Construction of a Large Cross Section Tunnel with Low Overburden and under the Water Table

A. Fernández¹, J. Rodríguez¹, B. Celada², J. Cuadrado²
¹ADIF, Madrid, Spain; ²Geocontrol S.A., Madrid, Spain

1 Introduction

The Strategic Plan of Infrastructures and Transport (PEIT) for Spain was developed in 2005 by the Ministry of Transport for planning infrastructure construction up to 2020.

Figure nº 1 shows the actions planned by PEIT related to the High-Speed Railway Lines, in which the connection between the cities of Orense and Santiago de Compostela is included; both cities belong to the Autonomous Region of Galicia, which is situated in NW Spain.

This work is managed by the “Administrador de Infraestructuras Ferroviarias” (ADIF), owned by the Ministry of Transport of Spain.

The construction of the new high-speed railway line between Orense and Santiago, planned for a speed of 350 Km/h, has been divided into two big stretches: Orense-Lalín and Lalín-Santiago.

The 40.2 Km stretch, Lalín-Santiago, is divided into five sections. The A Pena Tunnel, with a length of 810 m, is located within the Lalín-Baxán/Lalín-Anzo section.
2 Tunnel project

The A Pena Tunnel project was drawn up by the Spanish engineering APIA XXI and constructed by the company Ferrovial-Agroman; with the characteristics detailed in the next sections.

2.1 Cross section

The A Pena Tunnel was designed with single tube typology, including the two traffic lanes. In order to minimize the effects of overpressure, generated by the train circulation, the free section above the level of the railway was 110 m².

The cross section of the A Pena Tunnel can be observed in Figure nº2, which shows a total cross excavated section of 156 m².

![Figure nº2. Cross section of the A Pena’s Tunnel](image)

2.2 Ground characteristics

The ground within which the A Pena Tunnel was excavated is composed of schists and gneisses with values of RMR between 58 and less than 25 points. The overburden thickness above the vault ranges from 10 to 15 m; that is approximately the width of the tunnel to be constructed.

The water table is situated above the vault, between 10 and 14 m high; apart from a fault zone where the water table was situated under the tunnel floor.

Figure nº3 shows the longitudinal geological profile of the A Pena Tunnel.

2.3 Construction method

The construction of the A Pena Tunnel was planned with mechanical excavation, by means of backhoes dividing the section in three phases: top heading (67.5 m²), bench (73 m²) and invert (15.5 m²) as is shown in Figure nº2.
The design required the tunnel to be supported using rock bolts or steel arches and shotcrete in the normal stretches and using forepoling’s umbrella, steel arches, shotcrete and an elephant foot in weaker zones where RMR<30.

Figure 3.- Longitudinal geological profile of A Pena Tunnel.

3 Problems with the top heading construction

The top heading construction was stopped on 1st September 2008, when the excavation had advanced 183 m because the ground encountered consisted of a very weathered clays. The cause of this stoppage was the strong convergences and the movement of the tunnel vault that was being measured.

Figure nº4 shows the movements of the tunnel vault, measured by topographical levelling. It shows that after 35 days the movement was 95 mm and was still not established.

Figure nº4.- Vertical displacements in the vault at chainage 602,87.

While a solution was sought for continuing the works, the face of the heading was protected by shotcrete and self-boring rock bolts of 8 m long were placed in the tunnel while a temporary invert was constructed with shotcrete.
Photograph nº1 shows the tunnel face protected by shotcrete and the self-boring bolts placed in the tunnel side walls.

I.- Tunnel face protected by shotcrete.  
II.- Self-boring bolts placed in the side walls.

Photograph nº1.- Tunnel face protected by shotcrete and self-boring rock bolts in the side walls.

4 Planned solution for existing problems

The first step in order to solve the problem was to define a conceptual solution, by a back-analysis, to find the representative rock mass properties of the A Pena Tunnel. Afterwards, the calculations to define the constructive solution were made.

4.1 Conceptual solution

As the measurements clearly indicated that the tunnel support was sunk on the ground, the suitable solution appeared to be to reinforce the support with micropiles which should be drilled through the elephant foot and joined to them with epoxy resin and longitudinal beams made of steel reinforced shotcrete.

The conceptual design for the suggested solution is shown in Figure 5; this could be combined with self-boring bolts already being placed or with a temporary invert.
4.2 Determination of rock mass properties

The ground in which A Pena Tunnel was stopped is constituted of silty clay, produced by the surrounding weathered schists.

The ground to be excavated is under the water table and, due to this, it can be considered totally saturated. In these conditions, from a conservative point of view, it seems reasonable to make the calculations working with total stresses; as, accepting a Mohr-Coulomb failure criteria, the resistant parameters are reduced to undrained cohesion, $C_u$, because the friction angle is $\phi = 0$ under these conditions.

According to the data from the project and the in situ experiences, the initial properties assigned to the clay were:

- Bulk density: $\gamma_{ap} = 2.2 \frac{t}{m^3}$
- Deformation module: $E = 15.000 \frac{t}{m^2}$
- Poisson coefficient: $\nu = 0.49$
- Undrained cohesion: $C_u = 20 \frac{t}{m^2}$

The initial stress state in saturated grounds of clays has to be close to the hydrostatic model and consequently in this case has been adopted a value of $K_0 = 1$.

With this initial data the calculations were made, with the model represented in Figure nº6, to obtain a safety factor of $FS = 1$ in the top heading construction.

The process to calculate the safety factor consists of decreasing the initial undrained cohesion, $C_u = 20 \frac{t}{m^2}$, until model instability is produced. Figure nº7 represents the displacement velocities distribution of the last stable calculus, corresponding to $C_u = 13.79 \frac{t}{m^2}$ and the first one unstable; that matches with a $C_u = 13.10 \frac{t}{m^2}$. According to this data, it was considered that the undrained cohesion is $C_u = 13.79 \frac{t}{m^2}$, instead of the initial value of 20 $\frac{t}{m^2}$. 

Figure nº6.- Details of mesh and structural elements of the used model in the back-analysis.

The initial stress state in saturated grounds of clays has to be close to the hydrostatic model and consequently in this case has been adopted a value of $K_0 = 1$.
I. - For the last stable calculus
\( (C_u = 13,79 \, \text{t/m}^2) \)

II. - For the first unstable calculus
\( (C_u = 13,10 \, \text{t/m}^2) \)

Figure nº7.- Displacement velocities distribution of the model in last stable case (I) and in first stable case (II).

4.3 Comparison of several reinforcement solutions

Once the undrained cohesion value was adjusted, it was determine the safety factors of three possible reinforcement solutions with the following results:

1. Self-boring bolts and a concrete temporary floor FS = 1,17
2. Micropiles L = 12 m. FS = 1,10
3. Micropiles and a concrete temporary floor: 1,34

From these results the solution based on micropiles and a concrete temporary floor was selected.

4.4 Determination of the length without support to built the bench and the invert

Once the solution for the reinforcing of the top heading construction is decided, it was possible to calculate the maximum length without support that is admissible during bench and invert excavation.

For that, using the model presented in Figure nº8, safety factor was calculated for excavation lengths without support of 4, 8 and 10 m, obtaining safety factor values of 1,31; 1,27 and 1,24 respectively.
5 Tunnel construction completion

After the calculations were made to define the reinforcement of the top heading construction, the works started on 16-10-2008 and finished on 2-4-2009.

The bench and invert construction started on 4-11-2008, progressing in stretches of 10 m, and finished on 13-4-2009.

Photograph nº2 shows the bench face excavation while Photograph nº3 presents the layout of the support in one bench’s side wall. Finally, Photograph nº4, presents a general view of the problematic stretch, already built and waiting for the definitive lining.

Photograph nº2.- Bench face excavation.

Photograph nº3.- Support of the one side wall bench.
6 Conclusions

The situation created in the central part of A Pena Tunnel, excavated under the water table, weathered rocks with RMR < 20 and overburden thickness similar to the excavation diameter, can be considered as making extremely difficult to build.

To define the appropriate solution a back-analysis has been made, working in total stresses, that has permitted the definition of the undrained cohesion value of $C_u = 13.79$ t/m².

A study of the three possible reinforcement solutions has been carried out and the best was selected consisting of placing 12 m long micropiles and a temporary invert, during top heading construction.

The completion of the tunnel has been carried out by excavating the bench and invert with 10 m long advances, according to calculus results.

The application of the reinforcement solutions proposed has permitted the completion of the construction of the tunnel without any significant problem.