Proceedings of the 17th Pan-American Conference on Soil Mechanics and Geotechnical Engineering (XVII PCSMGE), and 2nd Latin-American Regional Conference of the International Association for Engineering Geology and the Environment (IAEG), La Serena Chile, 2024.

Interaction between nearly tunnels. The RP82 road tunnel, Mendoza, Argentina

Interacción entre túneles cercanos. El túnel carretero en RP82, Mendoza, Argentina

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ABSTRACT: The final geometry of a tunnel, is influenced for parameters of the geo-materials, its intended use, and the constructive method sequence that can be applied. If there is another tunnel in the vicinity it will introduce a greater complexity for the design and construction. The case study is a road tunnel located near Potrerillos site (Mendoza, Argentina). The tunnel design and construction have two conditions that make it unique, a lower tunnel, corresponding to an aqueduct, and the presence of the CFRD dam. The lower tunnel is more than 15 years old and it crosses 9.00 meters below the new road tunnel. The excavation method was drilling and blasting in granitic mass. The design took into account a complementary reinforcement system to the lower tunnel, for attending the blasting vibrations and the decompression of the rock mass, that could cause falling blocks. A previous failure experience in a similar tunnel near the project introduced another sensitivity factor in order to make decisions for the new design. The failed tunnel shows fall of blocks, paralyzing its construction.

KEYWORDS: road tunnel, nearby tunnel, blasting vibration, rock fall, seismicity

1 INTRODUCTION.

The design of a road tunnel implies the definition of a transverse geometry that is suitable for its operation. The type of tunnel influences not only the geometry of the cross section and the longitudinal profile, but also the support and construction system used. The existence of sensitive structures near the tunnel is an additional factor to be taken into account in the definition of the construction system.

This paper shows the study carried out for a road tunnel, recently built in the Province of Mendoza, Argentina. It has a length of just over 370 meters. Its cross section has a height of 8.02 meters, with a maximum span between sidewalls of 11.51 meters. Figure 1 shows a diagram of the cross section of the tunnel.

The tunnel is located in a geological environment known as "Stock of Cachueta". The affected rock mass is a plutonic igneous formation. The geological structure was studied in detail during the construction of Potrerillos dam (CFRD type), located 400 meters from the axes of the project.

The formation is a small batholith (stock) that has emerged by denudation of the rock cover and the tectonic action. In the geological process of its evolution, the rock mass received bending efforts that failed to deform it, consequently, the upper part of the rock mass is fractured vertically and subvertically. This caused the rock to present an intense fragmentation, with a set of discontinuities. After the constitution of the original rock mass, the site received important tectonic actions.

Along with the complexity of the tunnel location site, two important elements have affected the design. On the one hand, the presence of a lower tunnel, corresponding to an aqueduct that serves for the provision of water to a city of more than one million people that cannot be interrupted. On the other hand, the presence of a dam of the CFRD type, recently built, which conditions the layout of the new tunnel.

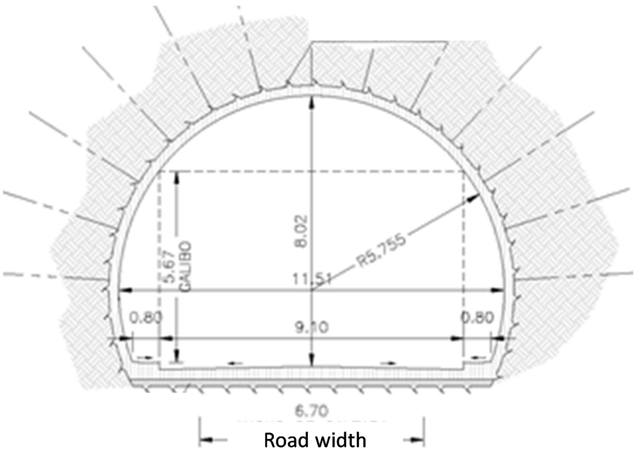

Figure 1. Transverse profile of the tunnel.

The aqueduct is a metallic conduit that is inside a tunnel several decades old. This tunnel crosses below the road tunnel, at a minimum distance of 9.00 meters, measured between the base of the road tunnel and the roof of the aqueduct tunnel. The main limitations of the design, in relation to this element, were based on the definition of a reinforcement system complementary to the one currently used by the aqueduct. A protection device for the aqueduct's metal conduit was designed to protect it against possible block falls due to blasting during the construction of the road tunnel.

Along with the previous conditions, there was complementary information corresponding to a previous experience in a similar tunnel near the project. The history of its excavation showed that there were falls of blocks from the roof, which determined the collapse of a pilot tunnel..

In the initial stages of the studies of the additional support on the aqueduct tunnel, blasting tests were carried out, to evaluate damages in both nearby structures. It has been studied the effect of wave propagation and the potential drop of blocks in the aqueduct. The results were used for the design of the advance system by sequential excavation and also were used to measure the effect of the construction on the nearby dam.

The analysis for the initial definition of the excavation supports has been carried out according to the Norwegian support method (based on the Q parameter), the Bieniawsky models (corresponding to the RMR and the Elastic Behavior Index, ICE). These parameters allowed to establish, in a preliminary way, the characteristics of the support and the beneficial effect of the excavation of the pilot tunnel prior to the excavation of the entire section.

An additional element that has influenced the definition of the construction process, and in the design of the lining, was the presence of a Concrete Face Rockfill Dam (CFRD) type dam. The dam has a height greater than 100 meters, and is located at a distance of 400 meters with respect to the axis of the tunnel.

Both, dam and road tunnel, are within an area with significant seismic activity. The studies carried out have concluded with the recommendation to use as design earthquake (SEE) with an acceleration equal to 0.36 g, and as operation earthquake (OBE) an acceleration equal to 0.14g.

Finally, the main components of the established design are presented. These elements were built without significant changes during the development of the work, allowing its habilitation in 2018.

2 SITE CHARACTERIZATION

The granite that forms the stock presents textures with different grain sizes. For this reason it is divided into two groups, granite of fine grain and granite of coarser grain. There are two similar materials with different geo-mechanical properties. The resistance and the deformability are the variables that show the greatest difference.

The granitic mass is holocrystalline, phanerite of medium grain and reddish color. Mineralogically it is composed of Cz, Kfs, Pl (majority) and Bt, Hbl (minorities or accessories). The strength of the intact rock is very variable, with values of simple compressive strength between 30 MPa and greater than 100 MPa.

There are discontinuities, with a genetic origin, resulting from cooling and recrystallization. There are also others, of orogenic origin, generated by the location of the batholith. The first are erratic, the second are oriented according to tectonic efforts.

The sector is fractured, with faults of great intensity. The fracturing is greater on the surface, where there is less confinement. This generates a greater opening of the fissures and decreases the effect of interrelation between the blocks. This type of structure presents complex stability conditions for carrying out excavations.

In addition, the immediate antecedent showed that a similar tunnel, near this project, was abandoned after the fall of blocks from the crown. Within the process of characterization of this structure, Flores (2013) studied the ground in detail. The axis of the road tunnel is slightly displaced from this old excavation, however; the observations made were considered indicative of the rock mass that would be found when excavating the road tunnel.

The gallery, given the structural complexity of the massif, allowed obtaining a representative image of the geotechnical site. The degree of fracturing of the rock mass and the influence of the structural control on the excavations were clearly appreciated. A rockslide has been detected in the entrance portal. This information was considered very useful, in comparison with that which can be obtained from mechanical surveys or other indirect prospecting methods.. This sector could not be surveyed in detail due to the precariousness of the area.

All sets of discontinuities corresponding to the different sections of the tunnel have been plotted, indicating the poles (or normal positions to the planes of the discontinuities) in stereographic projection. In all cases, the lower hemisphere has been graphed. The results are shown in Figure 2.a. The statistical analysis of the poles is represented by contour lines, as shown in Figure 2.b. The main joint sets are 189/10 (6.8%); 282/41 (4.8%); 252/26 (4.4%); 6/17 (3%) and 3/57 (2.7%). These systems are applied on the axis of the trace, which has a strike of the order of 150°.

As previously indicated, the projected main tunnel is located near an aqueduct. The aqueduct tunnel is located below the projected highway tunnel, with an orientation of 105 °. The position of both tunnels is shown in Figure 3. As indicated above, together with the complexity of the environment in which the tunnel was installed, its design has been influenced by the presence of a tunnel below the axis of the road tunnel.

During the design of the construction system of the road tunnel, they were studied the actions required for the improvement of the support in the aqueduct tunnel and the design of conduit protection devices against possible falls of blocks. The surveys carried out inside the tunnel of the aqueduct have been also used for the evaluation of mechanical properties of the rock mass. These variables were later extrapolated to the different sectors of the road tunnel.

The geostructural survey carried out in the aqueduct tunnel has included a classification of the massif according to the Q and the RMR parameters. These are part of the Norwegian Tunneling Method (NMT, Barton et al 1974) and Bieniawsky methods of rock classification, respectively.

These methods are widely used in the characterization and classification of rock masses for the pre-dimensioning of the support system. They have the characteristics of an "expert system". The Norwegian method summarizes the experience in the excavation of a large number of tunnels (of the order of 1,000).

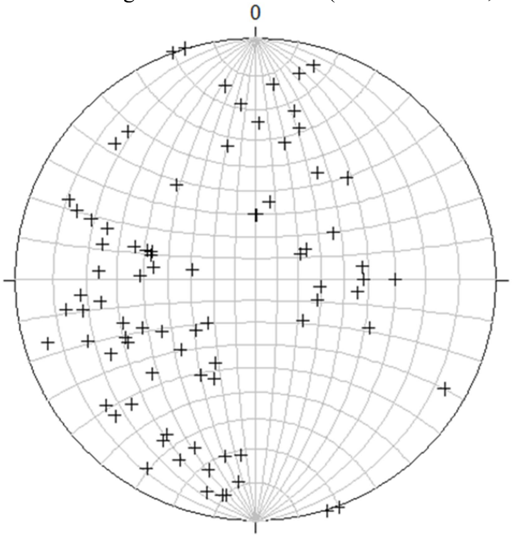

Figure 2.a. Stereographic projection of poles of the families of diaclases surveyed every 10 meters. Lower hemisphere projection. Source: Flores (2013).

Figure 2.b.Statistical analysis of poles of the diaclases families of Figure 2.a. Lower hemisphere projection. Source: Flores (2013)

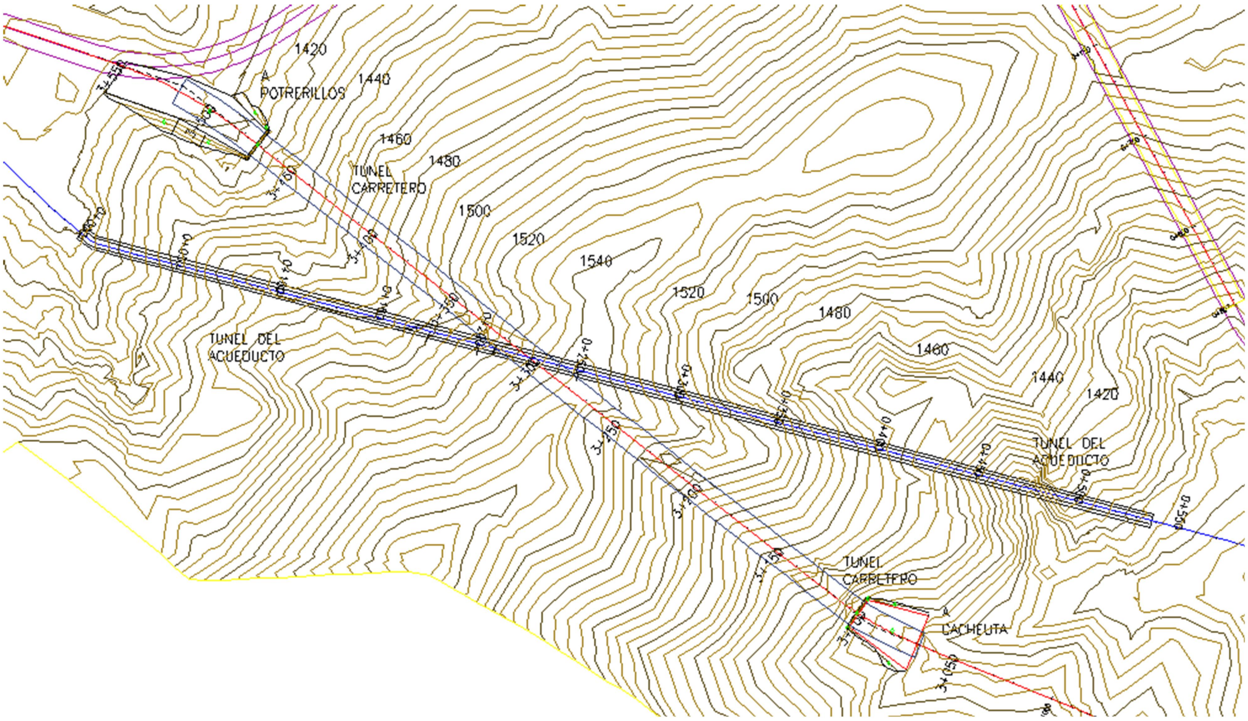

Figure 3. Location of the road tunnel (*Tunel Carretero*) and the aqueduct tunnel (*Tunel del Acueducto*).

The method that qualifies the massif according to the Rock Mass Rating parameter), uses another database of constructed tunnels. (RMR, Bieniawski 1973 and ICE, Celada et al, 2014) Both systems have correlation equations with each other. The values obtained are between 0.14 and 1.22, for Q and between 37 to 52, for the RMR.

The blast effect of the propagation in the rock mass of the mechanical waves was analyzed. Its incidence on the aqueduct tunnel and on the nearby dam was evaluated. For this, blasting tests were carried out in the initial stages of the study. The results have allowed for establishing laws of attenuation in the site, to evaluate the potentiality of fall of blocks in the aqueduct and the possible incidence of these actions on the near dam. These results have determined the main conditioning elements of the construction process, the execution sequence of the blasting for the advance in the excavation of the tunnel.

3 METHODOLOGY

3.1. Preliminary Study

The preliminary analyzes have been carried out based on the NMT, RMR and ICE premises.

The NMT system uses as essential support elements, the shotcrete with added fibers (Sfr), and the passive bolts (B). Additionally, for very weathered or fragmented rocks, use concrete with steel ribs (RRS) and cast concrete in situ (CCA). The rock mass in which the tunnel is located corresponds to the classification 5, 6 and 7 of that method. According to this qualification, the use of shotcrete with fiber aggregate is recommended. The thickness of the shotcrete varies between 6 to 14 cm. In addition to it must include a support using passive bolts of 3 meters in length.

The classification method based on the RMR parameter has also been used. The values obtained correspond to group III (regular rock) and IV (poor rock). In the regular rock, a support is recommended formed by passive anchors of 4 meters in length, with regular spacing. In addition, a steel mesh should be placed on the crown. Then, shotcrete should be placed with a variable thickness between 5 and 10 cm in the crown and 3 cm in the sidewalls. In poor rocks, the anchors have a length of between 4 and 5 meters, the shotcrete has a thickness of 10 to 15 cm in the crown and 10 cm in the side walls. If needed, complete with light trusses, spaced 1.5 meters.

Finally, the ICE was defined according to the following equations:

If Ko < 1 $$ICE = \frac{3.704\,\sigma_{ci} e^{\frac{RMR-100}{24}}}{(3-k_o)\,H} F \qquad (1)$$

If Ko > 1 $$ICE = \frac{3.704\,\sigma_{ci} e^{\frac{RMR-1}{24}}}{(3k_o-1)\,H} F \qquad (2)$$

Where: σ_{ci} = uniaxial compressive strength of intact rock (MPa); Ko = ratio of the horizontal to vertical virgin stress; H = tunnel depth (m) and F = shape coefficient, see Table 1.

Table 1. Parameter F

Underground excavation	F
Circular tunnel, ϕ = 6 m	1.30
Circular tunnel, ϕ = 10 m	1.00
Conventional tunnel, 14 m wide	0.75
Caverns 25 m wide x 60 m high	0.55

ICE makes it possible to predict the stress-strain behavior of the faces of the tunnels classifying them into five categories as show in Table 2.

According to the above, only the excavation faces in which ICE < 70 will present substantial deformation for significant variations in the determination of RMR.

Table 2. Stress deformation behavior vs ICE

ICE	Stress deformation behavior
>130	Completely elastic
70-130	Elastic with incipient yielding
40-70	Moderate yielding
15-40	Intensive yielding
<15	Mostly yielding

The calculations carried out have been based on considering a value of F=1.00 for the complete section excavation and 1.70 for the case of the pilot tunnel. The F value indicated for the pilot tunnel is not within the range indicated by the authors of the ICE, consequently, it has been defined by extrapolating values according to the section of the pilot tunnel. The analysis results for both cases are shown in Figure 4.

According to this analysis it can be observed:

- The initial construction of the pilot tunnel allows almost the entire tunnel to be traversed with ICE values greater than 70.
- Sectors with ICE greater than 130 require supports with passive bolts and 5 cm of shotcrete. Sectors with ICE values between 70 and 130 require a higher density of bolts and 10 cm of shotcrete.

These results show the feasibility and convenience of executing the pilot tunnel throughout the tunnel development, allowing for simple initial support. The construction of the pilot tunnel makes it possible to verify the mechanical properties defined in the geotechnical campaign.

3.2. Detail Study

Based on the premises indicated above, the analysis of the local and general stability of the tunnel has been studied in accordance with the following composition elements of the support:

- Excavation support. These are elements that can be placed quickly once the tunnel has been completely or partially excavated. This set consists of passive anchors with cement grout, complemented by an initial layer of shotcrete, with a variable thickness between 5 and 7 cm. Where the rock is deteriorated or with a high degree of weathering, and the support elements are considered insufficient, metal trusses or spilling type anchors are used.
- Final lining. They are structural components of the tunnel, which has a useful life equivalent to that of the tunnel. Although the suppliers of the passive anchors used indicate that these parts are efficient in the long term, it has been considered that they do not participate in the support. The lining has been formed with shotcrete placed in several layers, until reaching a minimum thickness of 0.30 meters.

For the modeling of stability, two forms of representation of the rock formation have been considered. For the first analysis, it has been considered that the tunnel crosses a discrete system of blocks, defined by the families of discontinuities observed in the aqueduct.

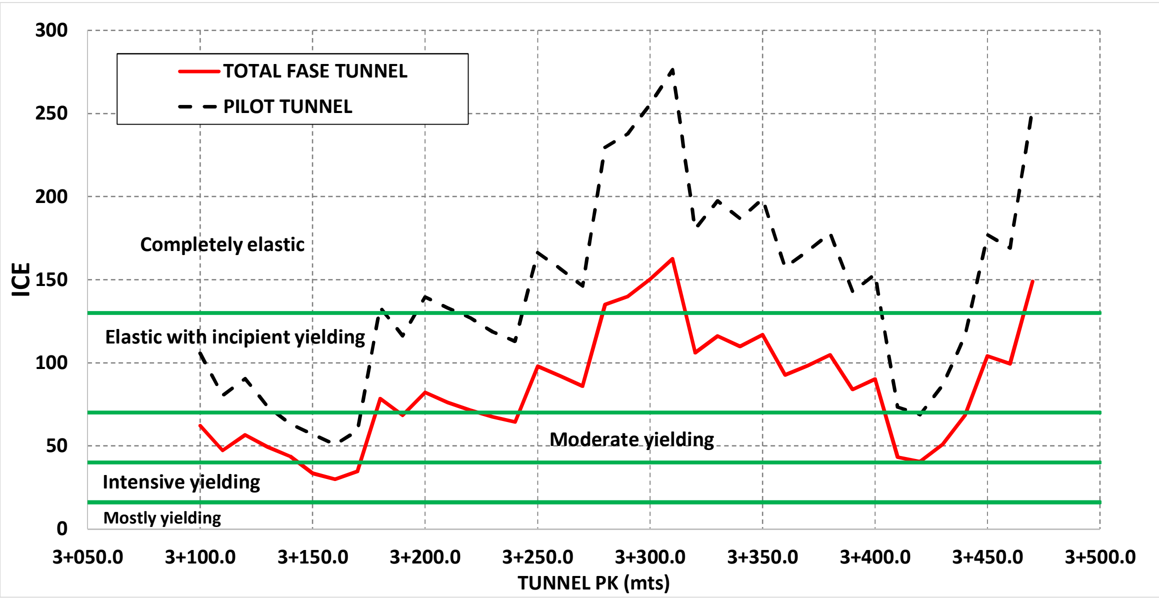

Figure 4. Model of ICE in road tunnel.

The stability conditions of wedges have been evaluated, considering the contribution of shear resistance generated by the shotcrete, together with passive anchoring systems. In a conservative way, a calculation value of 50 tn/m^2 (500 kPa) has been estimated. In the second analysis, it is considered that the tunnel is built in the continuous medium. The resistant and deformational parameters are established according to the classification of the RMR and its extension to the GSI system (Hoek and Brown, 1980).

As an example of the modeling carried out with wedge systems, Figures 5 shows some of the sections studied. The road tunnel has been fragmented into sectors of 25 meters long. For each of them, the stability condition was evaluated according to the type of wedge that could be formed. This has allowed fixing the requirements of passive bolts and shotcrete in the stage of support of the tunnel.

The results obtained allow us to formulate the following observations:

- The proposed support system allows to achieve adequate safety conditions (under static stresses), in most of the extension of the tunnel. In the sectors in which the support does not achieve the admissible safety factors, a reinforcement was planned using metal trusses and crown protection systems of the spilling type.
- A lining formed by a continuous and regular layer of shotcrete with a thickness of not less than 0.30 meters is satisfactory in the long-term behavior. The combined action of the proposed support and revetment systems results in even better safety conditions than those obtained for the case of the coating exclusively conformed by the shotcrete.

For the analysis of the rock mass as a continuous medium, a finite element model was developed. This model has been used to calculate the variation of stresses, induced by excavation. Likewise, it has been used to define the efforts on the support elements in the different stages of construction.

As an example of the results obtained in the modeling, Figures 6 are presented. These show the main vertical and horizontal stresses, in the initial stage of the excavation (with a pilot gallery of recognition and research) and with the entire section excavated.

The results obtained allow us to formulate the following observations:

- The tensional behavior of the rock mass shows conventional characteristics for these types of excavations. In the crowning and the floor of the tunnel, reductions in the pressures of the rock mass were observed, without these generating a traction state of importance for the development of the construction.
- At the level of the vertical pressures, in the sidewalls, an increase with respect to the initial pressure of the massif was observed. These increases determine that the rock mass reaches vertical pressures of the order of twice the original. The values obtained are lower as those obtained as simple compressive strength of the massive rock. These results have ruled out the possibility of rock stress instabilities.

As indicated before, the incidence of the construction of this tunnel on the aqueduct located below was analyzed. For the construction of the road tunnel, the drilling and blasting system was used. Consequently, it is interesting to evaluate how the blasting influences the stability of the aqueduct. Blasting for excavation of the tunnel generates mechanical waves from the point of explosion. These waves propagate through the rocky mass, reducing their energy as they move away from the point of origin. The energy that reaches the axis of the aqueduct was studied, measured in variables such as speed of vibration or accelerations (Langefors and Kihlstrom, 1973). Studies show that the effects attenuate sensibly with distance.

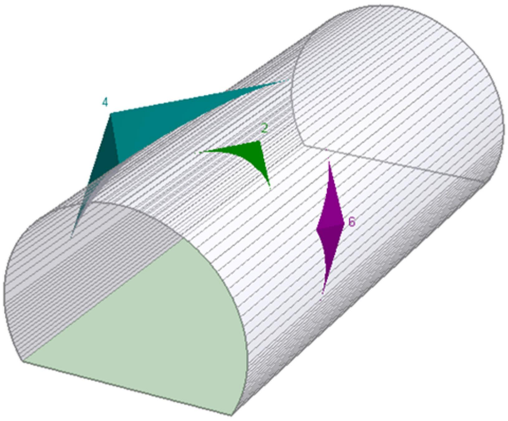

Figure 5.a. Modeling between 3+210 to 3+230 without protections

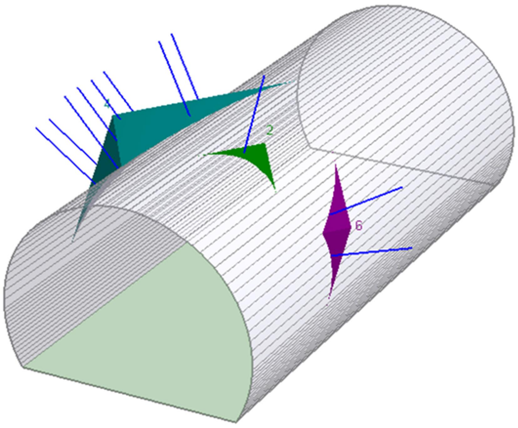

Figure 5.b. Modeling between 3+210 to 3+230 with protections

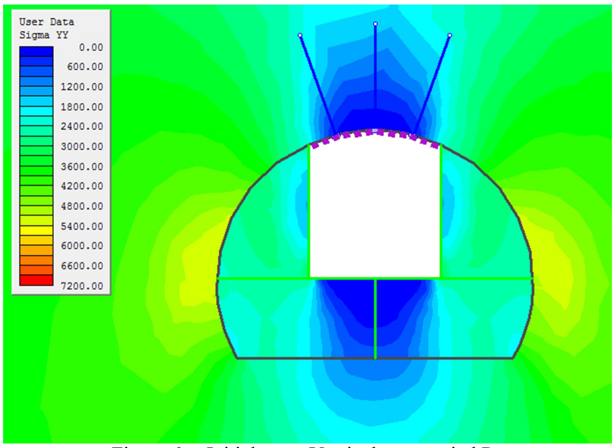

Figure 6.a. Initial state. Vertical stresses in kPa

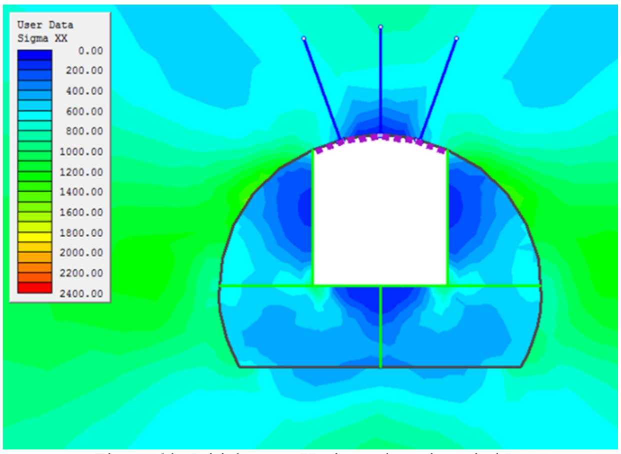

Figure 6.b. Initial state - Horizontal tensions, in kPa

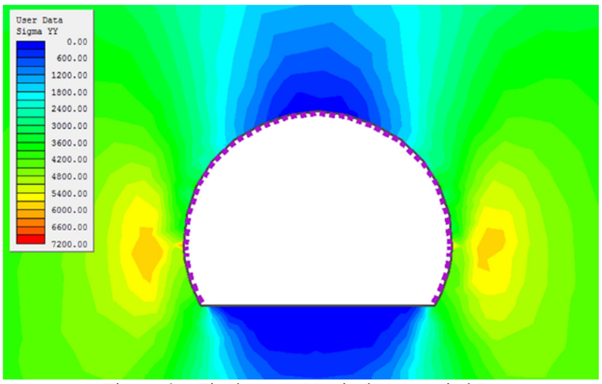

Figure 6.c. Final State - Vertical stresses in kPa

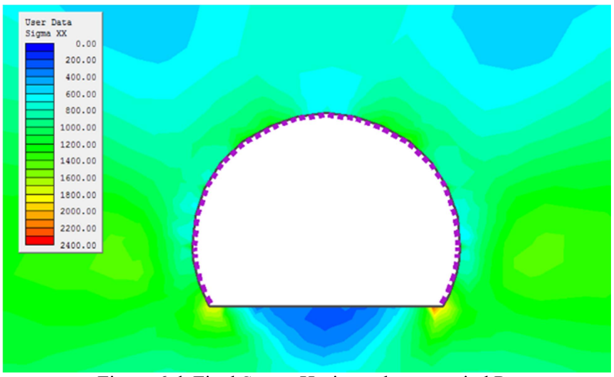

Figure 6.d. Final State - Horizontal stresses in kPa

According to the aforementioned, the maximum incidence of blasting occurs in the area where the tunnel axes intersect. The variables considered in this problem are: the energy released by the explosion (E), the distance between the emission and reception point (R), the density of the rock (ρ), the longitudinal propagation velocity in the rocky medium (V) and the circulation time of the wave (t).

In the practical, the maximum vibration speed of the particles (V) is used as an indicator for monitoring. This value must be lower than admissible values in order to generate tolerable blasting by sensitive structures close to the emission site. The relationship between velocity and released energy can be expressed as a function of the distance between the point of explosion and the point of detection (R), and the amount of charge used (W), of the form:

$$V = b(R/W^{1/3})^{-a} \qquad (1)$$

Where a and b, are coefficients of calibration of the relation. They vary according to the "relative distance ($R/W^{1/3}$)" with limit change on the value 4 (Langefors and Kihlstron, 1973). For values greater than 4; b = 2100 mm and a = -1.60, while for values less than 4; b = 11700 mm and a = -2.80. A summary of variants of this expression can be found in Guraudi et al (2009).

The damage generated by explosions has been studied using the speed vibration of the particles as a reference variable (Ambraseys and Hendron, 1968). A commonly accepted criterion is that vibration speeds less than 63 mm/sec, do not cause damage to buildings. The statistical analysis indicates that the vibration speed of 193 mm/s is the limit for not causing significant damage to concrete structures. For tunnels in rock, a vibration speed of 300 mm/s has been adopted as the limit for the start of the fall of blocks in unprotected tunnels and a speed of 610 mm/s for the formation of new cracks in massive rocks. As an additional reference, the values indicated in Table 3 have been used.

Table 3. Summary of expected types of damage due to blasting, in relation to the speed of particle vibration. Source: Cornejo Alvarez, 2003.

Speed of the particle (mm/seg)	Damage
50	No fissure
70	No apparent fissure
100	Insignificant fissures
150	Fissures
225	Large fissures
300	Falls of blocks in galleries
400	New fissures in rocks.
1400	Fissures in the tunnel lining

The velocity values calculated for the blasting load applied and the distance between the point of emission and the point of reception have been transformed into accelerations, according to the expression:

$$V = a_1\,Ac^2 + a_2\,Ac, \qquad \text{with V in cm/seg} \qquad (2)$$

The correlation factors used correspond to Abrahamsom et al (2014), Campbell and Bozorgnia (2013) and Chiou and Youngs (2013).

With the results obtained in the previous calculation, it has been evaluated the stability against falling blocks of the characteristic profiles of the aqueduct tunnel. The critical dynamic loads (accelerations) were calculated for the stability of the section. To minimize potential actions on the aqueduct tunnel, the maximum blasting load applicable in each sector of the road tunnel was established. These analyzes are complemented with the fixing of tasks of improvement of the support in the aqueduct tunnel, generating a solution indicated in Figure 7.

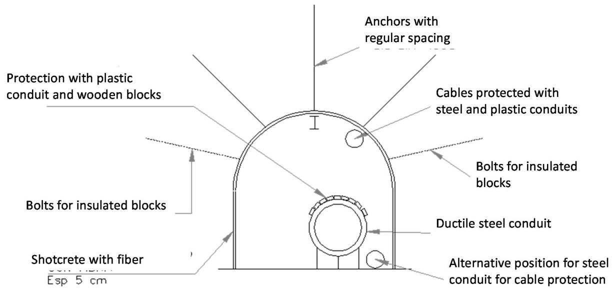

Figure 7. Aqueduct tunnel support.

4 RESULTS

The constructive system applied is the sequential excavation, whose typical cycle has the following components: a. topographic mark of the drilling site, b. drilling, loading and blasting, c. ventilation, d, manual cleaning and removal of material, e. placement of shotcrete with fiber, 7 cm, f. topographic marking and perforation of the anchors and g. restart of the sequence.

The component phases of the advance are the following:

- Execution of pilot gallery. The entire length of the pilot was excavated and supported. The execution of this part of the complete cross section allows fulfilling two objectives. On the one hand, establish through a tunnel of smaller section, and with greater capacity of self-support, an operative communication between both ends of the tunnel. On the other hand, it allowed direct observation of the affected rock, which was used for the validation or rectification of the calculation hypothesis used and the adequacy of the support forecasts.
- Excavation of the sides of the gallery. The work was done advancing from each portal. The left side is excavated and, after advancing three modules in a longitudinal direction, excavation of the right side begins.
- Excavation of the lower level. The left sector of the bank was excavated and after three sections, you start with the right sector.

In this way, the sequence of generation of the cross section is indicated in Figure 8.

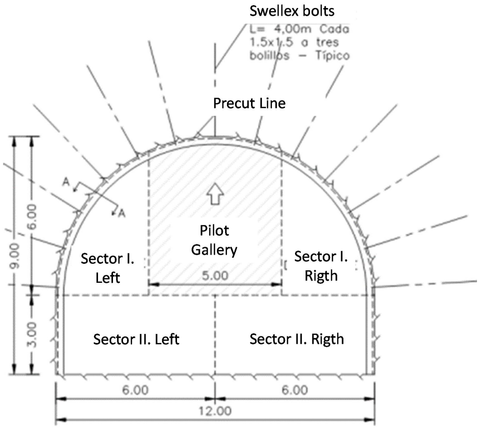

Figure 8. Excavation and support of the road tunnel.

5 CONCLUSIONS

As a conclusion of the studies carried out, the characteristics of the support, the lining and the construction sequence of the road tunnel were defined. The construction sequence of the tunnel was based on the partial excavation of the section. An upper gallery was executed along the tunnel, and then the excavation of the rest of the sections of the transversal profile continued.

In relation to the studies carried out for the evaluation of the interaction between both tunnels, the following concluding observations can be made:

- The critical conditions, derived from the geotechnical geological surveys along the tunnel, showed that the most sensitive vibration actions were those that are located in the sector between the 160 meters and 260 meters from the portal. In this sector, the critical accelerations, obtained to achieve a safety factor equal to one, and derived from the calculation of the vibration speeds, indicate values significantly lower than those used for the rest of the tunnel.
- The reinforcements applied produce a general effect of improvement of the safety factors against the fall of blocks.
- If a permissible vibration speed of 200 mm / sec is considered, the effect achieved by the reinforcements is that the sector of the superposition of both tunnels will reach the vibration speed limit established. This result justifies the application of the protection to make uniform vibration admissible for the entire tunnel. These works allowed an estimate of the disposition of the explosive load according to the progress of the tunnel. The applicable load diagram is as shown in Figure 9.
- The progress of the construction of the road produces a change in the tensional state that shows the tunnel of the aqueduct. Since the overburden cover is relatively modest, the variation due to the blasting effect has been considered significant. If the maximum tension of the wave is much higher than the tensile strength of the rock mass, blocks can fall into the aqueduct tunnel. As the tensile strength of granite rock is very high, the blocks that move will be the existing ones defined by the discontinuities, and it is unlikely that new fractures will be generated.

Under the previous premises, the tunnel has been built presenting general behavior within the parameters foreseen in the design. Figures 10 show images of the tunnel.

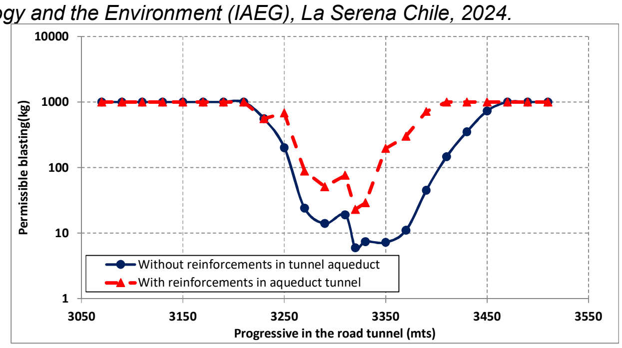

Figure 9. Maximum applicable blast loads per phase, in the road tunnel.

Figure 10.a. View of the tunnel in the construction stage of the pilot tunnel

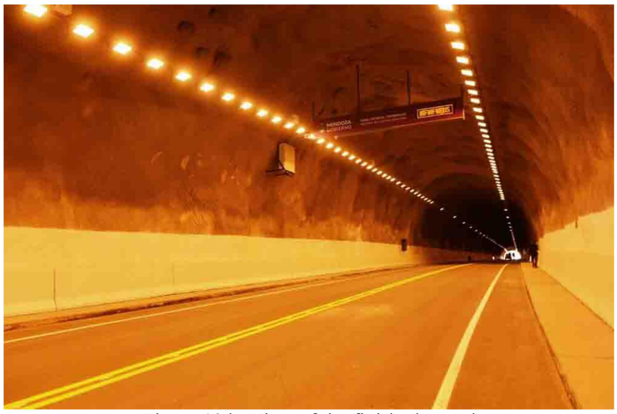

Figure 10.b. View of the finished tunnel

ACKNOWLEDGMENTS
The authors wish to express their sincere gratitude to the authorities of the Provincial Department of Roads of Mendoza, as well as to the technical staff of the Construction Companies, J.J. Cartellone and Rovella, for their support in the development of these studies.

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The paper was published in the proceedings of the 17th Pan-American Conference on Soil Mechanics and Geotechnical Engineering (XVII PCSMGE) and was edited by Gonzalo Montalva, Daniel Pollak, Claudio Roman and Luis Valenzuela. The conference was held from November 12th to November 16th 2024 in Chile.