2016

MARÃO TUNNEL (SECOND PHASE). MOTORWAY A4 - PORTO-BRAGANÇA

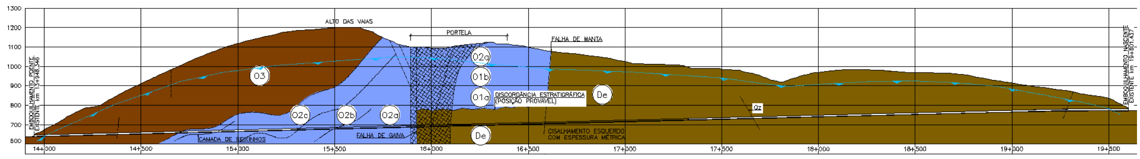
General features

Municipality: Amarante / Vila Real – Year of completion: 2016 – Purpose: Motorway
Type: Road tunnel
Cross-section: 9.80 m (h) x 12.80 m (l); Area: 97,5 m^2; Length: 2x 5 657 m; Maximum coating: 515 m
Terrain: Palaeozoic formations of the Ordovician and the Cambrian schist-greywacke complex
Owner: IP Infraestruturas de Portugal - Designer: Cenor (presently TPF) – Technical assistance during construction: Cenor - Contractor: Teixeira Duarte-EPOS
Observations: the longest road tunnel of the Iberian Peninsula, at the time of completion.

Work description

- Main purposes

The Marão Tunnel Project is part of the European Road 82 - E82, which begins in Oporto (Matosinhos) and extends the A4 Motorway until Bragança and Spain. It consists of 5 667 m long twin tunnels, resulting in a total length of 11 335 m.



Longitudinal profile of the south tunnel

- Geological and geotechnical conditions

The tunnel crosses the southern flank of Serra do Marão in an area of rugged terrain characterised by very high ridges and a very embedded hydrographic network. The concerned rock masses comprise Paleozoic formations of the Ordovician and Cambrian schist-greywacke complex, which is represented by the Desejosa Formation of the Douro Group. The schist formations are affected by contact metamorphism related to the setting of the Amarante granite, denoted by the presence of disseminated chiastolite minerals (mottled schists). The regional joint system is very complex and four families of discontinuities have been identified. Among them, the ones with direction N70ºW and N10º-20ºW stand out as concerning the tunnel, more specifically, the Manta and Gaiva Faults, being the latter intersected at chainage PK15+900 along a vast influence length, due to the complexity of this geological accident and the numerous faults associated. Considering the tunnel's depth, a base scenario characterised by a predominantly closed rock mass was set and the water inflows arriving at the excavation front were assumed to be, in general, of little importance, being its significance expected to increase near the aforementioned fault.

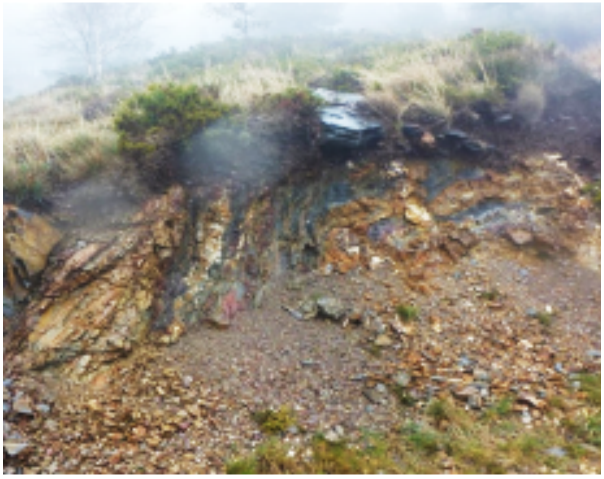
- Other constraints

The Consortium Autoestradas do Marão, with other counterparts in the design and construction phase, began the first phase of excavation works in July 2009, which was definitively suspended in July 2011, when both tunnel portals were almost completed and a total length of 7 340 m of both tunnels excavated. In February 2014, a new tender was launched for the design and construction of the works necessary for the tunnel's completion, with the winner being the Consortium Teixeira Duarte-EPOS, in association with the engineering consultancy firm Cenor (presently TPF). The works restarted in October 2014. Given the significant stoppage time, it was necessary to develop a specific analysis on the safety conditions of the already excavated tunnel. This aspect, in association with the characteristics of the occurring rock masses, as well as with an extremely short execution timeframe, represented relevant challenges for the development of the project. The current cross-section of the tunnels presents horseshoe shape with height and width of 9.8 m and 12.8 m, respectively, and was defined to ensure a minimum net area of 97.5 m² and, during exploration phase, a minimal utile height of 5.0m, enabling the insertion of two traffic lanes in each direction.

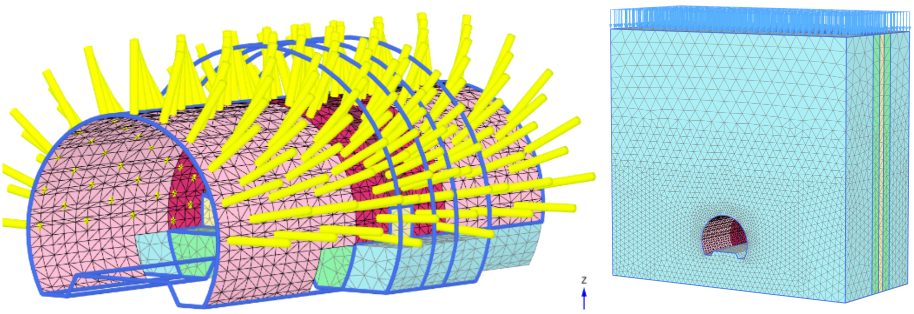
View of the east and west tunnel portals

- General description of the solution. Primary support

The tunnel was excavated by means of the conventional NATM sequential excavation method using explosives and heavy excavation equipment. Three primary support solutions were conceived, which applicability was defined in accordance with the geomechanical and hydrogeological characteristics of the surrounding massif. Thus, with the sole purpose of ensuring an adequate protection against rockfall and delaying rock mass deterioration until the execution of the final lining, the crossing of the most competent rock masses (RMR>45) was recommended to be executed under a steel fiber-reinforced shotcrete support (fiber ratio of 30 kg/m^3) applied in thicknesses varying from 5cm to 10 cm, in association with the eventual installation of punctual 4.0 m-long Swellex rockbolts – Primary Support Type PS1. In turn, the development of a stable and resistant self-supporting arch when crossing geotechnical horizons with good to average resistance and deformability characteristics (25<RMR<45) was achieved by means of the application of a 10 cm-thick steel fiber-reinforced shotcrete support, this time in complement to the systematic installation of Swellex rockbolts of the same length and arranged in a quincunx mesh and with 2.0 m x 2.0 m spacing – Primary Support Type PS2. Finally, a specific primary support was defined for application in fault zones, where the tunnels could cross highly fractured and weathered formations, with the possible presence of water, and therefore, characterized by very poor geomechanical properties (RMR<25). Considering the overburden height, of roughly 400 m, this would constitute an exceptional situation of high technical and executive complexity, since it could be associated with phenomena widely mentioned in deep tunnel's literature such as squeezing (although this was not foreseen in the expected geological-geotechnical scenario for the tunnels to be excavated), which needed to be foreseen and mitigated. Therefore, the adopted solution consisted of 2Ø16+1Ø25 lattice girders installed every 0.5 m to 1.0 m, as well as a 30 cm-thick NC70 electrowelded mesh reinforced shotcrete, in association with the eventual execution of provisional 20cm-thick shotcrete inverts reinforced with the same electrowelded mesh – primary support type SP3.



View of a fault zone on a road slope, at PK16+000



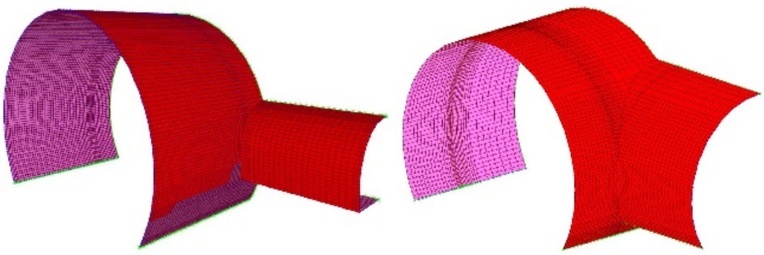
Three-dimensional numerical model developed for analysis of the fault box crossing

On what concerns the excavation procedure, the Primary Support PS1 was the only one that was associated with full face excavation, in excavation advance lengths comprised between 4.0 m and 6.0 m. On the contrary, SP2 application zones would be excavated in top heading, with advance lengths from 1.5 m to 3.0 m, followed by a bipartite bench excavation, with advance lengths from 3.0m to 4.0 m (max. 6.0 m). Finally, SP3 was naturally the support corresponding to a more restrictive execution procedure, with top heading excavation to be undertaken respecting advance lengths between 0.5 m and 1.0 m, followed by the materialization of the full face, divided into excavation of the central bench and alternate side benches, with advance lengths from 1.0 m to 2.0 m. Still on what concerns PS3, in the case of adopting provisional inverts, the one associated to the top heading would be executed in 3.0 m-long advance lengths, respecting a maximum distance to the excavation front of 4.0 m, while the one closing the full face would be materialized in advance lengths of 2.5 m.

View of the excavation section

Installation of the waterproofing system and lining reinforcement

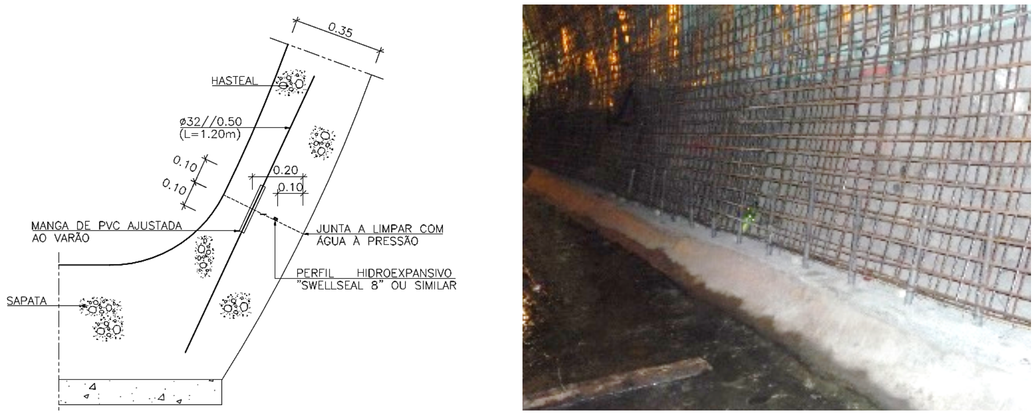
Several complimentary measures were also proposed. Its adoption was to be decided by the On-site Technical Assistance team, depending on the actual characteristics of rock masses, as well as on the readings obtained from the monitoring system. These measures included (i) rock mass treatment by means of grout injections around the excavation, (ii) its pre-support by means of spiles installation (Ø25 mm steel rods sealed with cement grout in Ø76 mm holes) or tubular forepoles (SCH40 pipes with Øext of 73mm and 5.16 mm thickness sealed inside and outside with cement grout), and also (iii) support reinforcement, either by treating the excavation front with injected fibreglass nails, by reducing the distance between girders (while maintaining the excavation advance lengths), or by means of the combined installation of two types of support: a primary one, consisting of Swellex rockbolts and a thin layer of shotcrete, with the aim of allowing stress relief through the controlled deformation of the massif, which would be followed by the (delayed) installation of a stiffer support. The project also envisaged the possibility of (iv) changing the excavation geometry by means of imposing and over-excavation as a way of ensuring the controlled deformation of the rock mass without affecting the thickness of the final lining to be concreted in a subsequent phase, as well as (v) changing the excavation procedure, i.e., the possibility of installing the support elements (lattice girders and reinforced shotcrete) in a later phase to minimize the acting loads. On what concerns vi) auxiliary drainage devices, the installation of 20m long sub-horizontal geodrains (made of corrugated and perforated Ø65mm PVC pipes, wrapped in geotextile) was also recommended.



Structural analysis of the final lining in the intersection zones with the transversal galleries (pedestrian and vehicular)

- General description of the solution. Final lining

The fact that 65% of the tunnel length was already excavated at the moment of work resumption and that the definitive footings were already executed along a 500 m stretch (North Tunnel / West Front) was a major constraint to the definition of the final linings. On the other hand, when the concrete linings were analysed in the framework of the previous contract works, it was verified that the reinforcement did not comply with the applicable regulations. The solution which was conceived in order to avoid the need of redefining the profile of the excavation and/or demolishing of the foundations already concreted was to define the materialisation of a structural hinge between the base of the sidewalls and its footings, resulting in the accomplishment of a force distribution which was compatible with the previously-defined final lining geometry.



Detail of the side-wall-footing connection of the final lining

The definitive support consisted of a reinforced concrete lining, with a constant thickness of 0.35m, founded by 0.45m-thick footings, which was replaced by a definitive invert when in presence of poor-quality rock masses.

Pavement construction sequence and infrastructures installation

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Specification LNEC E 378, LNEC E 464-2007 and LNEC E 465-2007. EN 14490-2010. ASTM A767:2009.

2009

REFURBISHMENT OF THE SALAMONDE II HYDROPOWER DEVELOPMENT. CAVERN POWERPLANT

General Characteristics

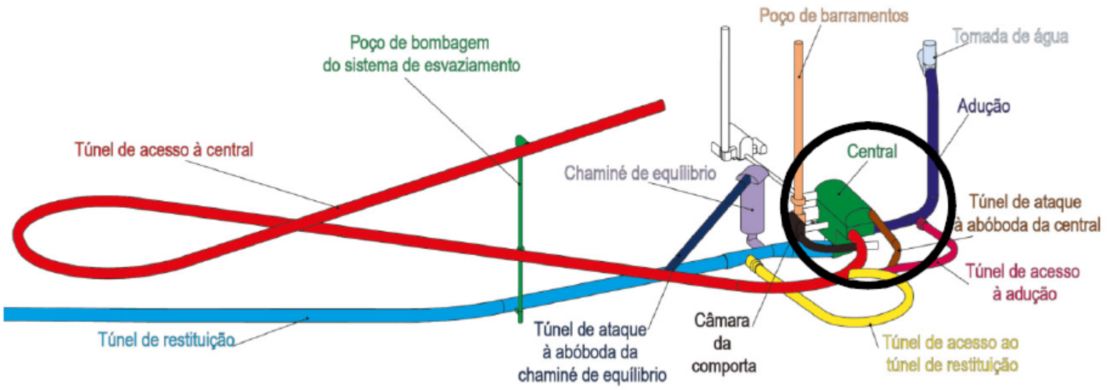
Municipality: Vieira do Minho
Purpose: power generation
Client: EDP - Owner: EDP – Design Engineer: Coba
Contractor: Group of Contractors Teixeira Duarte., Epos and Seth
Supervision: Joint-Venture Fase & Gibb

Description of Work Characteristics

- Framework and key objectives

The Salamonde HPP is located in the surroundings of the Serra do Gerês National Park, on the left bank of the Cávado river between the Salamonde and Caniçada reservoirs, near the village of Salamonde. It is composed by several infrastructures, in particular by its underground plant:

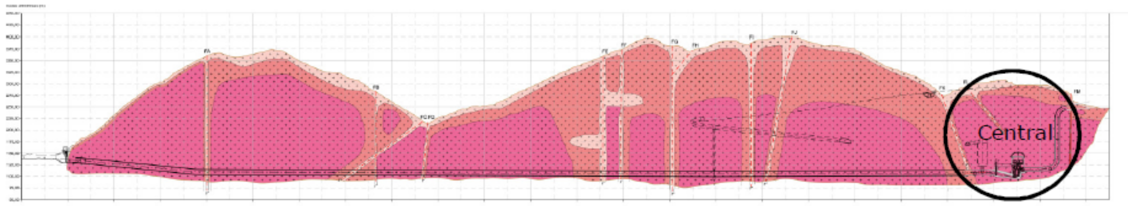
Units:	1 (Francis vertical pump reversible)
Rated capacity:	206 MW
Nominal flow:	200 m³/s (turbine); 163 m³/s pump)
Net head (turbine):	115 m
Net pumping head (pump):	120 m
Geometry:	height - 57 m; length - 67 m
Overburden:	165 m



General diagram of the Power Strengthening of the Salamonde Power Plant (Salamonde II)

- Geological and geotechnical conditions

The undertaking is located in the NW sector of the great paleogeographic structural unit known as the Central Iberian Zone (sub-zone Galicia Média-Trás-os-Montes), and the project is included in the Gerês granite mass. This granite presents, in general, coarse facies in the periphery and medium to coarse grained porphyritic towards the interior of the mass. The region is densely fractured, and the fractures are often filled by quartz veins. Regionally, the predominant fracture systems are: NNE-SSW, N-S to NNE-SSW and NNW-SSE. The N-S prevails in this granite.



Zonamento Geotécnico

ZONA	W	F	RQD (%)	σ_c (MPa)	γ (kN/m^3)	RMR	GSI
ZG1	≤ W2	≤ F2	> 70	100	27	> 70	70 - 85
ZG2	W2 a W3	F2 a F3 (**)		60	26	50 - 70	50 - 70
ZG3	W3 a W4	F3 a F4 (***)	> 20	45	25	30 - 50	30 - 50
ZG4	W4 a W5 (*)	F4 a F5	< 20	0,5	24	< 30	<30

(*) Pontualmente W3
(**) Pontualmente F4
(***) Pontualmente F4-5

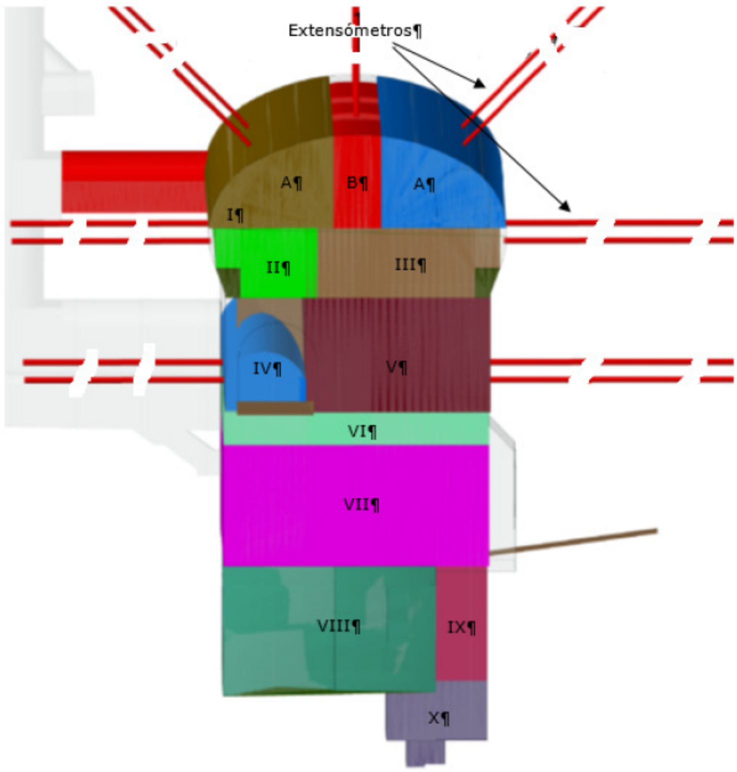
Interpretative geological-geotechnical longitudinal profile

- General description of the solution

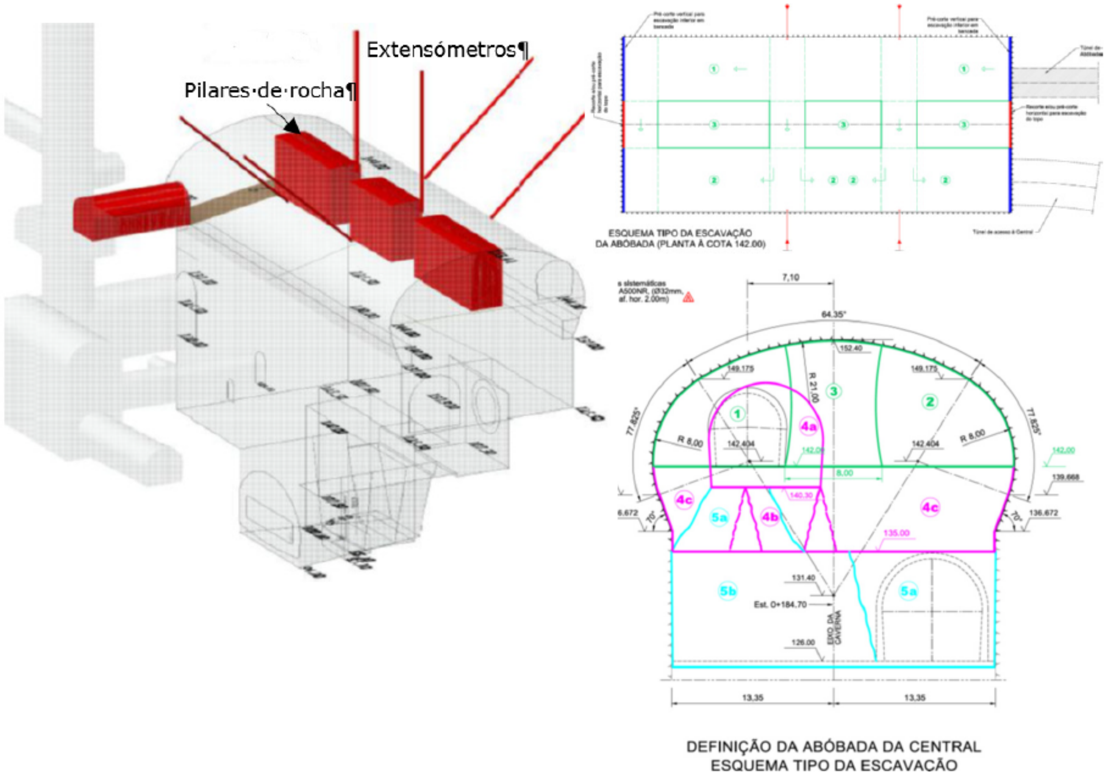
The excavation of the power plant cavern was phased and began with two longitudinal galleries, on each side of the dome, which allowed the first strain gauges to be installed. Once the excavation of the entire dome was completed and the respective support was applied, the beams of the overhead crane were built and the corresponding ground anchorages were installed. Subsequently, the remaining excavation was carried out for the execution of the piers of the overhead crane beams and for the insertion of the generator unit. The support of the excavation walls of the power plant was made exclusively with rock bolts, while the dome was made of shotcrete reinforced with steel fibres and rock bolts. In both solutions, rock bolts were characterised by steel bars, A500 NR, executed with cement grout, with diameters of 32 mm or 25 mm.

- Main design aspects

Depending on the construction schedule, the excavation, concreting and equipment assembly activities were distributed over time and with a time lag in such a manner as to require, in a provisional phase, the support of both the beams and the overhead crane itself, using ground anchors instead of the traditional reinforced concrete piers. The excavation of the power plant cavern was staged and began with the construction of two longitudinal galleries, positioned on either side of the dome.



Excavation phases of the cavern



Construction staging of the cavern dome

All excavations were accompanied by a monitoring plan to ensure the safety of the works and to assess the stability of the rock mass. Monitoring of deformations inside the excavations of the power plant cavern was carried out using 12 multipoint extensometers and 6 convergence sections with 9 topographic targets. 11 piezometers were installed for knowledge of the hydrostatic pressures installed in the surrounding rock mass. The evolution of the anchorages' behaviour was carried out using load cells (total of 7 units).

Excavation of the entire dome and execution of the anchored beams of the overhead crane. Ramp and excavation of the lower level

- Main construction aspects

The excavations were carried out using blasting. One of the most relevant aspects related to this project and to the construction methods adopted for the excavations was the restrictions imposed in terms of limiting the vibrations caused by blasting. The distance between the excavations and the dam, the Salamonde I underground power plant (in operation) and the other structures and mechanical and electrical equipment comprised the main constraints on the design and staging of the blasting plan, whose implementation was accompanied by a tight control system of the maximum vibrations specified in the design.

Beginning of cavern excavation by the longitudinal gallery on the water intake side (turbination)

Excavation of the power plant. In the foreground, the assembly hall. In the background, the area of the pumping unit

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2001

OPORTO LIGHT RAIL TRAM. BOLHÃO UNDERGROUND STATION. OPORTO

General Characteristics

Location: Oporto
Purpose: Light Rail
Owner: Metro do Porto – Design Engineer: Coba
Client: Transmetro - Contractor: Transmetro - Supervision: CGK
Construction period: 2002 / 2004
Detailed Design and Technical Assistance: 2001 / 2004

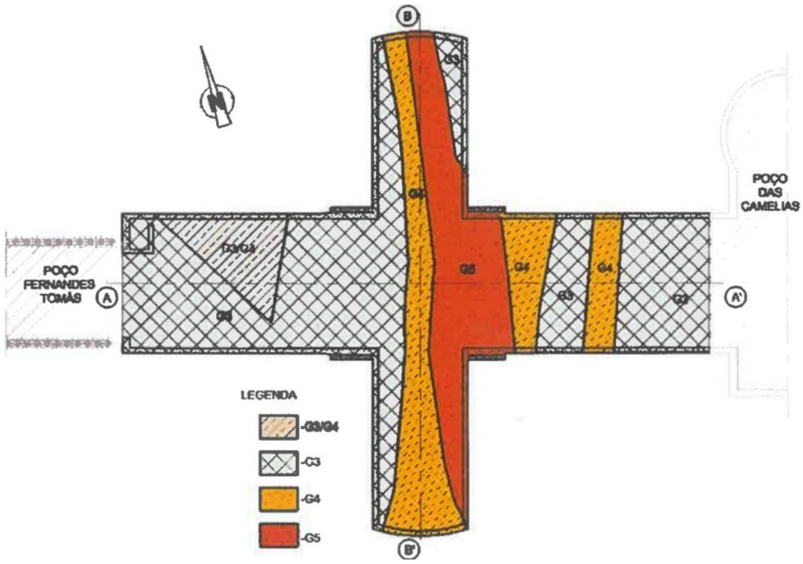
Description of Work Characteristics

- Framework and key objectives

The Bolhão Station is integrated in the common stretch of lines A, B and C of the Porto Light Tram, in a densely urbanised area, in the centre of the city's commercial activity. It develops along the Fernandes Tomás Street, at the intersection with Alexandre Braga Street, with the "Capela das Almas" Chapel (historical heritage of the city) on one of its corners and the traditional Bolhão Market on the other. The station essentially consists of a main cavern where the railway platform is located, with a length of 70 m and an excavation cross section of 186 m^2 (surrounding the track tunnel, previously executed with TBM and segmental lined with precast concrete) and a transversal gallery running orthogonal to the first, with a length of 62 m and a circular section of 160 m^2, through which the access to the low mezzanine at platform level is made. It has a gross floor area below ground of 3 400 m^2. At the intersection of the two galleries, the height of the cavern section is increased in a section about 20 m long, with the excavation cross section now having 244 m^2, in order to allow for the insertion of the access to the high mezzanine, achieved out through a third gallery of smaller dimensions. Completing the set of underground works are the access shafts and operational rooms excavated in the open air on Alexandre Braga Street and under Fernandes Tomás Street, as well as an access emergency shaft exit to the Camélias Park and the tunnel connecting to the main gallery.

- Geological and geotechnical conditions

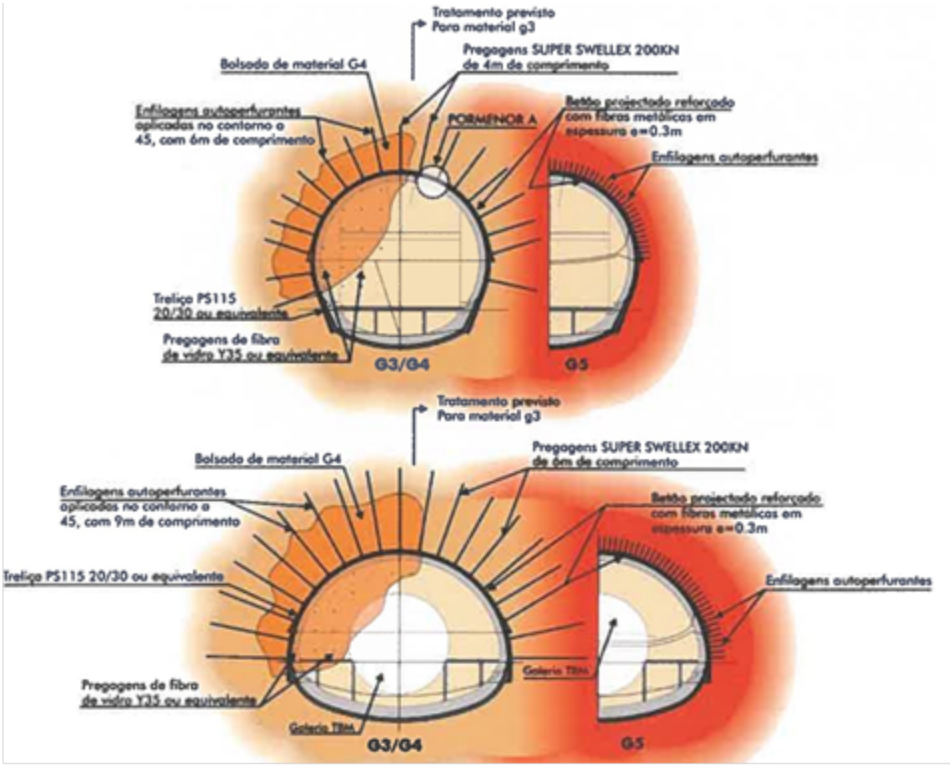
In geological terms, the Station site area is characterised by the occurrence of the designated "*Granito do Porto*" [Oporto Granite], underlying fill deposits, alluvium soils and granite saprolites, which can reach a total thickness of about 13 m. The "Granito do Porto", generally a two-mica granite, medium to coarse grained, leucocratic, presents several weathering degrees that result in variations in its mechanical characteristics. The typical scenario of the intervention area corresponds to granite materials of different weathering degrees with pockets and strips of material of poorer quality, disseminated among materials of better quality.



Geological and geotechnical model

- Main design aspects

Since the ceiling levels of the two caverns were similar, the crown of the station had an almost flat configuration, requiring the construction of a reinforced primary support structure with sufficient load-bearing characteristics and rigidity to withstand the adverse conditions encountered during construction. At the time of the station excavation (carried out by the NATM Observational Method, with two excavation phases, upper section and bench), the railway tunnel had already been built, using a tunnel boring machine and lined with segmental precast reinforced concrete. On the one hand, this allowed the rock mass to be drained prior to excavation and on the other it made it necessary to include the removal of the segmental liner in the construction phase of the station. As the excavation of the main gallery would take place after the execution of the railway track tunnel with which it is aligned, and acknowledging the presence of water in the ground, it was decided to drain it in advance using long drains, executed from that gallery. The constructive sequence adopted considered two main excavation phases, for both the main gallery (including the intersection) and the transversal gallery. A first phase corresponding to the excavation of the upper section and a second corresponding to the bench. After excavation of the first phase of the main gallery at the western end of the station and also of the intersection chamber, work began on the reinforce structure of the primary support of the intersection. Once the reinforcement of the intersection chamber was finished, the transversal gallery was excavated, under the protection of a set of forepolling umbrella arches, initially from the south side (opposite the "Capela das Almas" Chapel) and later from the north side. At the same time, the excavation of the gallery benching was started, passing through the transversal gallery, gave access to the excavation fronts of the lower gallery. Subsequently, the execution of temporary concrete invert was carried out in the intersection chamber and in adjacent extremities of the transversal gallery. The excavation of the gallery's main section was carried out in two excavation phases. The first phase corresponded to the opening of the upper semi-section of the upper section with the temporary concrete invert at elevation ~70 m. The second phase corresponded to the excavation of the lower section carried out in several sub-phases. The primary support applied in geomechanical complex G3 (W_3 mass, F_4 to F_3) was made of shotcrete with 0.30 m thickness reinforced with metallic fibers, metal lettice girders type PS 115 20/30 and 200 kN expansion bolts, every 1,5 m with 6 m in the cavern and 4 m long in the transversal gallery. In geomechanical complex G5 (W_5 mass, F_5), it was decided to install rigid inclusions, specifically self-drilling forepolling or grouted dowls, spaced 0.4 m apart, in a conical umbrella. The distance between lattice girders in addition to the running lengths was 1,0 m



Main gallery and transversal gallery. Excavation and primary support phases

The main gallery and the transversal gallery intersect perpendicularly in the central area, practically at the same level. For architectural reasons, the intersection has a larger section height. The junction occurs in the intersection of the Santa Catarina and Fernandes Tomas streets, directly affecting the building of the "Capela das Almas" Chapel, considered historical heritage of the city, on the northeast side. The total overburden in the symmetry axis is of about 12 m, of which only 4 m correspond to the rock mass on the western half. The remaining 8 m correspond essentially to soils. For this reason, the goal was to reduce the height of the section to a minimum, conditioned by the minimum clearance defined in the architectural design. The reinforcement of the intersection chamber was made with a reinforced shotcrete element. For construction staging reasons, it was necessary to install bar anchors at the reinforcement pillars, as well as the construction of reinforced concrete distribution beams at the base of the temporary invert, which allowed the beginning of the excavation of the transversal gallery and later the bench of the cavern. The reinforced concrete adopted in the final lining of the station was designed for a lifetime of 100 years. Thickness varied from a minimum of 0,5 m in the crown of the main gallery and transversal gallery and 0,6 m in the crown of the intersection, to a maximum in the invert of the Cavern and intersection, with a thickness of 0,7 m.

Intersection between main gallery and transversal gallery

Primary support

- Main construction aspects

The project assumed the application of the Observational Method. The construction method and primary supports were adjusted according to the real geological and geotechnical conditions encountered and according to the response of the rock mass to excavation, quantified in the monitoring system installed. Surveying was carried out in advance, with the execution of sub-horizontal rotation drilling, in complement to the geological and geotechnical mappings carried out in the gallery executed by the tunnel boring machine and in the cavern itself and adjacent wells. The underground station was excavated practically entirely from the Fernandes Tomas Street shaft, contiguous to the development of the main gallery. An area with a G5 mass was encountered during construction, so it was necessary to adapt the support for G5 of the main section to the intersection, as well as to reinforce the portals of the transversal gallery, by means of the execution of a set of five forepolling umbrella arches, with a fan distribution, with a 50° inclination in the first umbrella, 40° in the second and up to 10° in the fifth at the entrance of the transversal gallery and excavation of the full section of the upper section.

It was thus possible to overcome the immediate tunnel portal difficulties created by the flat roof geometry of the crown and the nature of the rock mass. Whenever the geological and geotechnical mapping of the work front recorded the occurrence of intrusions of geomechanical complex G4 (W_4; F_{4-5}) with expression and significance for the works, the expansion bolts were replaced with self-drilling forepoles inclined at 45° in a conical arrangement, with a length of 9 m in the cavern and 6 m in the transversal gallery. In both cases, the recommended excavation length was 1,5 m.


General view of the intersection area after completion of excavation

Given the urban nature of the works, located in an area of high commercial activity in the city, the main constraint to the project and to construction was the need to ensure the safety of people and the integrity of adjacent buildings, which was achieved through the installation of monitoring devices and the observation of their behaviour during construction. Naturally, as it is a building of high historical and heritage value for the city, special attention of given to the “Capela das Almas” Chapel.


Excavation from the shaft at Fernandes Tomas Street

The observation plan included inclinometers, multipoint strain gauges, accurate levelling surface marks, piezometers and convergence targets inside the galleries read by optical methods. 8 monitoring profiles were read along the Fernandes Tomas Street. For the buildings, particularly for the “Capela das Almas” Chapel, the distortion levels attained were of the maximum order, of 1/1600 at Sta. Catarina Street and 1/2200 at Fernandes Tomas Street, values well below the limits imposed by the study of risk assessment of the buildings, which indicated maximum acceptable distortion values for the “Capela das Almas” Chapel of 1/850 (limit of attention).

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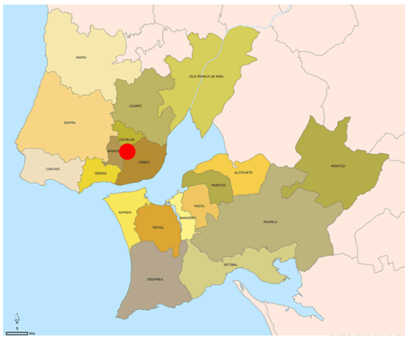
1993

CARENQUE TUNNEL. A9 MOTORWAY (LISBON OUTER RING ROAD). LISBON



General Characteristics

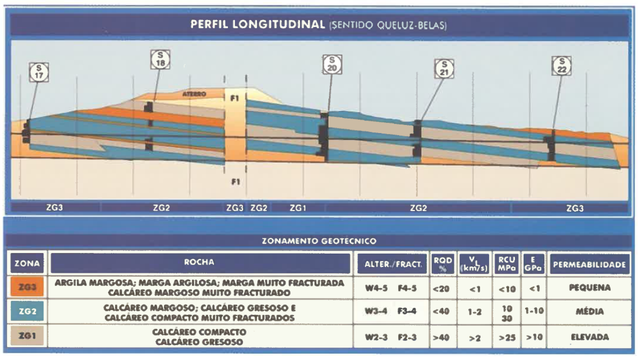
Year of completion: 1993
Location: Lisbon
Purpose: preservation of an area of high palaeontological value with records of dinosaur footprints from the Cretaceous period on a layer of fractured and thin limestone
Type of Works: Road Infrastructure
Owner: Brisa – Design Engineer: Coba - Contractor: Bento Pedroso Construções - Client: Bento Pedroso Construções – Supervision: Brisa

Description of Work Characteristics

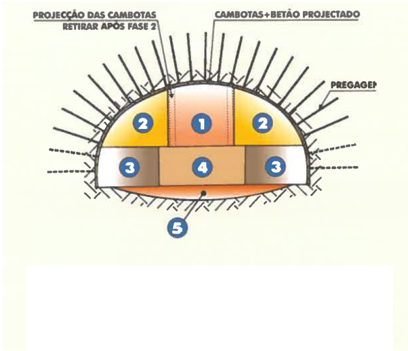
- Framework and key objectives:
The Carenque tunnel is integrated in the section of the A9 Motorway - CREL (Lisbon Outer Ring Road – Estádio Nacional/Alverca), sub-section Queluz-Belas. It is a project designed to preserve an area of high palaeontological value with records of dinosaur footprints from the Cretaceous period on a layer of fractured and thin limestone. It consists of two twin galleries about 280 m long, with an excavated area of about 170 m^2, separated by an 8 m wide rock mass pillar. The road clearance required by the Client was adopted when defining the cross section, i.e. with dimensions of 15,50 x 5,10 m, for a carriageway with three 3,5 m-wide lanes, a 1,0 m-wide left shoulder and a 4,0 m-wide right shoulder, in order to allow for a possible future widening to 4 lanes.

- Geological and geotechnical conditions
The main geotechnical constraints of the works are associated with the poor quality of the rock mass, the sub-horizontal attitude of the strata and the thin rock cover (between 2 and 20 m) in relation to the width of the tunnels (20 m each). These constraints make it difficult to form a resistant rock structure on the tunnel crown. The dominant geological formation consists of Belasian limestones and marls of the Cretaceous period (middle Albian-Cenomanian). The rock mass consists of an alternating, monoclinic series of compact limestone, grey limestone and marly limestone, marl and marly clays. The presence of expansive clays in the marly layers and karstic cavities in the compact limestones is highlighted. The stratification has a strike of N 50° E; 10-20° SE, being, as secondary structure of rock joints grouped into 3 families. The geological model also showed the presence of a fault which was intersected by the excavation. From a hydrogeological point of view, it is a mass consisting of impermeable or poorly permeable rocks, but which can exhibit significant percolation, of seasonal character through some of the karst fractures and cavities. The water levels, measured during the basic design phase comprised a piezometric surface located under the tunnel roof, in general parallel to the topographic surface, except for the fault zone, where a notable depression was produced which indicated that the fractured zone functioned as a drain of the rock mass.

- General description of the solution
The excavation method defined in the design for tunnels in general was of advancing in partial sections, placing in each one the primary support required for the full section. A specific excavation and support process was defined for each geotechnical zone, which was necessarily adjusted to the real geological conditions: ZG1 - reduced advances (1,0 to 2,0 m), with a pilot tunnel centered on the crown with a maximum width of 6 m; immediate execution of a shotcrete layer with an average thickness of 5 cm to prevent the disaggregation of the fractured limestone; execution of rock bolts on a mesh with a maximum spacing of 2 m and a length of 4 m; placing of welded wire mesh, fixed to the shotcrete; execution of shotcrete with a thickness of 15 cm; lateral excavation until the complete section of the crown was reached. The different excavation phases were phased to allow for the proper functioning of the primary support; ZG2 - reduced advances (1,0 to 1,5 m), with a pilot tunnel centered on the crown with a maximum width of 6 m; immediate execution of a shotcrete layer with an average thickness of 5 cm to prevent the fractured limestone and clay marl from breakdown; execution of rock bolts where the rock mass was sufficiently compact to allow their correct operation, positioned on a mesh of 1 to 2 m, with a length of 6 m fundamentally in the crown and in the north shaft; placement of steel ribs with TH 36 section, with 1,0 to 1,5 m spacing in transition zones to ZG3; placing of welded wire mesh, followed by the application of shotcrete with an average thickness of 15 cm; ZG3 - reduced advances (0,6 to 0,8 m), with a pilot tunnel centered on the crown; immediate placing of a shotcrete layer with an average thickness of 5 cm to prevent decompression of the rock and disintegration of the clay marl; placing of steel ribs with a TH 36 section, spaced at 0,6 to 0,8 m depending on the depth; placing of welded mesh between ribs; execution of shotcrete with an average thickness of 15 cm. At portals, in order to achieve a greater support of the excavated span and to contribute to the longitudinal confinement and to allow for the execution of the primary support in safer conditions, it was planned a forepolling umbrella arch consisting of injectable steel pipes, in holes with 0,6 m spacing between them and approximately 12 m long. For the execution of the south portal, strongly conditioned by the existence of dinosaur footprints and thin overburden, the construction of the side drifts was contemplated, in addition to the forepolling umbrella. The excavation process planned for the zone ZG3 was adopted.



Geotechnical zones (Queluz-Belas direction)



Excavation scheme for the geotechnical zone ZG2

- Major design constraints

The existence of a limestone layer of marly nature, which contains the footprints classified as of heritage interest and which could be affected by the tunnel is about 15 cm thick, covering an area of about 500 m^2. The base of this layer is a marl layer of poor geotechnical quality and very susceptible to weathering and erosion.

- Main construction aspects

During construction of the tunnels, the hydrogeological model envisaged in the design stage was fully verified. A few infiltrations with reduced flow rates were detected through the most important discontinuities, directly associated with rainfall periods. In the excavation, on front and on side attack, roadheaders excavators with a power of about 300 kw were used. The procedure adopted, in accordance with the philosophy of the conventional method of tunnel construction, was to analyse the behaviour of the rock mass through time, carefully observing the progress of the displacement curves as result of the excavation progress and the execution of the primary support and adapt the excavation methodology and the primary support to the real conditions encountered. On the eastern half, the tunnels were excavated on a slightly deformable and very sensitive rock mass, especially in the crown. On the western side, the rock mass was more deformable and, consequently, monitoring of its response was intensified. Daily monitoring records were analysed in real time and the geological model was studied in the greatest detail in order to take the most efficient measures to guarantee the stability of the full section of the galleries. The footprints were also moulded with a latex film and were kept in reserve at the Lisbon Natural History Museum for possible future restoration. The several tracks identified were covered with soils of different grain size, placed on a geotextile mat as a protection measure.

Photo of underground excavation for geotechnical Zone ZG3

Portal overview

Overview of the Western Portal

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2018

ALGIERS METRO. EXTENSION A. UNDERGROUND WORKS

General features

Location: Algiers, Algeria – Year of completion: 2018 – Purpose: Metro
Type: deep excavations and underground works
Soil: sericite schists
Owner: Entreprise Metro d'Alger - Designer: Cenor (presently TPF) - Technical assistance during construction: Cenor (presently TPF)
Contractor: GMAC (Andrade Gutierrez, Teixeira Duarte, Gesi TP, Zagope)
Awards, distinctions: Ricardo Esquível Teixeira Duarte 2018 Award. SPG.

Work description

- Main purposes

The rapid development of the Algerian capital (which nowadays presents around 4M inhabitants, a number that doubles if we consider its metropolitan area) led to the execution of the first stretch of the Algiers Metro network, the second on the African continent. This segment was opened to public in 2011, but it was not until 2018 that the operation its Extension A began, spreading the existing network by approximately 1.75 km towards North and adding two stations, Place des Martyrs and *Ali Boumendjel*, to the ten that initially formed Algiers metro system, as well as three ventilation shafts: PV1, PV2 (integrated into the second station) and PV3 (already partially executed, as part of a previous contract). In addition to the execution studies of both temporary works and definitive structures, it is important to mention the Technical Assistance during construction which was provided by Cenor (presently TPF) technicians, a critical success factor of such *sui generis* works as the geotechnical ones, especially in such delicate contexts.

- Geological and geotechnical conditions

The geological-geotechnical environment in which the Extension A of the Algiers Metro is developed comprises rock masses essentially composed of sericite schists with dispersed and non-continuous limestone intercalations, which occur under alluvium and anthropogenic deposits with a maximum thickness of around 7.0m and to which a suspended groundwater level is associated. The material of metamorphic origin presented a significant variability in terms of geomechanical characteristics and, for that reason, it was subdivided into three distinct classes: RC1, a rock mass of average quality (W2-3, σ_{ci} from 40 to 80MPa and GSI from 34 to 52), CR2, a rock mass of average to poor quality (W4-5, σ_{ci} from 15 to 30 MPa and GSI from 23 to 34) and CR3, a rock mass of very poor quality (W5, σ_{ci} of around 5 MPa and GSI of 10).

Sericite schists: during investigation and during tunnel excavation

- Other constraints

Among the main constraints to the project one can highlight the traffic and the dense surface occupation, predominantly consisting of century-old buildings (dating from the French occupation period or even the medieval era) in advanced degradation state, not only as a result of their age, but also due to the lack of maintenance and to the intense seismic activity that affects that region. On the other hand, it is also worth to mention the fact that, during the open-air excavation of the Place des Martyrs Station (initially conceived as a cut-and-cover work), several archaeological remains were found, which triggered a profound reformulation of its conception and led to the engagement of an underground excavation procedure, being its design developed in parallel with the onsite advancement of the excavation works. Finally, the impossibility of using many potential works fronts (whether due to their location, small size, or position along the critical path of the contract workplan), in association with the significant constraints to surface traffic, led to the need of designing several additional fronts to launch underground works, so that it was possible to meet the imposed challenging deadlines.

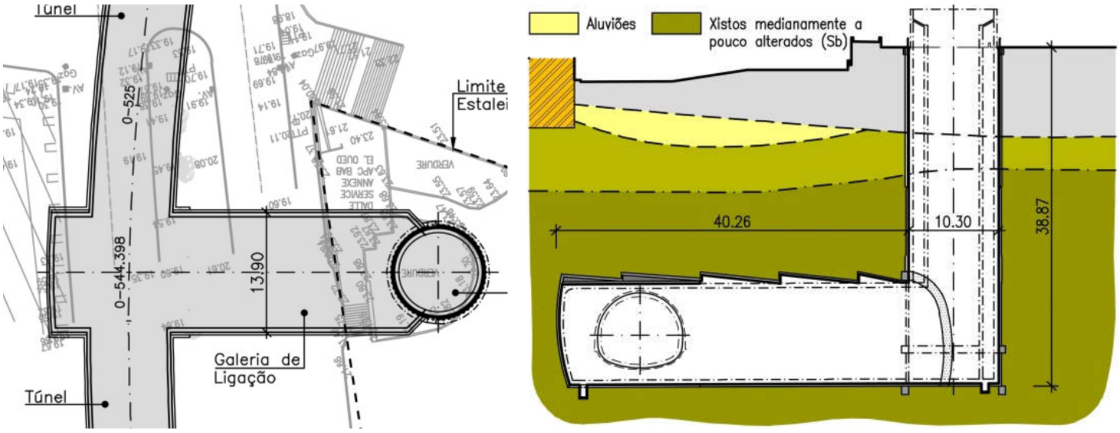
- General description of the solutions

Track tunnel

The track tunnel presented a cross section with 9.0m and 10.0m of height and width, respectively, and was excavated (in full face or top heading and bench) in accordance with the NATM procedure, employing fiber or electrowelded mesh-reinforced shotcrete, applied in layers of different thicknesses, in association with the installation of Swellex rockbolts with variable spacing or steel ribs. The eventual execution of temporary inverts was recommended as well as the use of other auxiliary support and drainage devices, such as steel or fibreglass rockbolts, injected tubular forepoles and deep geodrains for local phreatic drawdown.

PV1 Ventilation shaft

The PV1 Ventilation shaft has a circular geometry, with an internal diameter of 10m and a depth of 39 m. A 35 m-long and 14.5 m-wide underground gallery underground gallery leading to the tack tunnel was excavated from its base.

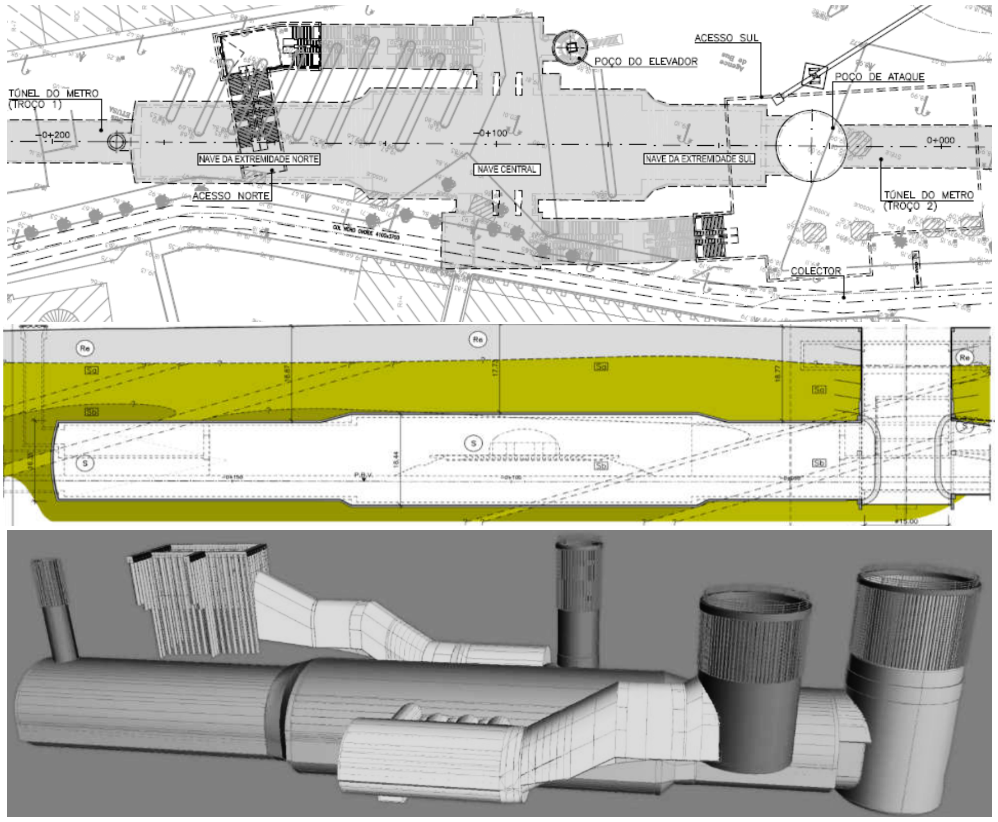


General plan and longitudinal section of the PV1 shaft

The shaft was excavated according to the principles of the NATM methodology by means of the installation of HEB180 profiles in Ø250 mm holes (properly sealed with cement grout) near the surface, where materials of fewer resistance and deformability where found, as well as of the application of a 0.15 m-thick shotcrete lining reinforced with two layers of electrowelded wire mishes. This thickness increased to 0.30 m in the lower half of the shaft. A reinforcement ring was also installed above the intersection with the gallery connecting to the tunnel by thickening the above-mentioned shotcrete lining, and a reinforced concrete edge beam was also executed along its perimeter. In turn, the primary lining of the underground gallery included the use of HEB180 steel ribs embedded in a 0.30 m-thick shotcrete layer, being its pre-support ensured by means of the installation of tubular injected forepoles.

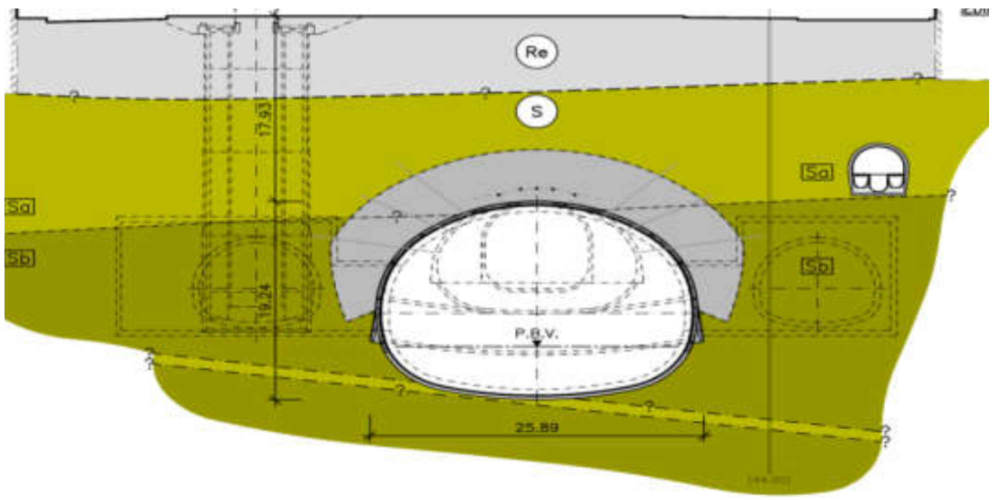
Place des Martyrs Station (SPM)

The Place des Martyrs station was characterised by a very complex geometry, presenting a 144 m-long main body which was divided into a central segment and two top stretches, with 26.0 m or 18.4 m of width and 19.2 m or 16.4 m of height, respectively, two accesses and a ventilation shaft, which ended up being used as an attack shaft, both to the main body of the station (through an exploratory gallery) and to the track tunnel itself. It is important to remark that the average overburden height of the station, of approximately 18 m, was smaller than its cross section equivalent diameter and comprised mainly weathered and fractured schists, which meant that any small local instability could rapidly progress up to the surface, with potentially catastrophic consequences. The excavation was carried out under a 0.40 m-thick lining of reinforced shotcrete associated with the installation of HEB240 steel ribs and considered the execution of temporary inverts in all excavation phases (pilot gallery, top heading and two benches).



General plan, longitudinal section and 3D perspective of the Place des Martyrs Station

It is also worth to mention the fact that the aforementioned exploratory gallery (excavated along the entire length of the station) was used to pre-treat the surrounding rock mass with cement grout injections, as well as the need of simultaneously installing fibreglass injected radial rockbolts, as a way of ensuring rock mass stability near the lateral enlargements of the top heading (its location was adjusted during the construction phase in order to match the stretch where the occurring materials presented better quality), from which the tubular injected forepoles necessary for the pre-support of the subsequent front enlargements where installed.



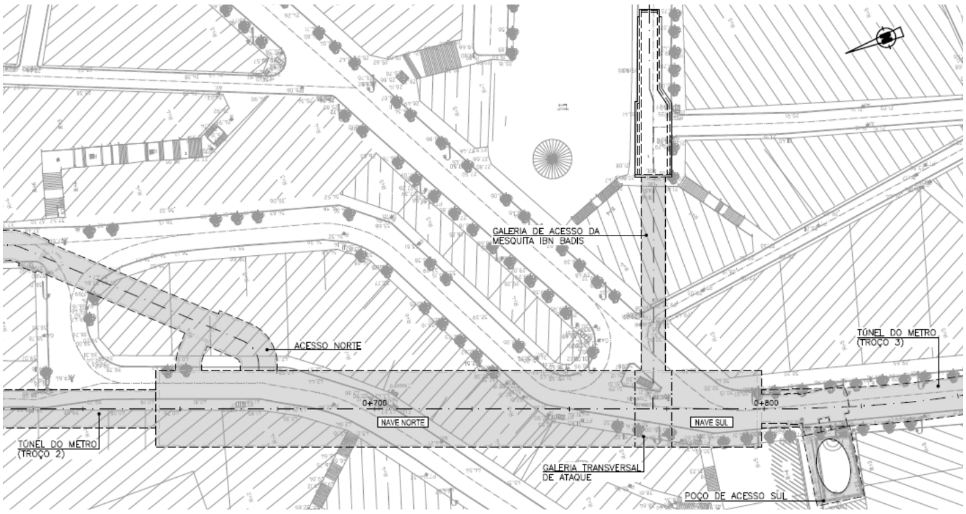
Cross-section of the Place des Martyrs Station

Access shaft, top heading and final section construction of the Place des Martyrs Station

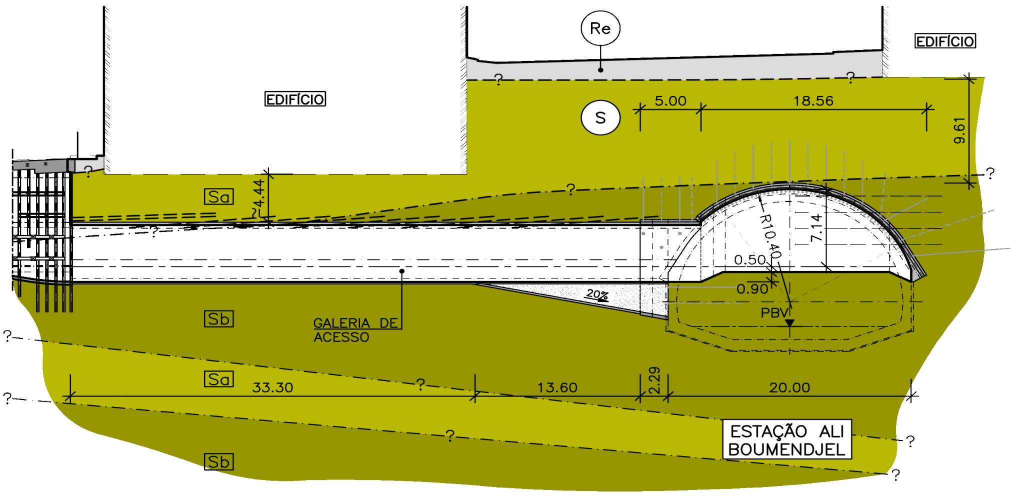
Mention should also be made to the excavation works of the accesses, which included several intersections with the main body which were very close to each other, leading to the definition of a detailed excavation sequence that included, among other particularities, the excavation of specific galleries for the execution of the final structure columns before the excavation of the accesses themselves.

Ali Boumendjel Station (SAB)

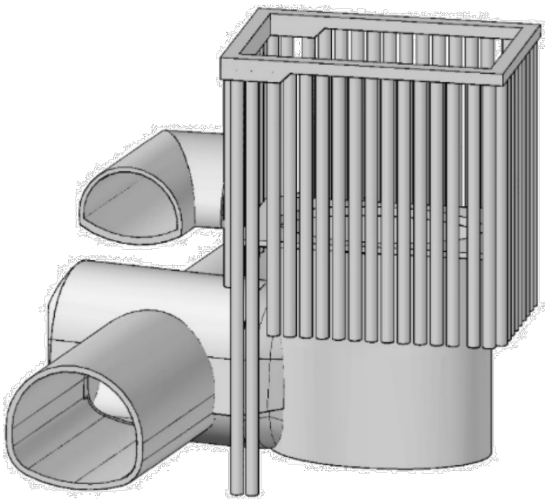
The Ali Boumendjel Station, which presented a 155 m-long, 20.5 m-wide and 14.5 m-high main body gallery, was excavated under a 12 m to 23 m overburden height, in a densely occupied urban area. Considering its relative location with regards to other Extension A works, it was of upmost importance that the underground works were executed from one of the accesses to the station main body. Ibn Badis Mosque access, which after an initial cut-and cover stretch consisted of an underground gallery (under an existing building of high heritage value), was the chosen one.



General plan of *the Ali Boumendjel Station*



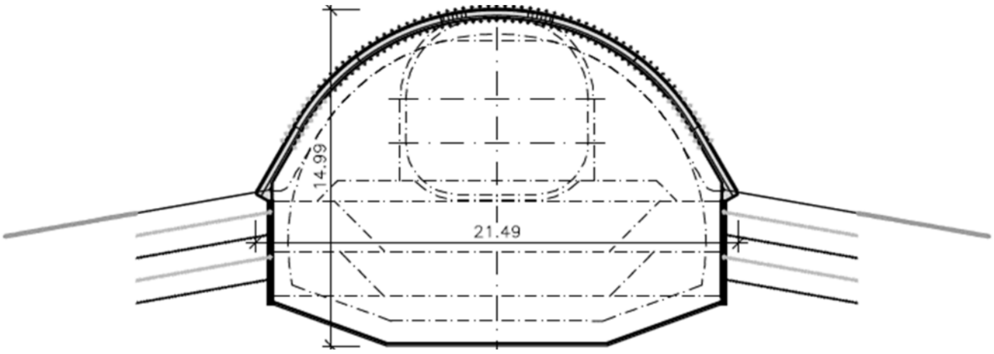
Longitudinal section of the access to the mosque of the Ali Boumendjel Station



South Access to the Ali Boumendjel Station

This underground gallery ended up being over-excavated at the level of its invert, as a way of materializing a ramp that would allow the use of this work front during the excavation of the three benches of the station. Its alignment, perpendicular to the station, and the reduced dimensions of its cross section led to an unusual solution in contexts such as the one described, which consisted in the "immediate" materialization of a narrow slice of the station's top heading. On the other hand, the complex geometry of the intersection, combined with the subsequent need of demolishing part of the initial cross section, led to the choice of a primary support consisting of steel fiber reinforced shotcrete in association with steel rod or fibreglass rockbolts, both in the final section of the access gallery and in the small attack stretch of the main body.

However, it should be noted that the primary support of the mentioned attack gallery included the installation of HEB240 steel ribs embedded in a 0.4 0 m-thick shotcrete lining, which allowed the accommodation of stresses rotation phenomenon, expected at the beginning of the longitudinal excavation of the main body. After the excavation of the top heading (preceded by the execution of an exploratory gallery, in the case of the Northern stretch, or a side-drift in the case of the Southern one) which presented a primary support consisting of the aforementioned HEB240 steel ribs and 0.40 m of reinforced shotcrete, the robust elephant's feet of the foundation of its sidewalls were anchored to the rock mass by means of ground anchorages prestressed at 600 kN and only then the bench excavation was initiated. In order to optimise the excavation sequence and to take advantage of both the nature of the materials and the geometry of the gallery itself, the bench excavation was divided into three horizontal levels and presented vertical sidewalls, which were supported by means of a 0.1 5 m-thick steel fiber reinforced shotcrete associated with the installation of self-drilling rockbolts slightly prestressed to 30 kN (in order to ensure their active operation, i.e., without the need to mobilise large displacements).



Cross-section of the Ali Boumendjel Station

Pilot tunnel and final section of the Ali Boumendjel Station

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