Hydropower scheme Los Cóndores. Construction strategy. Lining design and 3D numerical analysis of special sections

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Abstract

The Hydroelectric Power Project Los Cóndores, in the upper part of the Maule Valley, in Chile, includes the excavation of several tunnels with a total length in excess of 15 km. The adduction tunnel from the Maule dam to the pressure shaft, 12 km long, is excavated through very heterogeneous volcanic materials with up to 530 m overburden and high piezometric levels.

Excavation from the intake structure and 2 additional excavation faces from an intermediate adit ("ventana Lo Aguirre") are foreseen, with sections excavated in drill and blast and others driven with a double shielded TBM.

Description of the construction procedure, involving the launching of the TBM from a chamber excavated in the adit, "downwards" to the pressure shaft, partial dismounting in another chamber when arriving to the later, and repetition of the process "upward" until connecting with the face front coming from the intake structure.

Design of the lining with precast concrete segments in sections driven with TBM and 3D analysis of special sections are explained.

Key words

Drill and blast, TBM, volcanic materials, segmental lining, 3D analysis

1 Description of the project

The hydroelectric power project of Los Cóndores, in the upper part of the Maule Valley, in Chile, is currently under construction by Ferrovial Agroman for ENEL. The project aims to generate 156 MW with an annual average production of 620 GWh. The total head drop is 765 m with a flow of 25 m^3/seg and two Pelton turbines.

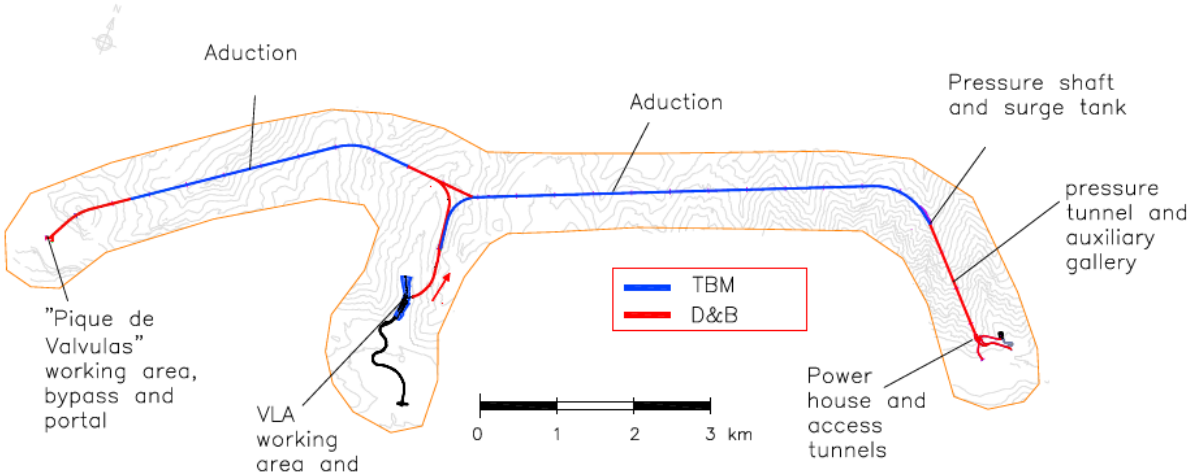

Fig. 1 Los Cóndores Project lay out

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As main civil works, the project includes the following:

- An adduction tunnel, from the Maule dam to the pressure shaft, 12 km long, to be excavated through very heterogeneous volcanic materials, partly driven with a double shielded TBM (circular section with 3.7 m and 4.56 m inner and excavation diameters) and partly excavated with drill and blast (horse shoe shape section 5.5 x 5.5 m).

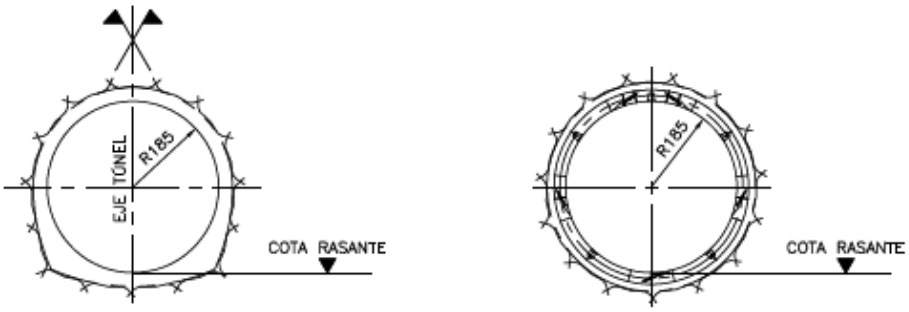

Fig. 2 Drill&Blast and TBM aduction tunnel sections

- 470 m vertical shaft with its head about 300 m under the ground surface, excavated by means of a raise boring machine.
- 1.5 km long pressure tunnel.
- Access, outflow and other tunnels with a total length of 6.5 km excavated with drill and blast.
- Surge tank, to be excavated also with a raise boring machine.
- Power house, situated in a cavern 40.0 m high, 29 m span and 100 m long.

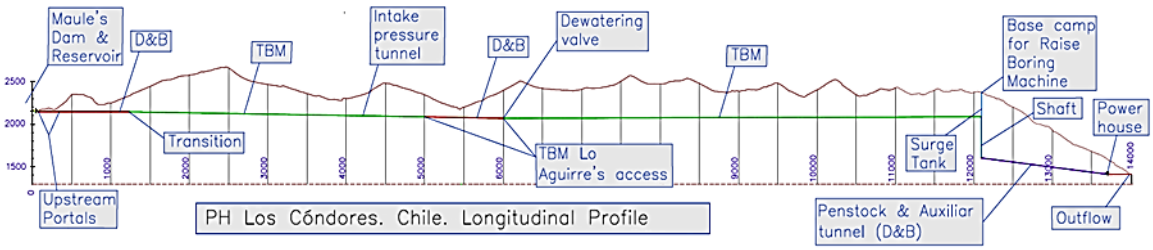

Fig. 3 Los Cóndores Project longitudinal profile

This document describes the construction procedure of the adduction tunnel, highlights main aspects of the design of the lining with precast concrete segments of the sections driven with TBM and summarizes the 3D analysis of a special section.

2 Construction strategy for the adduction tunnel

In order to shorten the execution time, the adduction tunnel is excavated using conventional drill and blast method and a TBM, working simultaneously in several excavation faces (see figures 4 and 5).

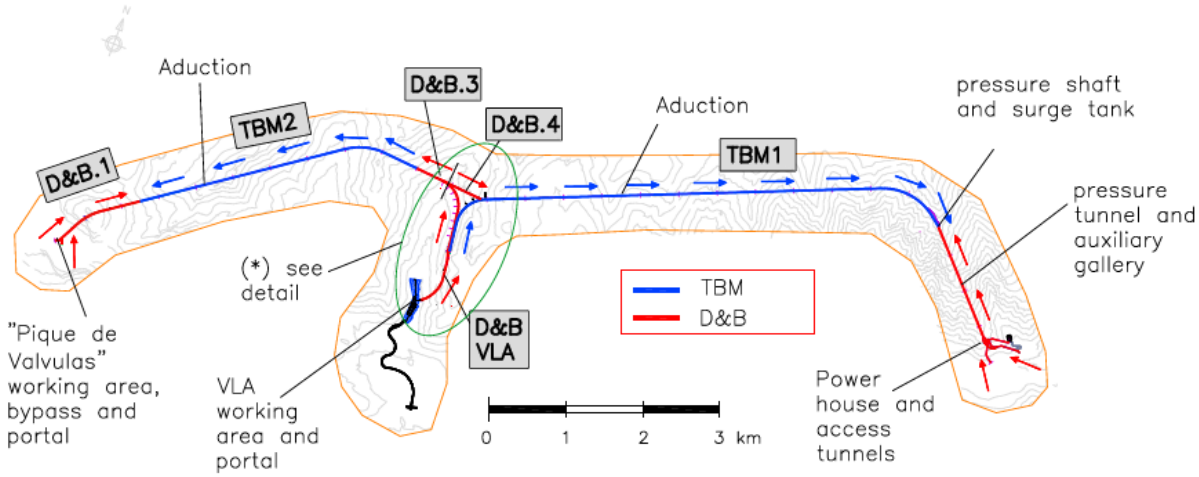

Fig. 4 Intermediate Access to adduction tunnel: Ventana de Lo Aguirre (VLA) tunnels.

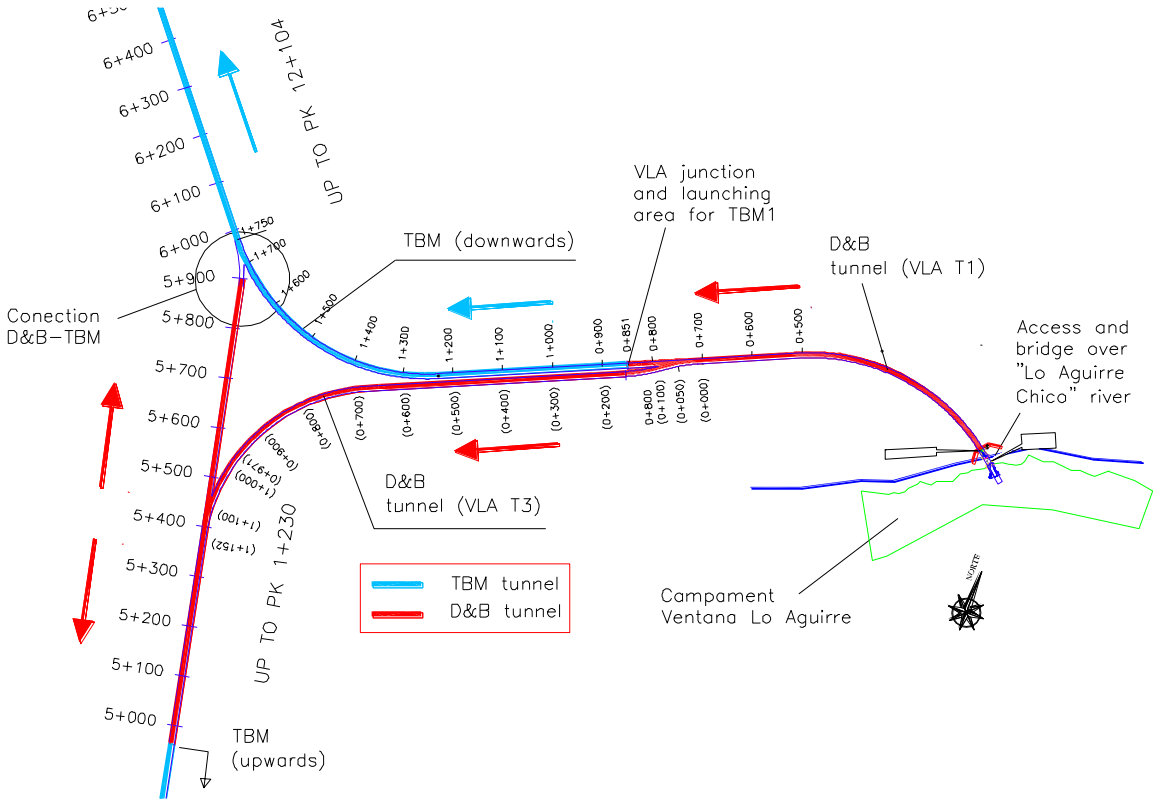

Fig. 5 Intermediate Access to adduction tunnel: Ventana de Lo Aguirre (VLA) tunnels.

- D&B.1. - The excavation face proceeds from the intake structure, following the conventional drill and blast method (or mechanical excavation when ground conditions allow it). This face will proceed downwards until its encountering the TBM 2, coming upwards from the Ventana Lo Aguirre intermediate access.

Fig. 6 VLA portal. To the left: Ventilation pumps. To the Right: Structure for the belt-conveyor and dumping system for excaveted rock from TBM.

- D&B VLA. - An adit has been executed at a transversal valley situated by the middle of the adduction tunnel (named Ventana Lo Aguirre, VLA), which provides several excavation fronts downwards and upwards, via circa 1950 m of access tunnels following an "Y" shape alignment.

Fig. 7 VLA bifurcation. To the right: Launching aerea for TBM.

The TBM is assembled at the entrance of this access, is transported through the "trunk of the Y" and is launched from a camera constructed at the north arm near to its fork.

- TBM1. - From the launching camera, the TBM follows a curve alignment until its intersection with the designed alignment of the adduction tunnel where it proceeds downwards until it reaches the shaft, where a cavern is constructed and the TBM partly dismounted.
  Apart for the shield, that is left behind the lining, and the cutting head, that is sawn into pieces, the TBM and its backup are driven backwards until its exit through VLA access.

- D&B3 and D&B4. - Simultaneously with the TBM, Through the other branch of the "Y", another intersection with the adduction tunnel is reached from which two additional excavation faces proceed down and upward, respectively.
- TBM2. - Once reassembled the TBM, it is driven through the upward branch of the VLA and the already excavated sections of D&B3 at the end of which a launching camera is waiting for the TBM. From this camera, the TBM proceeds until its encountering D&B1 excavation face.

The execution strategy here described provides a very flexible procedure and allows compensating possible delays in some of the excavation faces with additional advances on the others.

3 Geological description of the project's site

Volcanic rocks occur along the tunnel's alignment, namely Trapa-Trapa, Campanario and Cola de Zorro formations, consisting of andesites, pyroclastic and dacitic tuffs.

Youngest rocks are found near to the Maule lake, belonging to El Zorro fm., followed by the Campanario fm. that covers most part of the adduction tunnel and part of the area where the shaft is designed, to finish towards north with rocks of Trapa-Trapa fm. where the power house cavern and outflow tunnels are situated.

Faults or fault zones are found in two sectors, corresponding to the Cajon Lo Aguirre Chico and the Falla La Quebrada, both discontinuities will cut the tunnel.

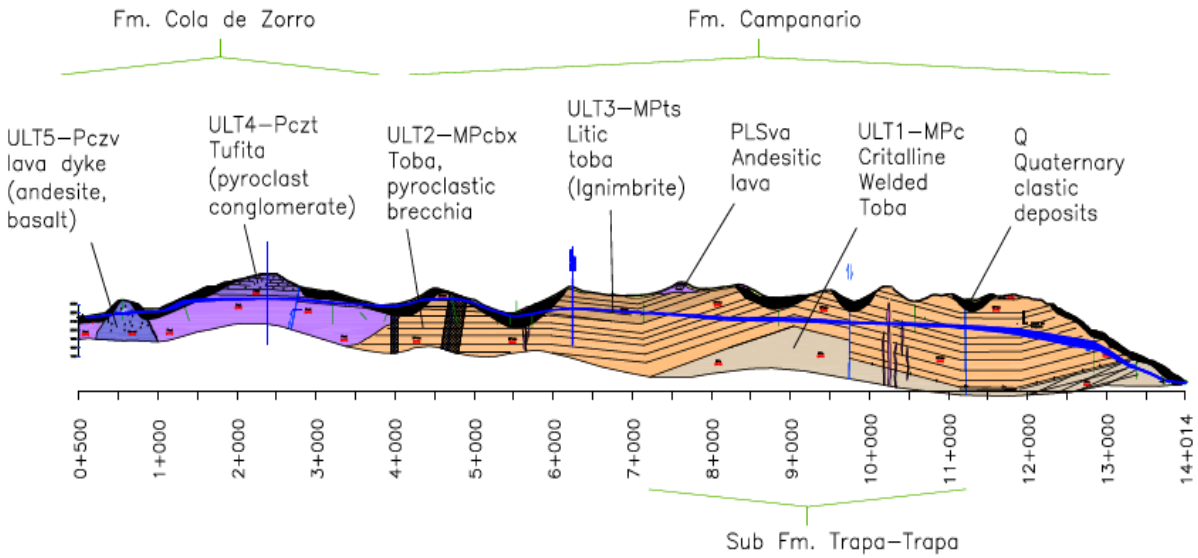

Fig. 8 Geological profile along the aduction, pressure shaft and pressure tunnels.

The geotechnical characterization of the rock mass has been made based on the Q (Barton) and RMR (Bienieawski) quality indexes. 21 geotechnical domains were established along the tunnel, for each of which properties of resistance and deformability were defined. Fault zones are treated as fractured and weathered rock.

A summary of these properties is found on Tables 1, 2 and 3.

Tab. 1 Geotechnical characteristics for different tunnels and soil/rock excavation

Area	Material	Specific weight (kN/m^3)	Internal friction Ø (º)	Cohesion (MPa)	Elastic mod. E (MPa)	GSI
Pique de Válvulas	Fills, Boulders, clayey gravels, sandy and silty gravels	21	40	0.02	25	----
Adduction Tunnel. Soils	Soils (0-340 m). Clayey gravel, sandy and silty gravel (morrein deposits)	18-20	38-40	0.02	25-100	----
Adduction Tunnel. Rocks Fm Cola Zorro	Brecchia (bch), Andesita (and), Tufita (tuf)	21-25	27 (tuf) 48 (and)	0.5 (tuf) 2.0 (and)	980 (bch) 7250 (and)	35-55
Adduction Tunnel. Rocks Fm Campanario	Brecchia (bch), Tufita (tuf), basalto (bas)	21-26	31 (tuf) 46 (bch) 49 (bas)	0.5 (tuf) 1.2 (bch)	965 (tuf) 5200 (bch)	35-40 (25-35 in DG16)
Discharge tunnel	Local fills and soils	21	40	2	25	----

Tab. 2 Rock Quality depending on formation (design estimation)

RMR	Rock classification	Fm. Cola de Zorro (%)	Fm. Campanario (%)
80 >RMR > 61	II	0	10
60 >RMR > 41	III	30-45	30-55
40 >RMR > 21	IV	40-55	30-50
20 >RMR	V	15-20	15-35

Tab. 3 Geotechnical parameters for faulted areas

	Rock mass characterization			
Faults (Depending on Geological Formation	Cohesion (MPa)	Friction (º)	Elastic Modulus E (MPa)	Poisson ν
Fm. Cola de Zorro (Pzct)	0-0.02	25-30	400-500	0.3
Fm. Campanario (MPc)	0-0.05	30	600-800	0.3

Piezometric levels along the tunnel where defined on an hydrological MODFLOW model calibrated adjusting the hydraulic conductivity of the rock mass that is able to fit water levels measured in 19 piezometers constructed along the tunnel alignement.

Water levels along the tunnel are shown on figure 9.

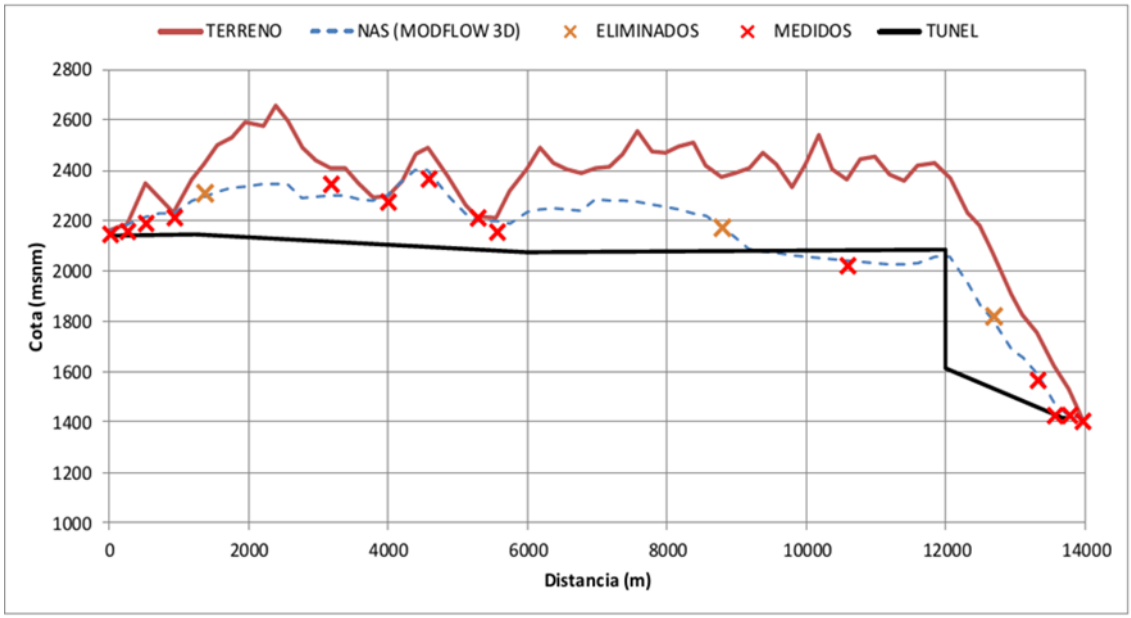

Fig. 9 Water level along the tunnel. Red crosses indicate position of piezometers

4 Design of the lining with precast concrete segments in sections driven with TBM

The lining of the tunnel has an internal diameter of 3.7 m and an outer diameter of 4.2 m. It is made with RC segments 0.25 m thick, joined by bolts. Each ring has 5 segments: a key segment, two trapezoidal segments and two rectangular segments. The ring is a universal type with an average length of 1.2 m. In Fig. 10 a sketch of the ring is shown.

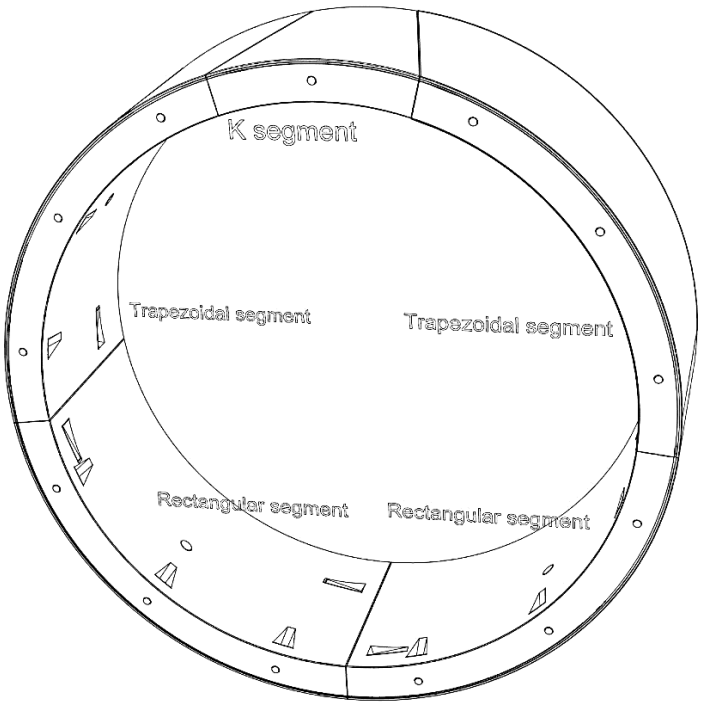

Fig. 10 Segmental ring lay-out

Three types of segments where designed, depending on concrete's strength: C40/50, C45/55 and C55/67, according with Eurocode 2 designation, to be used according to ground conditions and piezometric levels.

The design of concrete segments in "Los Condores Project", has been carried out based on the EN 1992-1-1:2004 EUROCODE 2 (EC2) and following the usual steps:

- Determination of the axial stress limit of the ring
- Design of the steel reinforcement
- Verification under conditions of construction and operation

4.1 Determination of the axial stress limit of the ring

In the first step, the maximum axial stress to be supported by the ring, N_{Ed}, is determined according to the following criteria:

- Maximum axial stress under high levels of compression (N_{Rd})
- Maximum axial compressive stress to avoid cracking (σ_c)
- Maximum axial stress in the longitudinal joints (F_{Rdu})

The lowest of this axial stresses determines the mechanical capacity of the ring.

For a rectangular cross-section, the maximum axial strength under compression loads, N_{Rd}, is (EC2 #12.6.1.3):

$$N_{Rd} = \eta f_{cd,pl} \times b \times h_w \times \left(1 - \frac{2e}{h_w}\right)$$

Where:

$\eta f_{cd,pl}$ is the effective compressive strength
b is the width of the cross section
h_w is the height of the cross section
e is the eccentricity of N_{Ed} in h_w direction

The coefficient of effective strength, η, takes the values:

$f_{ck} \leq 50$ MPa: $\eta = 1.0$
$f_{ck} > 50$ MPa: $\eta = 1.0 - (fck - 50)/200$

The compressive stress is limited in order to avoid longitudinal cracks, micro-cracks or high levels of creep, where they could result in unacceptable affection to the functionality of the structure (EC2 #7.2.2):

$$\sigma_c = k_1 f_{ck}$$

The recommended value of k_1 is 0,6

For partially loaded areas, local crushing and transversal stresses are to be considered. For a uniform distribution of load on an area, A_{co}, the concentrated resistance stress may be determined as follows (EC2 #6.7):

$$F_{Rdu} = A_{c0} f_{cd} \sqrt{A_{c1}/A_{c0}} \leq 3.0\, f_{cd} A_{c0}$$

Where:

A_{c1} is the maximum design distribution area with a similar shape to A_{co}
f_{cd} is the concrete design compressive strength.

For each class of segment of "Los Condores Project", results are summarized in table 4

Table 4 Axial stress limit of the ring classes

Concrete class	N_{Rd} (kN/m)	σ_c (kN/m)	F_{Rdu} (kN/m)	N_{Ed} (kN/m)
C40/50	4 647	6 000	4 571	4 571
C45/55	5 228	6 750	5 142	5 142
C55/67	6 229	8 250	6 285	6 229

4.2 Design of the steel reinforcement

Once the axial stress limit of the ring is determined, steel reinforcement is calculated as follows:

- Determination of the circumferential and longitudinal main steel reinforcement.
- Determination of additional reinforcement on the joints of the ring.
- Verification of reinforcement in the handling phases.

According to Eurocode, the total amount of circumferential reinforcement should not be less than As,min (EC2#9.5.2):

$$A_{s,min} = max\left(\frac{0.1N_{Ed}}{f_{yd}}, 0.002\,A_c\right)$$

Where:

A_c is the cross sectional area of concrete
N_{Ed} is the design axial compression force
f_{yd} is the design yield strength of the reinforcement

In the longitudinal direction, stresses on the segment are due to the thrust of the jacks, during the advance of the TBM. It should be verified that the applied stress does not exceed the design strength of concrete, f_{cd}

Usually, two cases are checked:

- Maximum standard stress: Hydraulic jacks of the TBM apply a pressure on the transversal joints of the ring. Maximum thrust provided by the hydraulic main circuit, under normal conditions of use is considered.
- Maximum exceptional stress: Maximum thrust that provided by the hydraulic main and supplementary circuits in an accidental condition (blocking of cutting head, shield trapping, and others).

If pressure applied by the jacks does not exceed the design strength of concrete, longitudinal steel reinforcement corresponds to the greater of the following values:

- Distribution reinforcement: 25 % of circumferential main reinforcement.
- Minimum geometric reinforcement: 2 ‰ of the concrete cross sectional area.

Concentrated loads of jacks on the circumferential joints, causes the appearance of tensioned areas in the segment. It must be verified if it is necessary to increase the steel reinforcement in these areas.

Two effects are checked:

- Bursting: Concentrated compressive loads produce tensile forces perpendicular to the compression direction
- Spalling: Oblique tensile forces occur at the corners of the joints

To estimate transverse and longitudinal tensile stresses of bursting and the spalling effect, two three-dimensional FLAC3D models where considered depending on the following

- Model 1. - Sliding between adjacent segments is allowed.
- Model 2. - Displacements in both lateral sides of the segment are fixed.

Transverse stresses are shown in Fig. 11. Spalling areas occur on the side where jacking forces are aplied between both shoes and between shoes and corners and bursting area occurs on the oposite side in front of the jacking shoes

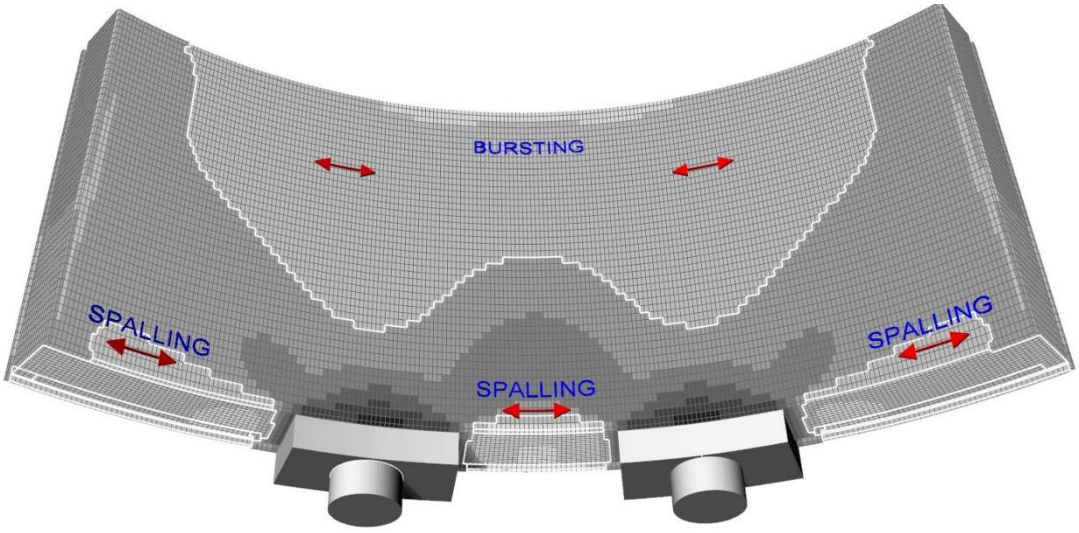

Fig. 11 FLAC3D analysis of bursting and spalling effects due the thrust of jacks

During demolding, handling, transport and mounting in the tunnel, segments are subjected to different states of loading, when strength of concrete has not yet reached its characteristic value. It should be checked whether these load states require additional reinforcement.

Following cases must be checked:

- Demolding: Removal and lifting of segment from its formwork and transport to stock area. Minimum required strength of concrete is determined.
- Stock after demolding. Segments of each ring are usually piled together. Required concrete strength is determined.
- Lifting and transport from stock area to the tunnel and lifting by the erector during its mounting in the tunnel. Segments are suspended from its central point. In this phase, concrete must have reached its characteristic strength.

4.3 Verification under conditions of construction and operation

The third step on the structural design of RC lining segments, is the verification under the conditions of construction and operation.

Three analyses are carried out:

- Structural analysis of the ring subjected to ground and water pressures.
- Tightness analysis of the ring during the operation phase subjected to inner water pressure.
- Seismic analysis.

4.3.1 Structural analysis of the ring subjected to ground and water pressures

As the tunneling machine advances, ground deforms until the gap between the segment and the excavation surface disappears, and the lining start to support loads tramisted by the ground.

In tunnels excavated in hard rock, ground deformation is usually smaller than the gap, so that segments do not support earth pressures. On the contrary, in weak rocks or deep tunnels,

this deformation is bigger than the gap, so that the entire ring will support the pressure of the ground. The way the rock is deformed depends on the dimensions of the tunnel, its depth and the characteristics of the ground. The magnitude of the deformation at a given point of the tunnel depends on the distance at which this point of the excavation front is located. For this reason, it is necessary to carry out the study using three-dimensional numerical models. In Fig. 12 principal compression stresses in the lining that were obtained in one of the calculations are shown.

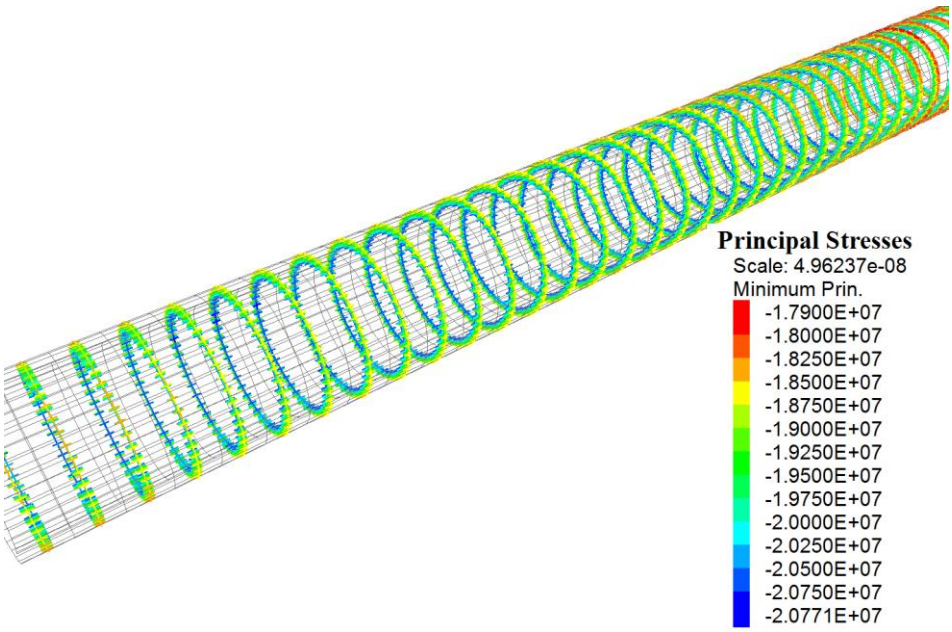

Fig. 12 Principal compression stresses in lining

In "Los Cóndores" project, there are two lithological formations: “Cola de Zorro” and “Campanario”. Additionally, several quality cases depending on Q and RMR values and fault zones were considered.

Other external loads as mortar injection, falling rock wedges, swelling of rock and external water pressure were also considered. The later being the most decisive for the design of the lining.

Tunnels situated below groundwater table, are subjected to a external water pressure. It is often assumed that the acting external water pressure on the lining is equal to the head of the groundwater table above the tunnel. Only in the case of an absolute tight lining this is correct, but in the case of concrete linings, this is a very conservative design. First, concrete has a certain permeability of $1e^{-8}$ m/s aprox. Second, due to the internal pressure, shrinkage and construction joints, cracks are generated in the lining, although the width of these cracks are lower than the required service limit (0.3 mm). Therefore, when the addution tunnel is empty or under free stream flow, seepage from the rock mass into the tunnel may develop and the external water pressure acting on the lining decreases. The tighter the rock mass compared to the lining, the less the lining is loaded by external water pressure.

Schleiss, A. (1997) quantifies the reduction of the external water pressure on the lining as a function of the permeability of the concrete lining and the rock mass, the width of cracks, the head of the groundwater table above the tunnel and the geometry of the tunnel. In Fig. 13, is illustrated the percentage of the external pressure respect the total water head versus the rock mass permeability, for the adduction tunnel assuming the existence of radial cracks into the reinforced concrete lining with 0,1 mm width and separated 3 m between cracks.

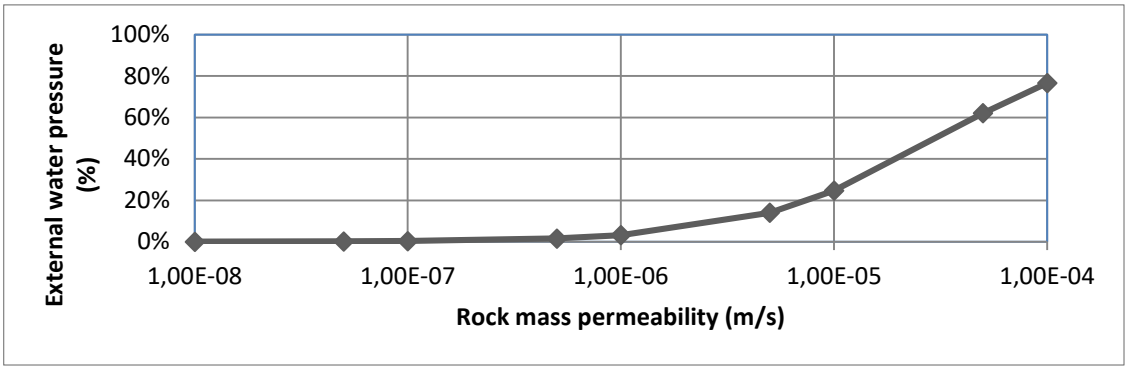

Fig. 13 Percentage of external water pressure vs rock mass permeability in permeable lining

In the Campanario fm., the maximum value of permeability is $9e^{-6}$ m/s, so the external water pressure acting on the lining is approximately a quarter of the total water head. In this case it should be extremely conservative to consider that the total water pressure acts on the outer side of the lining, and a reduction factor of 0.8 has been asumed.

In order to obtain loads on the segment rings, 189 three-dimensional calculations were carried out with FLAC3D varying rock caracteristics, type of concrete, overburden and external water pressure.

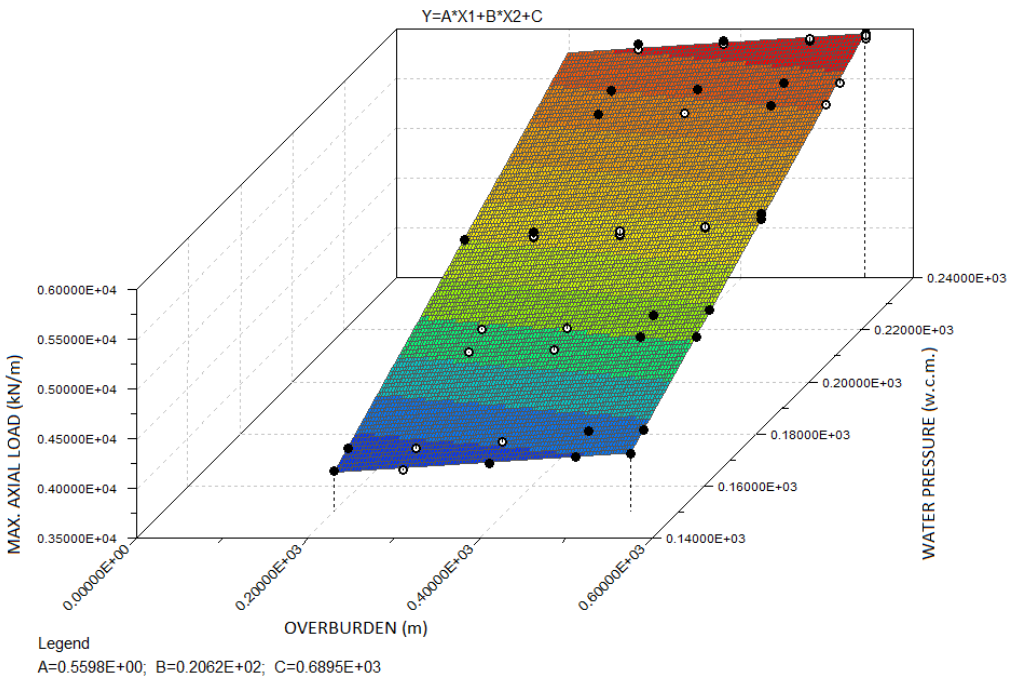

Fig. 14 Axial loads vs. Overburden vs. Water Pressure for Cola de Zorro Rock Class

As shown in Fig. 14 for the Cola de Zorro Rock Class, axial loads increase with external water pressure much more than with the overburden.

In order to verify the structural design of the RC segments, an axial force versus water pressure compatibility chart has been developed (Fig. 15). In this chart axial loads obtained with the simulations are ploted as a function of the external water pressure, for each rock mass class, along with axial stress capacity for each type of segment (horizontal lines) so as to determine the range of aplication for each of these.

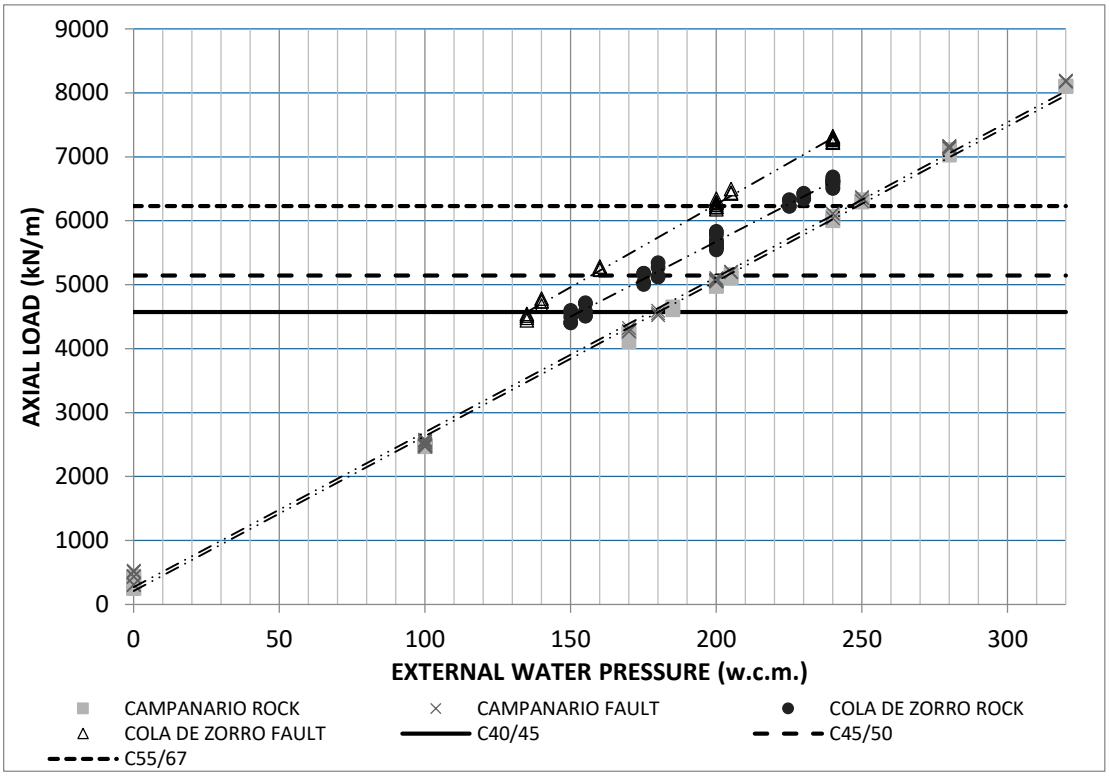

Fig. 15 Axil loads vs. External water pressure

4.3.2 Tightness analysis of the ring during the operation phase subjected to inner water pressure

During operational phase, the lining is subjected to an inner water pressure. In order to prevent water loss from tunnel toward the rock mass, it is necessary to verify EPDM sealing bands. According to the Technical Specifications of the project, the design of the EPDM seals must guarantee a minimum resistance agains internal water pressure of 10 bar at the end of a period of 100 years. This implies that the maximum allowable joint opening is 1.6 mm

In order to obtain the value of the aperture of the joints between segments, three-dimensional numerical calculations have been carried out asuming the following:

- The study is carried out in the zone of the tunnel with less overburden assuming a fractured rock mass.
- No external water pressure is considered.
- The ring-gap grout-rocky mass interaction is incorporated.
- Longitudinal joints are incorporated between segments, so the ring is not continuous.
- Bolts between segments of the ring are considered.
- Internally, the ring is under pressure.

In Fig. 16, the normal aperture of the segments joints at the position of the EPDM gasket is ploted. The largest aperture is 1.3 mm and occurs on the longitudinal joint located at the key of the tunnel.

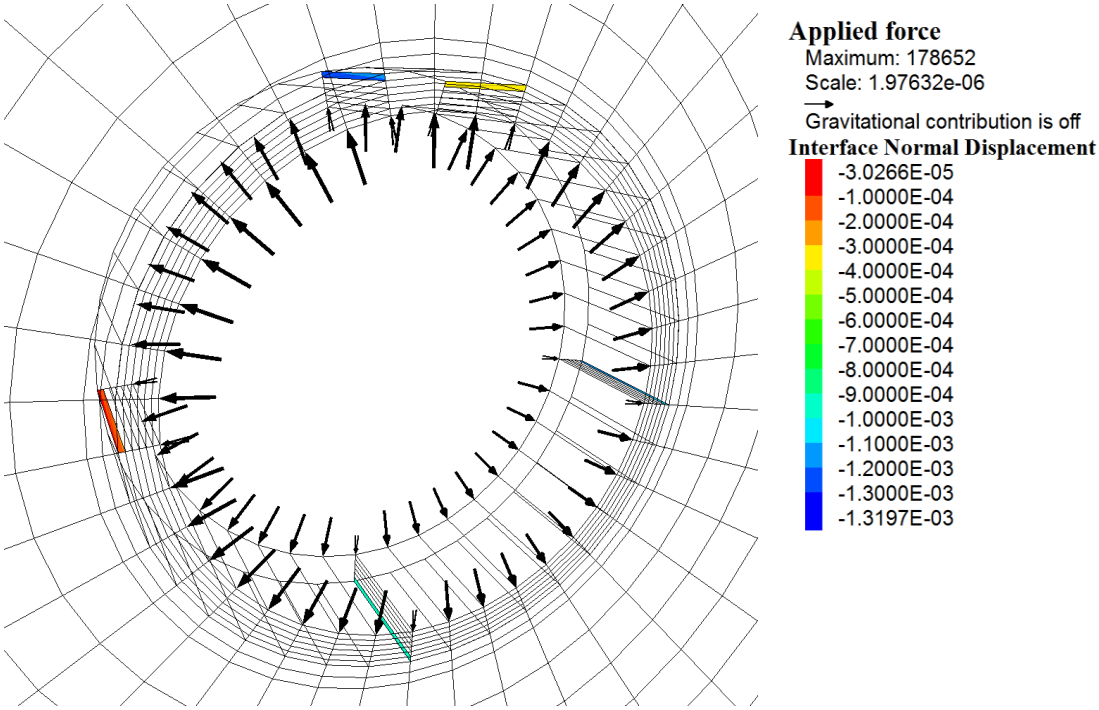

Fig. 16 Joint aperture at EPDM seal position

4.3.3 Seismic analysis

Following the ITA recommendations, for the seismic verification of the lining is required to estimate the response of the underground structure to ground deformation. Although the lining of the tunnel is significantly stiffer than the surrounding ground, a very conservative method of free-field racking deformation has been considered. To apply this method, it is necessary to obtain the free-field ground shear strain, γmax, which is expressed as:

$$\gamma_{max} = \frac{V_s}{C_s}$$

Where, V_s is the peak ground velocity for the design earthquake and C_s the propagation speed of S waves in the ground.

Table 5 summarizes the characteristics of the three types of earthquakes described in the Technical Specifications of the project.

Tab. 5 Earthquakes Design Characteristics

Earthquake type	Magnitude	Source-to-site distance (km)	PGA (g)
Subductive Interplates earthquake	8.5	227	0.256
Intermediate Depth Intraplate earthquake	8.0	140	0.450
Shallow Intraplate or Cortical earthquake	7.0	14.14	0.293

The one with the highest acceleration is the Intermediate Depth Intraplate type. However, it is necessary to correct the acceleration of the peak by a factor of 0.7 to take into account the depth at which the tunnel is located. The resulting design peak ground acceleration is 0.315(g).

Depending on the peak acceleration, the magnitude of the design earthquake, the distance to the epicenter and the type of ground in which the tunnel is excavated, it is possible to obtain the design peak velocity:

- Tunnel excavated in rock, V_s=0.3922 m/s
- Tunnel excavated in fault, V_s=0.7119 m/s

The propagation speed of S waves are indicated in the geotechnical report:

- Rock, C_s= 1050 m/s
- Fault, C_s= 300 m/s

It is obtained thus:

- Tunnel in rock, γ_{max} =0.00037
- Tunnel in fault, γ_{max} =0.0024

In order to verify the structural stability of reinforced segments, the two most unfavorable tunnel sections have been selected. For each of these, a three dimensional model has been developed in order to obtain the loads in the lining. Calculated shear strains are aplied to the nodes of the models, as it is shown in Fig. 17.

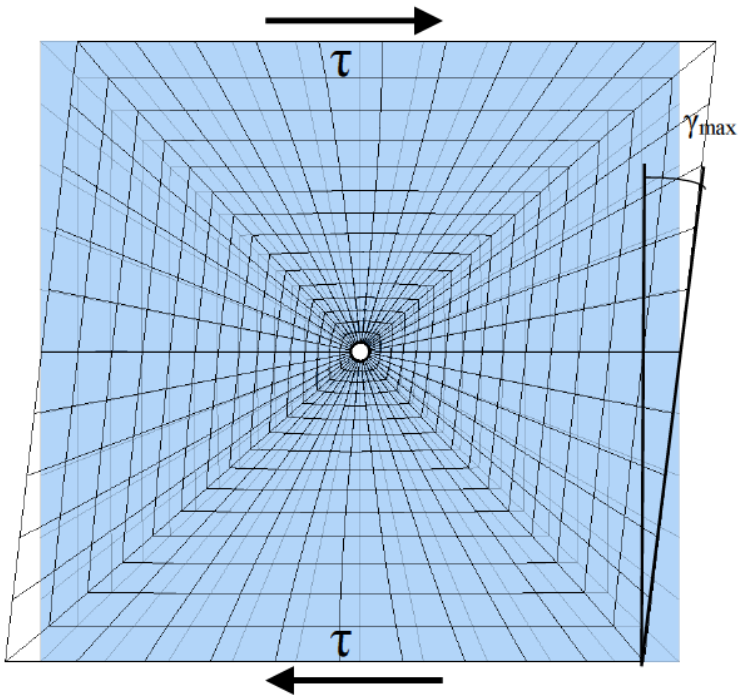

Fig. 17 Shear strain applied to the mesh

Imposed shear deformation is applied after the tunnel has been excavated and, then, new stresses in the rings are obtained. In this case, reduction strength factors are γ_c=1.2 and γ_s=1.0.

5 3D analysis of special sections

Junctions between two or more tunnels, section extensions of tunnels or connections between tunnels and shafts, have required to be modeled by means of a complete three-dimensional numerical model. The VLA junction, where the VLA adit connects with the tunnel excavated with D&B and the assembly room for the TBM, is one of the most complex sections.

The VLA access has an excavation section 5.60 m wide and 5.50 m high and is progressively enlarge to 6.37 m high. At this point, it branches into two tunnels. The assembly

room for the TBM is located near the bifurcation, enlarging progressively its dimensions in order to adapt to the necessary dimensions to house the cradle and the back anchor structure for the launching of the TBM and to execute assembly operations. The D&B tunnel has somewhat smaller interior dimensions: 5.50 in width and 5.50 m in height. At the bifurcation point, the distance between centerlines of both galleries is approximately 6.2 m. As the D&B gallery progresses, the axles are separated until they leave a distance of 16.8 m. This means that the rock pillar between both tunnels has a minimum width of 5 m.

A three-dimensional numerical analysis was carried out to design the required reinforcement for the geometry actually escavated. The model was built-up on excavation profiles provided by topography, so it was possible to create a very accurate geometry, as shown in Fig. 18, 19 and 20.

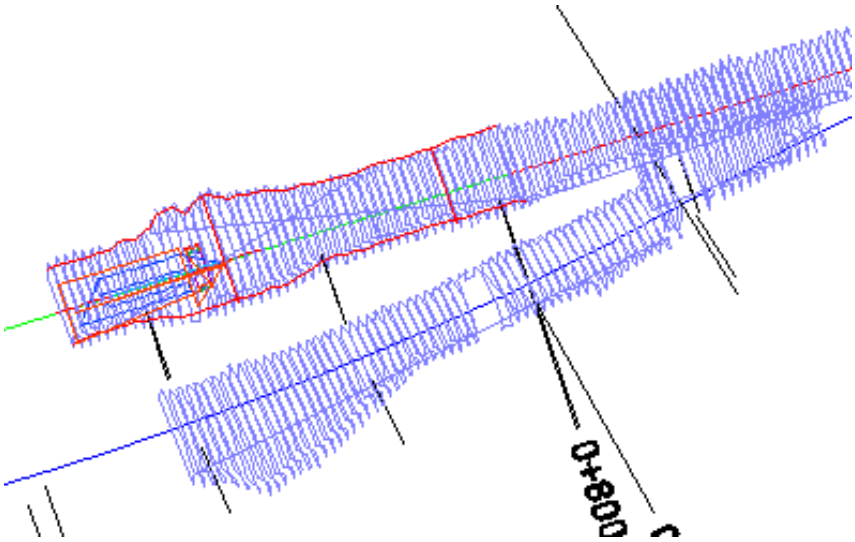

Fig. 18 Topographic profiles used to generate the model

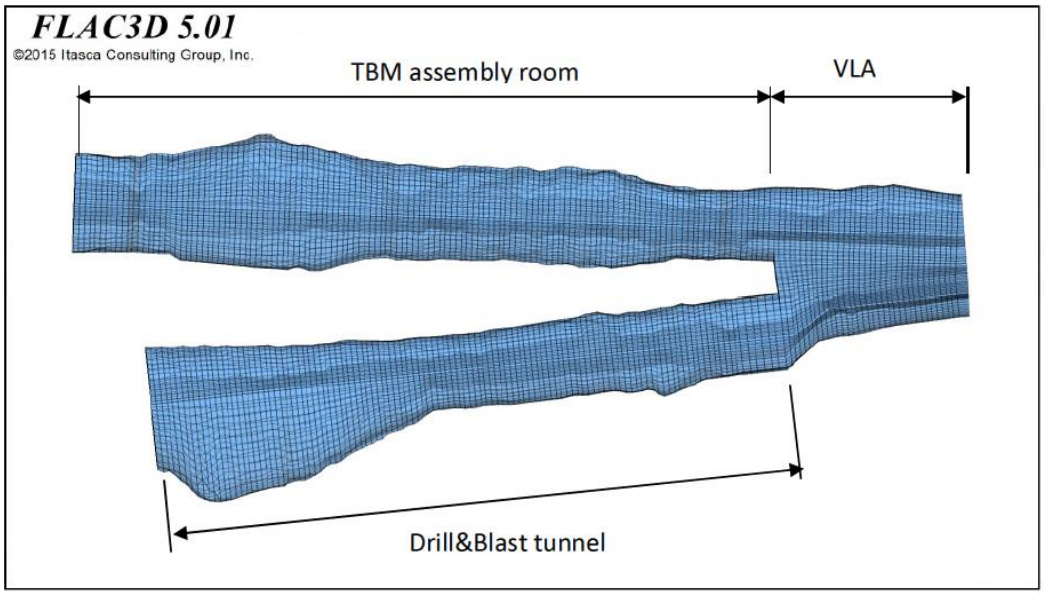

Fig. 19 Modeled tunnels

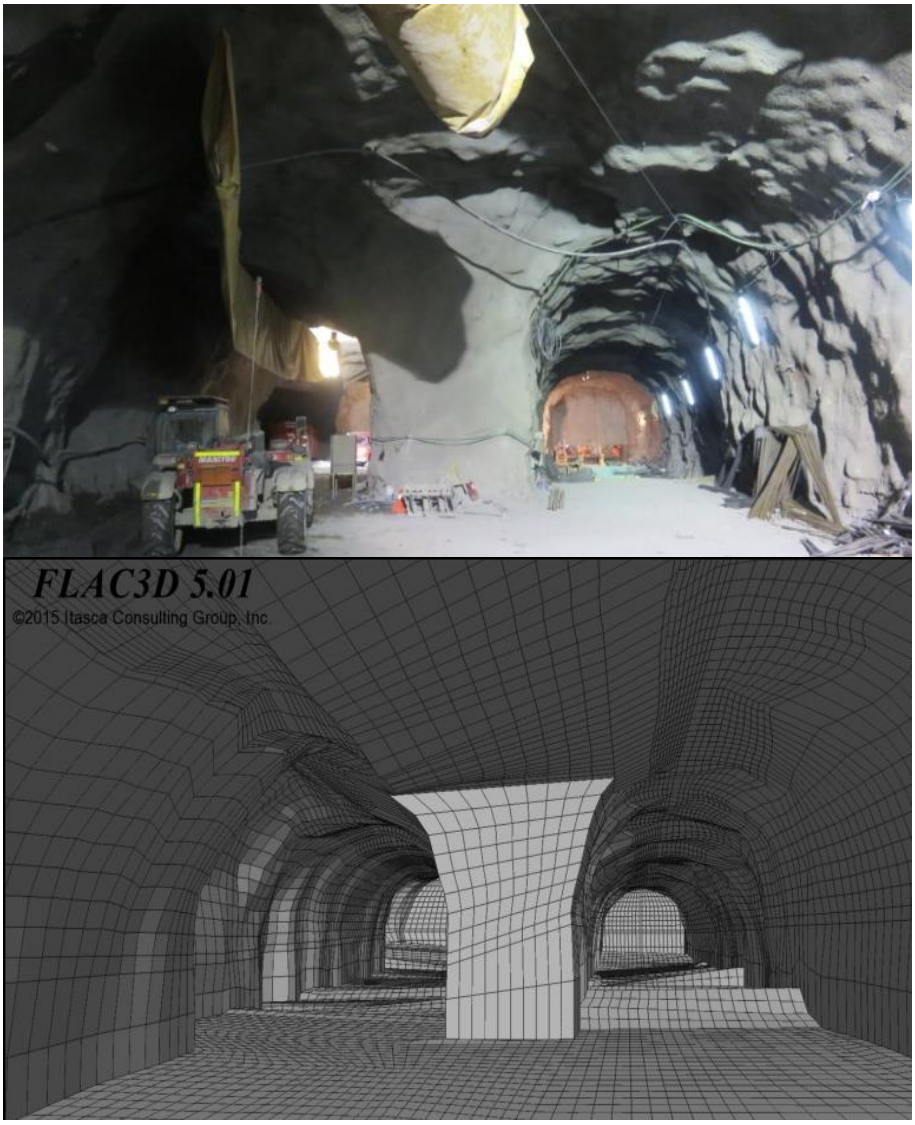

Fig. 20 Comparison between actual photo of the junction and model.

After the excavation of the TBM assembly room, benchmarks sections were installed in the area. Measurements indicated a maximum diametrical deformation of 5 mm during the excavation of the D&B tunnel.

In order to obtain the tensional state of the rock pillar, a back-analysis was carried out. Six models, whose stress-strain parameters were selected within the range of variation indicated in the Geotechnical Report were analyzed. Table 6 shows the parameters used.

Tab. 6 Parameter used in back-analysis

Case	UCS (Mpa)	Ei (GPa)	MR	Em (MPa)
1	50	30	-	9 216
2	30	30	-	9 216
3	15	30	-	9 216
4	50	-	300	4 608
5	30	-	300	2 765
6	15	-	300	1 382

In these models, control points coinciding with the position of the actual benchmarks chainages were located, obtaining the deformation in these sections after the excavation of the D&B tunnel. In Fig. 21 is shown the obtained values.

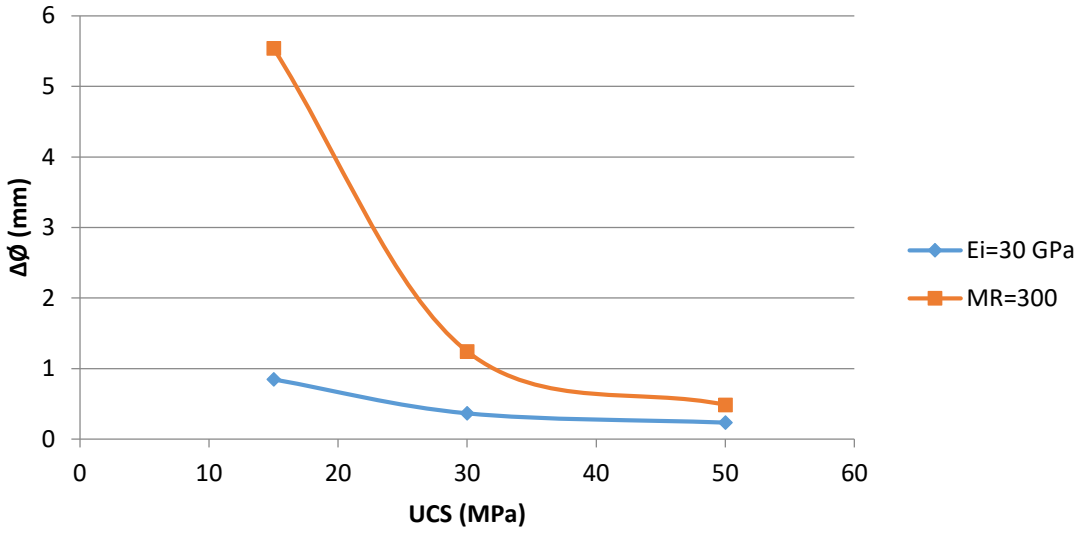

Fig. 21 Diametral deformation obtained in back-analysis

Strains in Case 6 match convergence measurements. This case would correspond to a rock mass with a UCS of the matrix rock of 15 MPa and deformation modulus obtained with an MR factor equal to 300.

In Fig. 22, the state of plasticity of the elements in the rock pillar is shown. In Fig. 23 the Minimum Principal Stresses (compressive stress) concentrations in the rock pillar are shown.

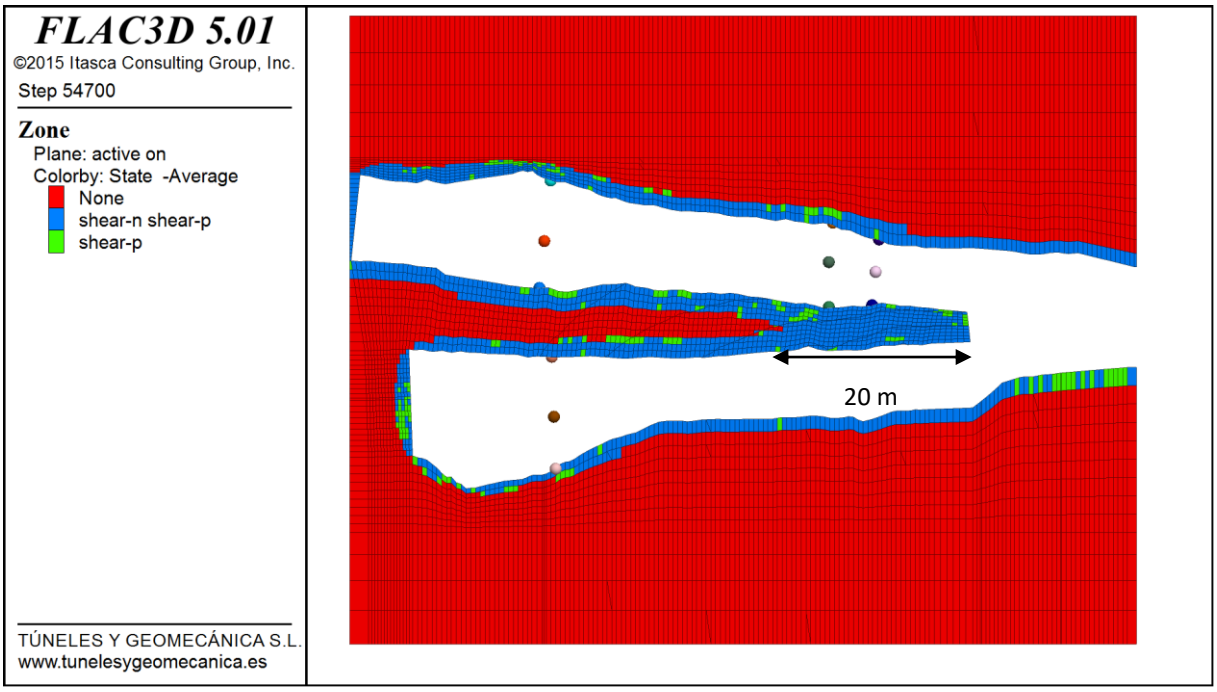

Fig. 22 Plasticity state of rock pillar

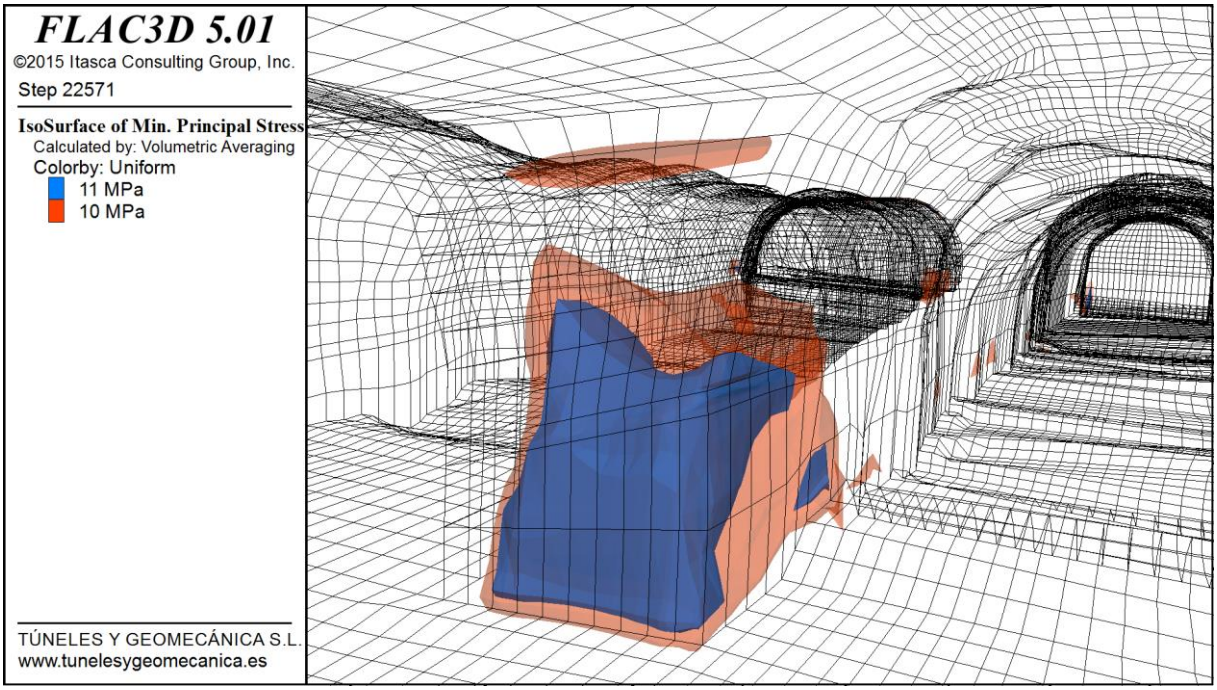

Fig. 23 Minimal Principal Stress in rock pillar

These stress concentrations and plasticity flags extend to the first 20 m of the pillar, the reinforcement was extended accordingly to this area.

6 Conclusions

The execution strategy defined for the excavation of the adduction tunnel in Los Cóndores hydroelectric power scheme includes conventional drill and blast excavation and TBM and precast segment lining sections excavated from up to 5 faces provides a very flexible procedure that allows compensating possible delays in some of the excavation faces with additional advances on the others.

3D FEM analysis constitutes nowadays a common mean not only to analyse especial sections but in general tunnel design as well. This is specially significant in the design of tunnel lining with precast concrete segments.

A sound design procedure for precast concrete segment lining has been described that allows a precise estimation of stresses on the lining provided an accurate definition of ground conditions supported by an extensive geological and hydrogeological investigation campaign is available.

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