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Rehabilitation of La Nerthe Tunnel on Paris-Marseille High-Speed Railway Line

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REHABILITATION OF LA NERTHE TUNNEL ON PARIS-MARSEILLE HIGH-SPEED RAILWAY LINE

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ABSTRACT

The La Nerthe tunnel is located on the Paris-Marseille railway line under the Étoile mountain range. This 4638 m long double track tunnel was built at the middle of 19th century under a maximum cover of 180 m and currently is used as part of the high-speed railway line (TGV). This masonry tunnel with hard limestone blocks at the sidewalls and bricks at the crown passes through marl, gypsum and limestone zones. A major fault with accompanying extensive tectonic deformations cuts across the tunnel axis. The in-situ stress tests performed on the new high-speed rail line in the zone with intense tectonic fractures highlighted the tectonic residual stresses with very high lateral earth pressure coefficients (k_0). The La Nerthe tunnel underwent ovalization deformation with excessive displacement at the springline, and pinching and bursting of the bricks at the crown.

In this paper, the geological context and the damage to the tunnel lining are described in detail. The theoretical tunnel profile has been compared with the actual real section in order to determine the size and the shape of the lining deformation. The initialization of the existing stress field in the tunnel masonry lining using the original excavation and support methods and also a simplified approach for considering the time effect has been explained in detail. Finite element analyses have allowed an understanding of the possible causes of the observed damages and to propose an adequate reinforcement method. The applied rehabilitation system and the constraints of work in this busy mainline railway tunnel have been discussed in detail.

INTRODUCTION

The tunnel of La Nerthe, located on the railway line from Paris to Marseille under the Étoile mountain range, has double electrified tracks and is entirely on tangent alignment. Built in the mid nineteenth century, it underwent localized deformations and structural damages, requiring French railway (SNCF) to study a rehabilitation program.

This paper points out the principal characteristics of the tunnel, then describes the geological context of the ground and presents the stages of the design of a system of reinforcement allowing the stabilization of the portions of this tunnel having undergone structural damages.

THE LA NERTHE TUNNEL

The tunnel of La Nerthe with a length of 4638 meters cuts through the mountain range of Etoile at the north of Marseille (Figure 1) with a maximum cover of 180 meters. It was built from 1843 to 1848 from the two portals and 24 intermediate shafts. This underground structure crosses the marl and gypsum of Jurassic and the marl and limestone of the Cretaceous from the station 0 at Paris portal to the station 1930 at Marseille side.

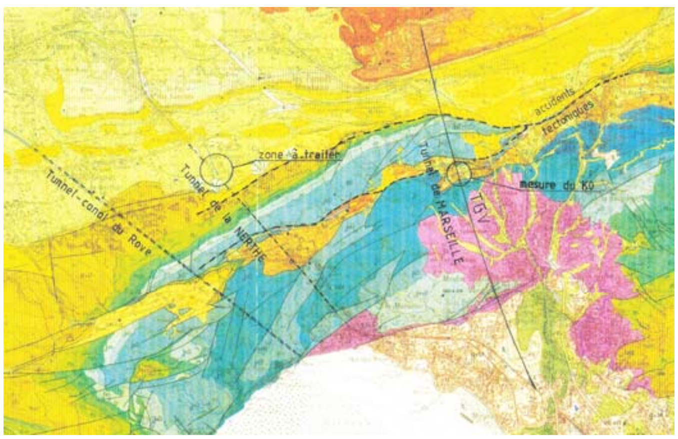

Fig. 1: Location of the tunnel and surface geological map

The theoretical cross section of the tunnel (Figure 2) is a section with multiple radiuses releasing a height under crown of approximately 7.50 meters with respect to the base of the rail and an opening between the geometrical start points of the vault of about 8 meters. The average thickness of the sidewalls built from coursed ashlars and the vault constructed from brick is approximately 0.70 meters. The invert composed of coursed ashlars has a thickness of about 0.35 meters.

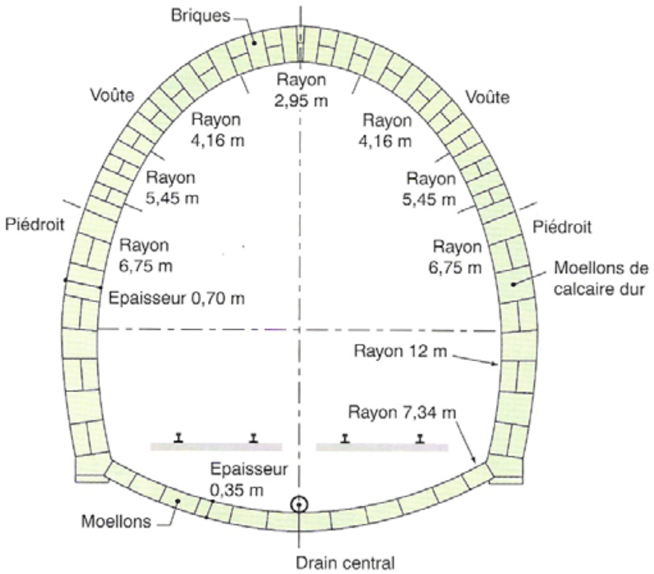

Fig. 2: Typical section of the La Nerthe tunnel

This tunnel underwent, with the passing of years, structural damages of which most characteristics were concentrated on the first thousand meters. Pronounced deformations in shoulders with pinching and bursting of bricks at crown key were observed in several zones in particular those of the station 740 to 765 and station 835 to 845 (Figure 3).

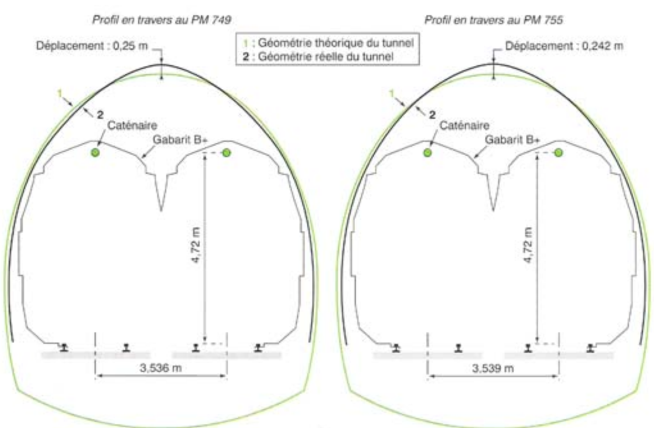

Fig. 3: Initial and current geometry of the tunnel

GEOLOGICAL SETTINGS

In the two analyzed zones, the mined formations belong to the Bégudien tier of the upper Cretaceous facies of Provence. These formations consist of alternations marl-limestone with intercalations of conglomeratic and lignitic levels.

In a distance of approximately 300 meters of this portion of the tunnel, towards Marseille, a major tectonic accident lapping of a dip of 40 degrees towards the south and direction NE-SW, cuts the axis of the tunnel. The limited available information on this part of the underground structure showed a sizeable tectonization that must have resulted in the existence of residual tectonic stresses.

The investigations carried out on the alignment of the Mediterranean high speed line (TGV) in the vicinity of the La Nerthe tunnel showed that an intense tectonic affects all these geological levels, with fractures, tight and buckled folds, overlapping scales, layers turned to vertical, glossy surfaces and sliding mirrors. Tests of hydraulic fracturing were carried out in 1994, in the commune of Cadenaux, in a zone of intense tectonic fracturing with appearance of Trias. These tests highlighted residual stresses of tectonic origin with a probable North-South orientation, and a high coefficient of at-rest earth pressure (K_0) with some values reaching 5.

These tests were carried out in Jurassic formations at more than 1000 meters of the northern portal overlapping on the upper Cretaceous. However, the final report recommended to extend these residual stresses to all the strongly tectonized zones including the northern part of the tunnel of Marseille in the upper cretaceous (Figure 1). These formations, in géotechnical point of view, are the same as those met in the zones of rehabilitation of the La Nerthe tunnel.

GEOTECHNICAL CHARACTERISTICS OF THE GROUND

The only available report on the La Nerthe tunnel relates to the laboratory analysis of ground samples taken in three cored borings carried out in 1978 at the station 780. According to this report, the ground of the La Nerthe tunnel is made up in major part of very jointed sandy marls.

These marls are slightly swelling. The pressure of swelling is 20 kPa. The swelling fraction (montmorillonite) is 15 % to 20 %. It is obvious that this value of swelling pressure cannot generate considerable deformations. In addition, during last visits carried out in this tunnel, the two zones previously described were practically dry. It can be said that in most the real cases of swelling in a tunnel, a heave at the platform level is usually observed and not the large deformations in the vault. This can be explained by the downward water flow by gravity in the direction of the invert. The mechanical properties of soil given in this report are presented in Table 1. They correspond to the rehabilitated zone, where the cover of the tunnel is approximately 32.5 meters.

For the analysis presented in the following section, the Poisson ratio of the ground was taken equal to 0.3 and its Young modulus Es = 31.75 MPa. The Es value was deduced from the relation between the void ratio and the normal pressure in the report mentioned above [$E_s = (1 + e_o)\frac{\Delta p}{\Delta e}$]. The coefficient of earth pressure at rest K_0 in the tunnel cross section was fixed at 2 based on the results of hydraulic fracturing tests in borehole and in order to take into account the tectonic stresses.

Table 1. Soil properties around the La Nerthe tunnel

Parameter	Value
Unit weight, γ (kN/m^3)	23
Compression index, C_c	0.057
Internal friction angle, φ (degrees)	30
Cohesion, c (kPa)	250

Due to the lack of direct tests in the tunnel, the mechanical properties of the brick masonry and coursed ashlars (unit weight, Young modulus, Poisson ratio, compressive strength and tensile strength) were taken from the published literature and are given in Table 2.

Table 2. Mechanical properties of the masonry liner

Parameter	Brick	Ashlars
Young modulus, E (MPa)	6200	25000
Poisson ratio, ν	0.2	0.2
Compressive strength, R_c (MPa)	10	20
Tensile strength, R_t (MPa)	1	2
Unit weight, γ (kN/m^3)	22	25

EXCAVATION METHOD OF THE LA NERTHE TUNNEL

Based on the construction documents in our possession entitled "General instructions on the excavation and support of the La Nerthe tunnel", the excavation approach used was the English method. The full section was mined in three levels densely supported by timber members (Figures 4a and 4b). Then the liner was installed, by initially carrying out the invert, then the sidewalls and finally the vault.

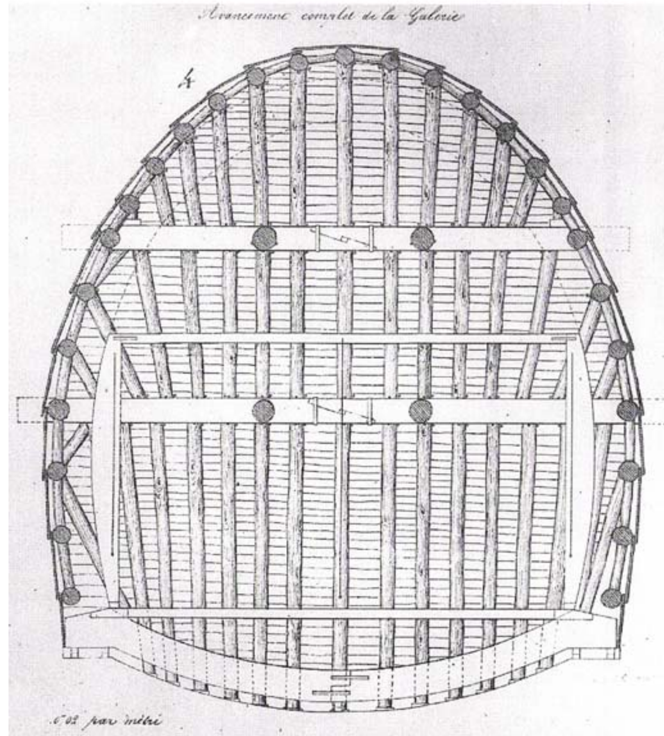

Fig. 4a: Timer support of the excavation face

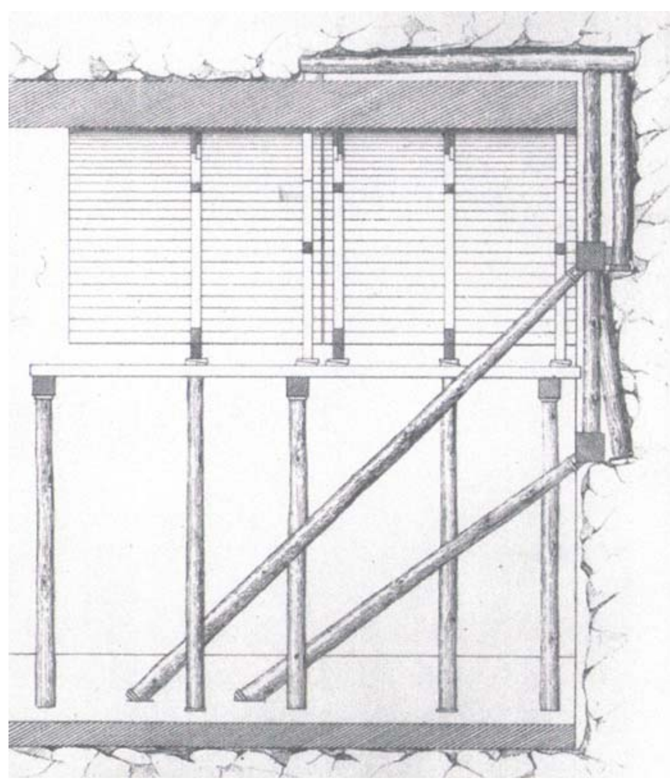

Fig. 4b: Longitudinal timber support with the completed masonry ring

Fig. 4b: Details of the La Nerthe tunnel construction

FINITE ELEMENT ANALYSIS

The problem was studied using the convergence-confinement approach and the finite element method. The finite element software used was the computer program CESAR-LCPC developed by the Laboratoire Central des Ponts et Chaussées. It allows taking into account the construction phases and the elastic and elasto-plastic behavior of the ground and the tunnel liner. The analysis included three phases corresponding to the excavation sequence, the installation of the liner and the long-term creep effects.

Phases of the analysis

In the first phase, the excavation of the tunnel was simulated. Considering the density of the timber support and its distance from the excavation face (approximately 5 m), the coefficient of deconfinement was taken equal to 0.4. It was assumed that the marls have an elastic perfectly plastic behavior, for which the Mohr-Coulomb criterion was used.

The second phase related to the modeling of the support installation. The remainder of the déconfinement force was applied to the extrados of the liner. The parabolic criterion was used to represent the elasto-plastic behavior of the brick and coursed ashlars masonry.

The long-term behavior of materials was taken into account in the third phase. The method used was a simplified approach; rigorous only in the case of linear viscoelastic materials. It consisted in simulating the change of mechanical characteristics of materials in long-term.

Figure 5 shows the finite element mesh used for the analysis. Because of the symmetry of the geometry, loading and boundary conditions, only half of the system was modeled. Also because of symmetry, the degrees of freedom of horizontal displacements along the axis of symmetry were fixed. In order to eliminate the boundary effect, the soil mass was modeled over a distance equal to four times the width of the tunnel in the horizontal direction and four times its height under the invert. To ensure the precision of the results, a sufficient number of finite elements layers were used in the two directions.

Analysis results and interpretation

Figure 6 presents the liner deformation resulting from the third phase of analysis. As it can be noted, the shape of the deformed section and the order of magnitude of displacements correspond to reality (Figure 3). The principal stresses at the corner nodes of the liner elements are presented on Figure 7. High value of the stresses between the crown and the shoulders and a less compressed zone between the shoulders and the sidewalls towards the intrados of the liner can be observed, which leaves a certain room for maneuver for the reinforcement in this zone. The contours of the plastic deformations are shown on Figure 8. A significant plastification occurs at the level of the intrados of the crown (because of the compression) and at its extrados (because of the tension). That is in agreement with the observations on the ground.

DESIGN OF A REINFORCEMENT SYSTEM

Figure 7 shows that the zone located between the crown and the shoulders was subjected to dominantly horizontal forces. To reinforce it, a mass of reinforced shotcrete supported on the shoulders of the vault was added to the existing masonry liner. Since the uniaxial compressive strength of the shotcrete is at least twice the one of the masonry, in the most conservative design (all the liner stresses are transferred to the reinforcing layer), the thickness of the required reinforcement will be 0.35 meters.

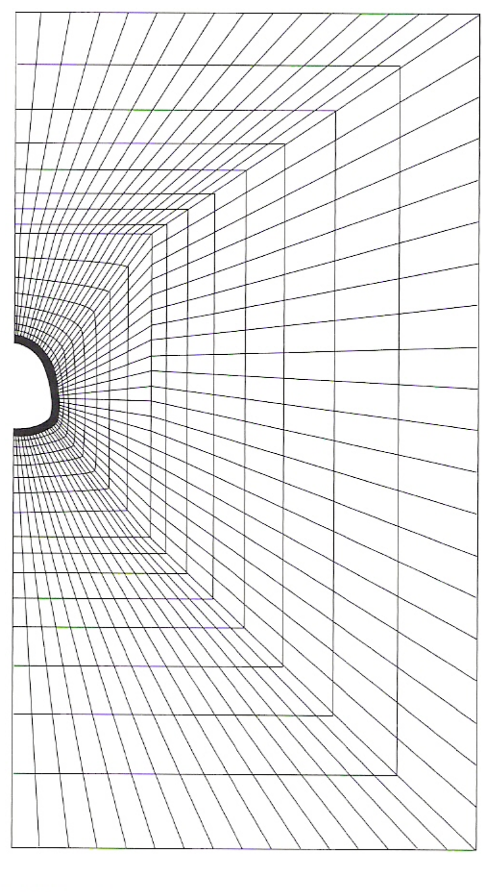

Fig. 5: Finite element mesh used for the analysis

First a portion of the existing brick liner at the shoulder area was cut and then a wire mesh was installed to reinforce the shotcrete and allow upward spraying. The mesh was sitting on the cut section of the liner and was hung from the roof by a grid of anchors at regular spacing (Figure 9). The next step consisted of filling the gap between the wire mesh and the existing liner with shotcrete while providing sufficient cover for the reinforcing mesh. In order to avoid the excessive stress concentration at the cut portion of the shoulders, the full cycle of existing liner cutting, wire mesh installation, and shotcrete filling was performed over alternate panels with limited length along the tunnel axis (Figure 10). Figure 11 represents a picture of the completed rehabilitated tunnel section.

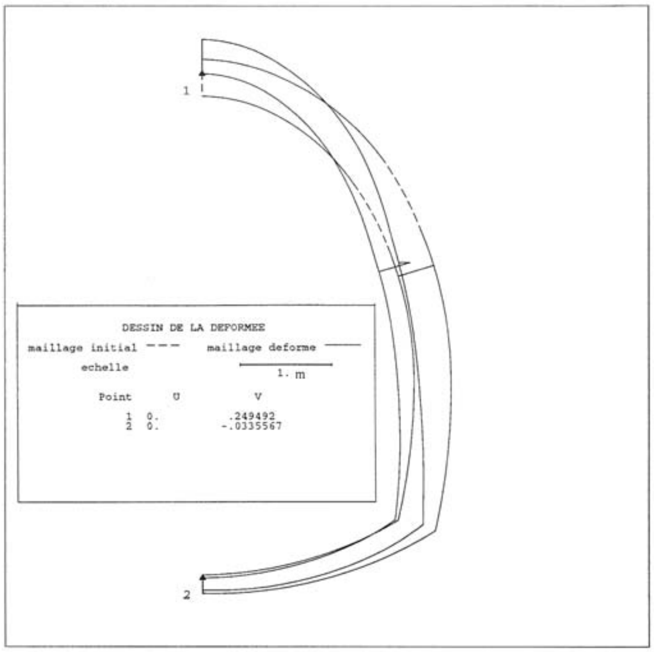

Fig. 6: Liner deformation at long-term

CONCLUSION

This paper presents an example of rehabilitation analysis and design for an old tunnel (La Nerthe tunnel). The theoretical cross section was compared to the surveying data of the current real section of the tunnel in order to determine the order of magnitude and the shape of the liner deformation. The results of hydraulic fracturing tests were used to evaluate the effects of the tectonic residual stresses in the zone of the structural damages. The modeling of the phases of construction and the long-term creep effects using the finite element method allowed to analyze the possible causes of the observed damages and to propose an adapted method of reinforcement.

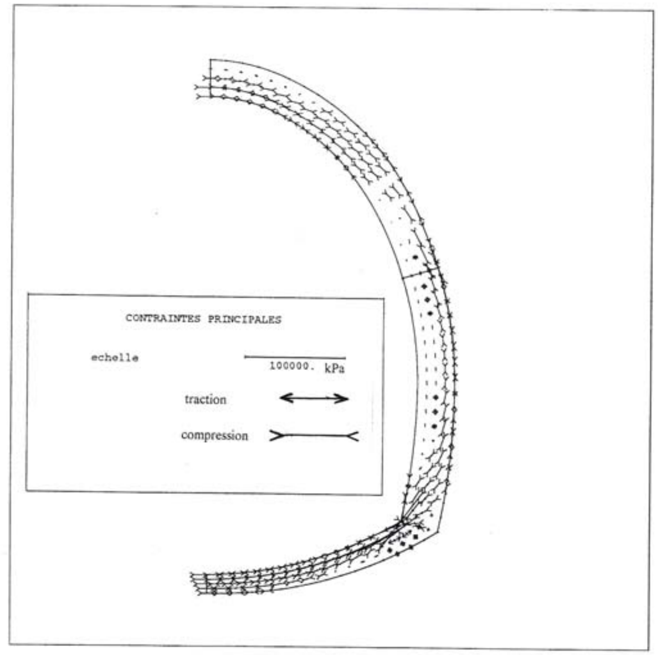

Fig. 7: Principal stresses of the liner

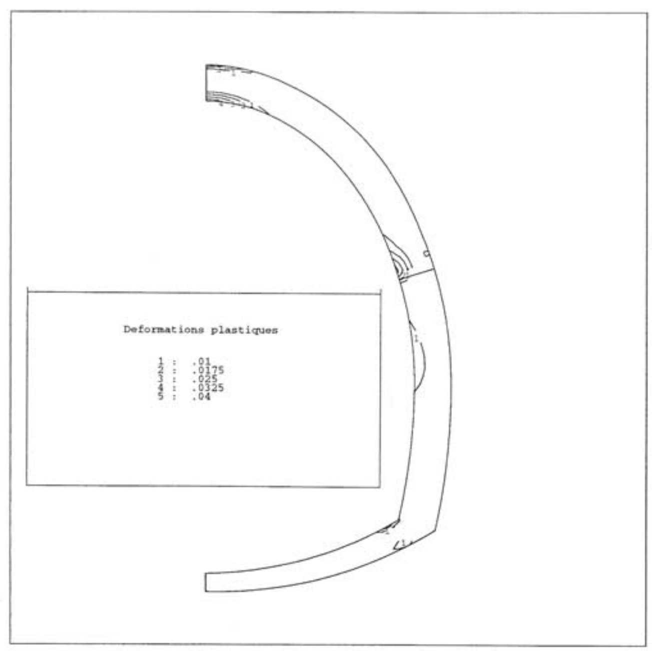

Fig. 8: Contours of the total plastic deformations

Fig. 9: Installation of the wire mesh

Fig. 10: Cutting the existing liner and installing the reinforcing support in alternating panels

Fig. 11: Completed reinforced tunnel section

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