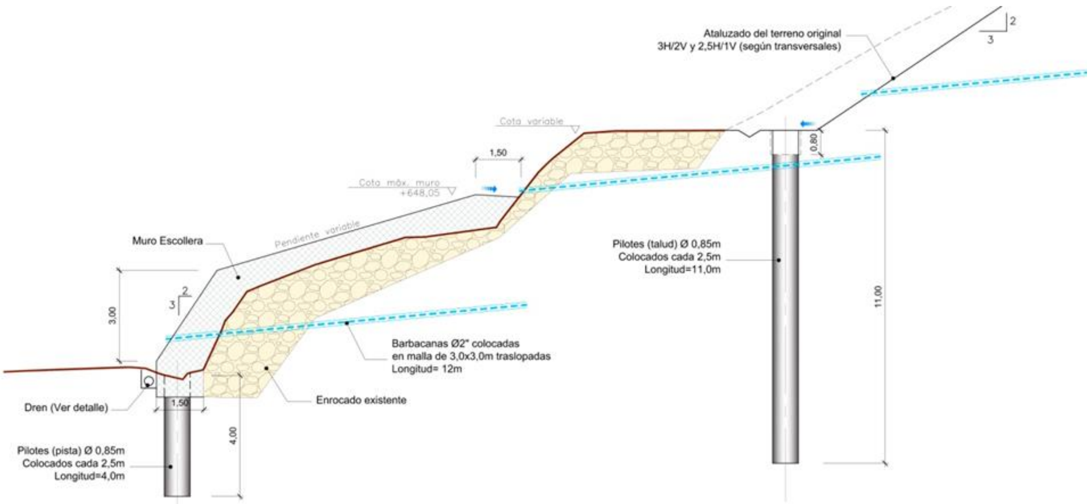

Figure 6. Slope reinforcement measures designed

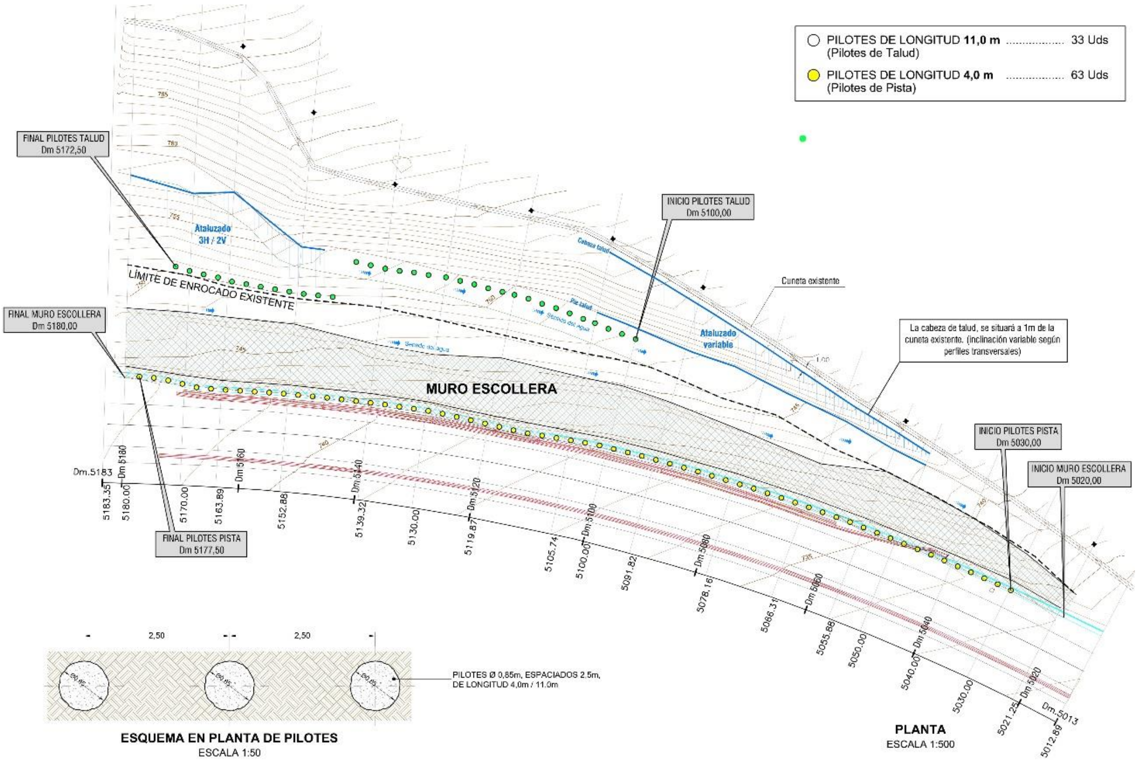

Figure 7. Plan view of the slope reinforcement measures

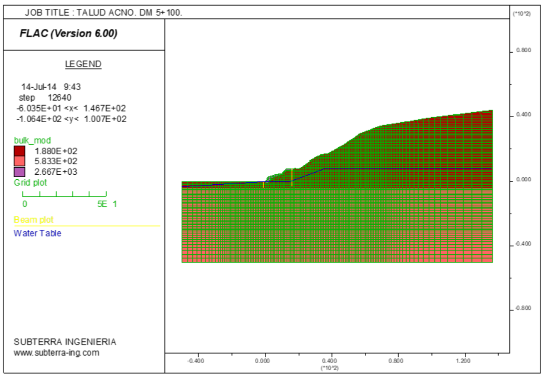

Figure 8. Slope reinforcement model

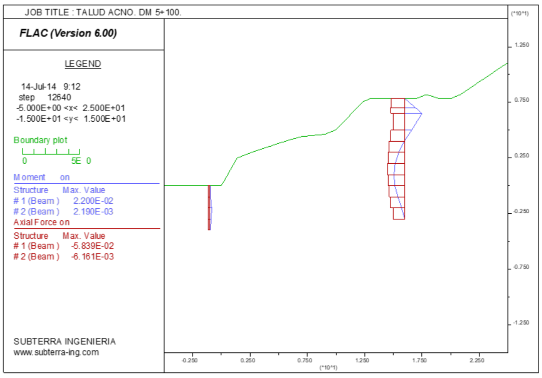

Figure 9. Moments and axial forces at the retaining wall

Figure 10. Execution of the toe retaining wall with reinforced piles

Figure 11. Execution of the concreted rockfill wall at the toe of the slope

Figure 12. Detail of the grouting of the rockfill

Figure 13. Final aspect of the motorway slope