



Numerical modeling of the pathological case of a damaged tunnel application to Djebel El-Ouahch tunnel (east–west highway)

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Abstract

This work deals with a numerical analysis of a pathological case of a part of tunnel T1 of Djebel El-Ouahch which suffered a collapse. The pathological analysis of state of stress and state of deformation in the soil and in the tunnel wall was examined. Numerical modeling is performed in 3D space using the PLAXIS 3D TUNNEL calculation code which is based on the finite element method; the behavior of the soil is described by the elastoplastic model of Mohr–Coulomb. The results are presented in terms of settlement at ground free surface, vertical displacement under the tunnel floor, stresses, and deformations developed at the tunnel retaining level and in the ground around retaining to be able to analyze the causes of the collapse.

Keywords Pathological · Settlement · Displacement · Collapse · Effective stress · Interstitial pressure

Introduction

Underground construction, especially in the case of urban tunnels, poses specific risks during all stages of the project and especially during construction and operation. The main risks encountered in the construction and operation of such underground structures are: (Grasso et al. 2004).

- Geotechnical and geological risks which are related to the insufficiency of information obtained through the reconnaissance campaign, and to the ability to predict the response of the ground to the digging action.
- Hydrological risks which are associated with the insufficiency of the information collected with regard to underground hydrology in the project area.
- Study risks that are mainly related to the difficulty of the project to adapt to the geomechanical conditions actually encountered the lack of construction.

- Construction or excavation risks related to the choice of inappropriate or poorly controlled construction method, instability phenomena, the experience of the constructor's team and contractual constraints.

We can distinguish several risks related to instabilities in underground structures such as (Oggeri and Ova 2004)

- The collapse: corresponds to a structural break, partial or total, of the tunnel, for example, the collapse of the metro station (Daikai, Japan, 1995), the collapse of the head of the Patra tunnel (Greece, 1998); according to numerous studies, it has been concluded that the fall of blocks (collapse) results from aging, land degradation and tunnel support failure (Kerisel 1975; Levy 2000).
- Local disorders: they include several forms of local instability in underground structures, such as falling rocks, cracking, weak water infiltration, damage to the walls and roof of the structure, and uplift to write off.

The tunnel (A38-Saltash, United Kingdom, 1997) is a good illustration of this problem, disorders of the Galibier tunnel (France, 1996).

- Flooding: this is a consequence of the invasion of an underground structure by a large amount of groundwater or by a large amount of rainwater; these phenomena lead to significant material losses and work stoppages in the surrounding areas. Works affected underground.

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The flooding of the road tunnel of Wushantou, Japan, 1992, is a good example.

- High deformation (extreme convergence): this is the reduction of section associated with strong convergences which compromise the use of the structure under conditions of optimal security. The strong deformation can result from several factors such as swelling, creep, plastic deformations, and tectonic stresses. The consequences of convergence are the closure of the section and the destruction of the support. Sometimes this phenomenon requires the re-excavation of the underground structure. The Tymfristos road tunnel in Greece is a good example.
- Surface settlement or differential settlement: the excavation of urban underground structures in soft terrain often results in a settlement of the soil above the tunnel, which can damage surface infrastructures.

These different types of instabilities are related to the conditions (idris 2007): geotechnical, geological, hydrological, and technical of the underground project.

In this article, we make the pathological analysis of the point of view of the state of stress and state of deformation in the ground and in the retaining of a tunnel part T1 which suffered a collapse of straight tube and a very important damage of the second tube terraced.

The objective of this work is to establish a three-dimensional numerical model of the collapsed tunnel part to try to see the causes and consequently find solutions to overcome this damage.

Presentation of the T1 tunnel of Djebel El-Ouahch

The T1 tunnel is part of the realization of the Maghreb Unit Highway (AUM) of about 7000 km in length, crossing Algeria with a length of 1200 km. The tunnel is part of section 1 of this highway, and it crosses Djebel El-Ouahch northeast of the city of Constantine. The motorway tunnel consists of two nearly parallel tubes with a total length of 1909 m, the right tube starts at kilometric point (PK: 205 + 407.5) and ends at point (PK: 207 + 316.5) and for left tube starts at (PK: 205 + 393) up to (PK: 207 + 302), the tubes are separated by a distance of 24 m. The cross-sectional dimensions of each tube around the theoretical excavation line are 16.92 m in width and 11.66 m in height.

The maximum overburden of the tunnel is about 119 m and the most critical section corresponds to the lowest overburden is 17.50 m. Figure 1 illustrates a general view of two tubes of tunnel T1.

The route of the Autoroute Est Ouest (Fig. 2) (section 4-2-2) crosses very prominent terrain. The geology of the



Fig. 1 General view of two tubes of tunnel T1

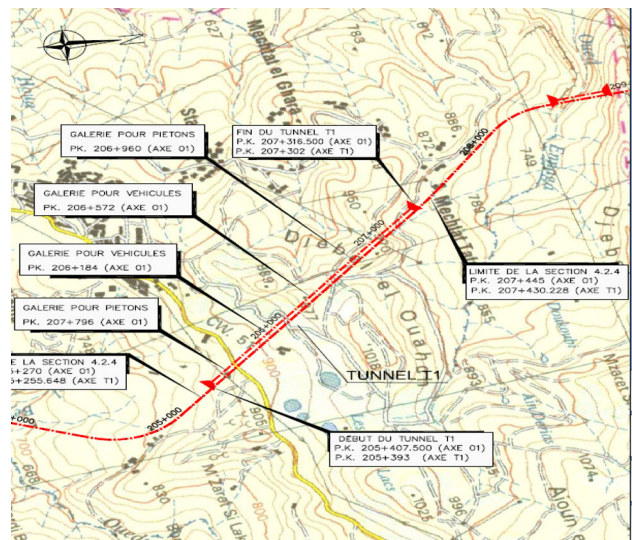


Fig. 2 Layout of the tunnel T1

study area is characterized by the presence of a series of faults fracturing the eastern sector developing tectonized formations. Indeed during the reconnaissance campaign by the cored holes, the Djebel El-Ouahch massif is characterized by a geology which proved to be noticeably very complex. The formations encountered on site correspond to sandstones, marls and argillites; sometimes they come in alternately (Fig. 3). The roof of these formations is surmounted in most cases by an alluvial formation. Laboratory tests indicate values that confirm the nature of the formations encountered. The values, whose formation appears to be of low consistency and low shear strength, generally correspond to the weathered zones.

The hydrological study is carried out to locate the aquifer levels and the shape of the possible aquifers, the presence of aquifers requires a quantitative in situ study to determine the characteristics of the terrain (permeability, porosity, etc.) which are essential to provide for the execution of a number of wells necessary for the drawdown of the sheet. In our case, the circulation of the groundwater

Fig. 5 The state of place of the structure after the collapse (left tunnel liner damage)



Fig. 6 The state of place of the structure after the collapse (lifting of ground above left tunnel, and destruction of roadway)



and water content and is accompanied by a change in specific volume.

Understanding the swelling phenomenon

During the study of the problem of swelling encountered on the structures, one realizes immediately that the swelling of the ground depends not only on the inflow of water from the external environment but also on the nature of the ground.

The International Society of Rock Mechanics Swelling Rock Commission (ISRM, 1983) has defined the swelling phenomenon as: “Swelling is the result of a combination of physicochemical reaction involving mainly water and water. Modification of the state of constraint”.

The phenomenon of swelling depends on the mineralogical characteristics of the clay material, the nature of the hydration solution and the state of stress.

A mineralogical description of clays is, therefore, essential.

Mineralogy of clays

In nature, the clays are very abundant, they cover about 42% of the volume of the earth’s crust, and they result from the decomposition of the siliceous rocks by physical and mechanical disintegration, then by chemical alteration. They can be plastic or stiff, we distinguish:

1. *Plastic clays* which are soft and very deformable,
2. *Stiff clays* which are indurated and show a more fragile than ductile behavior beyond the elastic limit, this is due to the presence of carbonates and quartz.

The water–clay system

The bonds that can be at the origin of the swelling phenomenon are those involving water molecules, knowing that this water is of different natures. There are three types:

- The water of constitution: it enters the chemical composition of the sheets, it is integrated in the crystal lattice which is strongly bound to the molecular structure, to break this connection it is necessary to bring a considerable energy, and by heating the clay has more than 300 °C.
- The interfoliar water: responsible for the swelling, which can be extracted by heating in an oven at 100° C but remains difficult to drain,
- Interstitial water: it is localized between the clay particles, and can be loaded and act on the physical properties of clays such as the limits of Atterberg.

Intraparticulate and interparticulate swelling Natural clay soils, clay rocks, marls, and composite rock anhydrite are subject to swelling. We can distinguish two types of swelling within clays:

The interparticle swelling It results from physicochemical phenomena between a clay particle and water in the peripheral space of the particle.

The water does not penetrate between the sheets of clay, from which one has the interparticulate swelling.

This mechanism affects all clays, but may be of a rather limited magnitude.

Intraparticulate or interfoliar swelling it is due to the introduction of water molecules inside the clay particles, between the elementary sheets. The nature of the swelling mineral and its primary structure has a great influence on the characteristics of this swelling.

The water penetrates inside the particles and is organized in mono-molecular layers from which one has the intraparticulate or interfoliar swelling.

Interfoliar swelling occurs at the smallest scale of the clay structure but can be very large.

The type of swelling depends mainly on the swelling mineral and its primary structure.

Mechanism of swelling

To understand the swelling, it is necessary to understand the swelling physico-chemical and mechanical without forgetting that the texture of the material that is to say the organization of platelets also intervenes on the swelling mechanism.

Multiple physicochemical interactions can occur between water and clay. Understanding the swelling of

clays is based on the physico-chemical theory of the water–clay system called “electric double layer theory”.

This theory was devised by Helmholtz, and then formulated by Gouy and Chapman then by Stern.

Experimental investigation in the laboratory

Depending on the objective chosen during the study of the swelling, one can determine the mechanical parameters necessary to characterize the swelling grounds. In everyday practice, we opt for a macroscopic approach distinguishing three notions:

Swelling pressure

Swelling pressure of a soil or rock element having a known initial physical state, it can be defined as the state of stresses to be exerted to maintain the constant volume during imbibition without distortion.

Free swelling

This translates the maximum deformation caused by the imbibition of an element of soil or rock having a known initial physical state subjected to a state of zero or almost zero stresses.

The swelling index

It reflects the importance of the resulting swelling deformation by unloading with respect to a given stress state.

Cinematic of swelling

In clay soils, swelling is a very slow phenomenon because of the low permeability of clays. It is obvious to begin the study of swelling by studying the kinetics, that is to say the relation between swelling deformation and time. Figure 7 illustrates the relationship between uni-dimensional swelling and the logarithm of time.

The graph obtained by a free swelling or unloading test shows that the deformation can decompose into primary swelling and secondary swelling.

The primary swelling phase refers to the migration of water into the specimen from its ends, related to the diffusion process. According to the nature and the state of the material and according to the loading, this phase can last a few hours, even a few days for a 25-mm test tube with imbibition by the two faces.

The secondary swelling phase is more complex, because the direction of the swelling deformation is opposite to that of the loading.

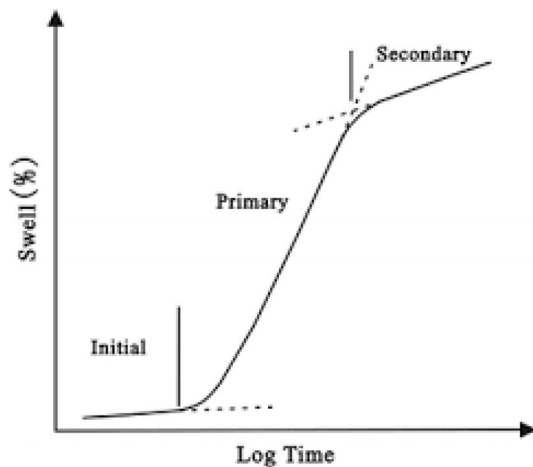


Fig. 7 Swelling curve as a function of time

It depends on the level of loading. The kinetics of secondary swelling is very slow and depends on the level of loading and for low loads it is often impossible to achieve equilibrium under reasonable conditions of laboratory tests.

Description of the phenomenon of swelling around a cavity

The problem of swelling manifests itself in a certain number of tunnels already dug, and is likely to affect a certain number of tunnels to come, since the phenomena of expansion of the clay minerals concern very varied formations such as molasses, marl, clays and argillites widespread; so it was interesting to specify the mechanisms of swelling around a cavity, proposals for classification of swelling mechanisms were issued by Einstein and Bischoff (1975); swelling can be defined as an increase in the volume of the natural land as a function of time caused by the modification of the stresses, the increase in the water content or a combination of these two factors. Depending on the interaction or order of appearance of these causes, the authors presented several types of phenomena.

Phenomenon 1

The swelling can result from a modification of the state of constraints, in particular in the form of a reduction or a rotation of the stresses (for example, as a result of the erosion of the ground of coverage, the creation of a valley by a river or the opening of an underground excavation). A similar phenomenon can be observed on a smaller scale by the bounce of the particles.

Phenomenon 2

An increase in volume over time is often related to the adsorption or absorption of water, resulting from differences in concentrations, unsaturated or partially saturated intergranular bonds or potential differences.

Phenomenon 3

Stress modifications may lead to adsorption and/or absorption of water which causes a further increase in volume. The volume increase due to the modification of the stresses, and that due to the adsorption or the absorption of the water can appear simultaneously or one after the other.

Phenomenon 4

It is the reciprocal phenomenon of the phenomenon 3. In this case, the delayed increase of the volume, as a result of the adsorption and/or the absorption of the water leads to a modification of the stresses which causes a further increase of volume. Here again, the two types of volume increase can appear simultaneously or one after the other.

Phenomenon 5

Adsorption of water accompanied by weakening of the bonds and/or reduction of the effective stresses can cause, as a function of time, a decrease of the shear strength. This decrease in shear strength in turn causes displacements whose characteristics are similar to those of swelling, especially in the case of underground excavations, although it is basically a phenomenon of creep.

Phenomenon 6

The phenomenon known as “squeezing” was defined by Terzaghi (1946) as an increase in the shear deformation of a terrain element over time, when it is subjected to a state of deviatoric constraints (during an excavation, for example). This phenomenon, which results in the slow development of plastic deformations, usually produces a limited volume deformation, depending on the dilatancy of the material.

It should be noted a gradual transition between the phenomenon 5 and the phenomenon 6, the latter progressing at a lower speed and causing smaller displacements. Phenomena 5 and 6 can occur concurrently with phenomena 3 and 4.

The foregoing remarks demonstrate that it is difficult to distinguish the different types of swelling. Swelling in the strict sense (phenomenon 1–4) is often associated with creep (phenomenon 5) and plasticity (phenomenon 6).

A numerical calculation was carried out using the software Plaxis 3D Tunnel which is based on the convergence–confinement method, taking into account the effects of swelling.

Presentation of the convergence–confinement method

The convergence–confinement method (Panet and Guellec 1974) is based on the observation that the deformation field obtained from an axisymmetric calculation, in a section perpendicular to the axis of the tunnel, is similar to that given by a plane calculation, in which the tunnel wall is supported by a fictitious pressure:

$$\sigma_f = (1 - \lambda)\sigma_0.$$

σ_0 : representing the value of the natural stress in place, assumed to be uniform and isotropic and λ a coefficient between 0 and 1 (Fig. 8). This result is valid provided that it is placed at a sufficient distance (of the order of $R/2$) from the tunnel face. The coefficient λ which characterizes the degree of decompression behind the face of the face is called deconfinement rate.

Based on this observation, Panet and Guellec (1974) suggested taking into account, in plane geometry, the stabilizing effect linked to the proximity of the face of the face, by applying the pressure σ_f on the periphery of the tunnel. The progression of the face of size is then simulated by increasing λ progressively from the value 0, corresponding to the initial state of constraints, to the value 1, corresponding to the state of deformation stabilized behind the face of size. In the case of a linear–elastic behavior of the ground, this value is reached at a distance of the order of two diameters of the face of size.

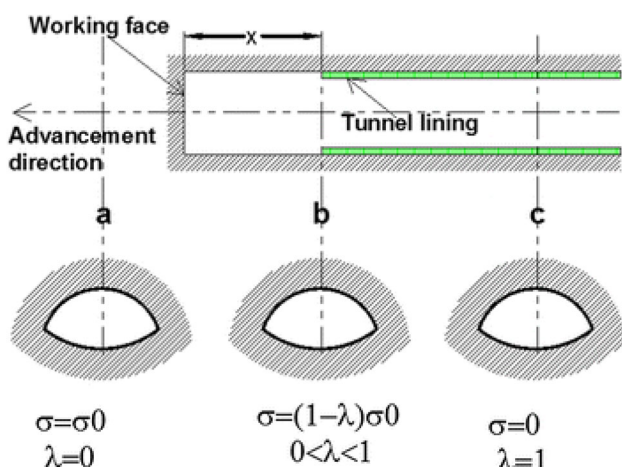


Fig. 8 Convergence–confinement method (after Panet and Guellec 1974)

The behavior of the ground–support ensemble (Fig. 9) is then analyzed in a diagram (P, U). The convergence curve (A) represents the response of the terrain. The activation of the support intervenes from its contact with the ground; this occurs for a value U in of the deconfinement rate. The progressive loading of the support is represented by the curve (B), called confinement. The point of intersection between the curves (A) and (B) characterizes the equilibrium state of the tunnel, and makes it possible to determine, by simple reading, the value of the radial displacement and the final pressure at the level of the support.

The method, initially developed in the case of a circular tunnel dug in a homogeneous and isotropic terrain, was later extended to other types of conditions, including anisotropic initial stress cases (Panet 1995). It is commonly used to represent the presence of the face in two-dimensional tunnel calculations of elements finished. In the next part, we will present an analytical calculation based on this convergence–confinement method by adding a swelling law for the behavior of the terrain.

Digital tunnel modeling

Geometric and geotechnical hypothesis of the model

According to the geological section of the ground, the thicknesses retained for the principal layers framing the section of tunnel are the following ones (from top to bottom):

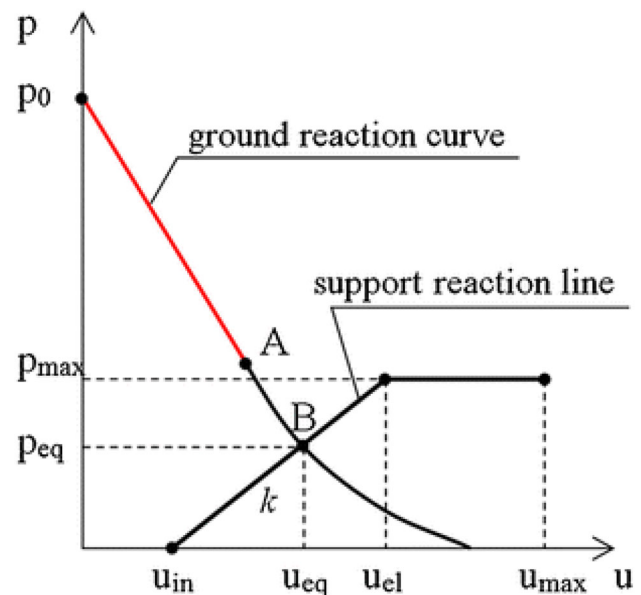


Fig. 9 Principle of the convergence–confinement method (Panet and Guellec 1974)

- Sandy-clay soil: 3.5 m.
- Argillites: 200 m.

The tunnel cover is about 119 m or the section under study is just below a pond (Fig. 10).

The mechanical characteristics of the materials

Land

The mechanical characteristics of the natural terrain of the studied section are summarized in Table 1 (from the results of a given geotechnical study of the project). The swelling ground is modeled as a perfectly plastic elastic material with a Mohr–Coulomb rupture criterion, taking into account the effects of swelling by the introduction of swelling parameters.

- Young's module E^{LT} ($E^{LT} = E/2$) long-term modulus of 50,000 (kN/m²).
- Cohesion C^{LT} along term of 100 (kN/m²);
- Internal friction angle ϕ^{LT} of 13°;
- A swelling pressure of 378 KN/m²;
- A swelling index B_g of 0.071;
- An anisotropic factor equal to 1;
- An apparent Poisson's ratio ν app equals to 0.17.

The temporary support

The temporary support of the excavated parts consists of HEB200 hangers and shotcrete plus anchor bolts are shown in Table 2. The same structure is adopted to cancel it. The mechanical characteristics of the various components are as follows:

- Shotcrete:

Area = 0.3 m², density = 2400 kg/m³, $E = 28.106$ KN/m², $I = bh^3/12 = 0.00225$ m⁴, Poisson's ratio $\nu_b = 0.2$.

- Crosslinked hanger:

Area = 1970 mm², $E = 210.106$ KN/m²,

- Anchor bolts D25:

$E = 2.1E8$ KN/m², $L = 6$ m, surface for $\varnothing 25 = 490.62$ mm².

In this study, it is proposed to strengthen the face of Fits (mechanical characteristics of fiberglass tubes are shown in Table 3). This technique involves the use of GRFP tubular inclusions based on glass fiber-reinforced polymer embedded in the terrain by an injection system using a cement grout to stabilize the cutting edges of the sections, calotte and stross, and to oppose the deformations and solicitations generated by the movement of the terrain in different directions.

The final coating consists of 40 cm of B25-encased concrete that is to say with a compressive strength of 25 MPa using the rules proposed in BAEL91 (1992).

For concrete, the characteristics used in the calculations are: density $\gamma_b = 25$ kN/m³, Area = 0.4 m², density = 2500 kg/m³, instantaneous deformation modulus $E_b = 30,000,000$ KN/m², Poisson's ratio $\nu_b = 0.2$, instantaneous deformation modulus E_b^{LT} ($E_b^{LT} = E_b/3$) = 10,000,000 KN/m².

Presentation of the numerical calculation

Calculation method

Numerical modeling consists of creating a simplified representation of tunnel digging; this representation is carried out by the finite element method (FEM) using the PLAXIS3D TUNNEL software, to deduce the different solicitations due to the soil–support interaction on the global tunnel behavior. Fifteen-node triangular elements will be adopted. The resting earth pressure coefficient is taken as $K_0 = 1$ (convergence–confinement method) by the introduction of a deconfinement rate.

The boundary conditions selected are the following:

- Zero horizontal displacement on the lateral boundaries;
- Zero vertical displacement on the lower limit.

It is assumed that the water table is below the tunnels.

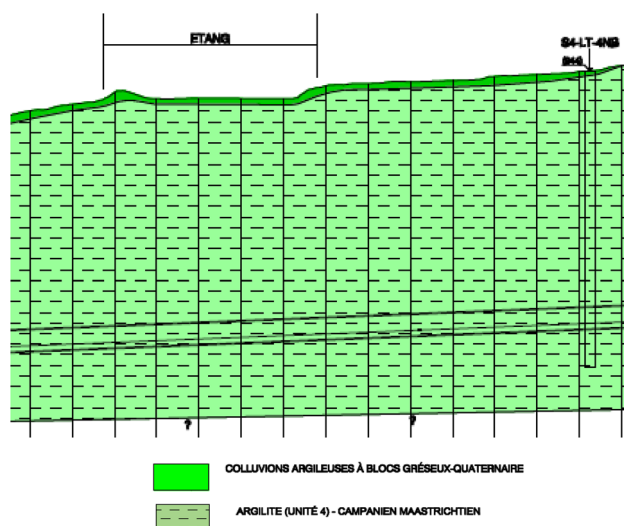


Fig. 10 Geological section of the terrain

Table 1 Mechanical characteristics of the natural floor of the face

Natural land	γ (kN/m ³)	γ_{sat} (kN/m ³)	C (kN/m)	φ (°)	E (kN/m)	ν
Sandy clay	19.05	21.59	0.29	19.5	2E4	0.33
Argillite	25.10	25.58	200	26	1.0E5	0.35

Table 2 Summary of the mechanical characteristics of the temporary support and anchor bolts

1	Tensile strength	600 N/mm ²
2	Shear strength	100 N/mm ²
3	Modulus of elasticity	20,000–30,000 Mpa
4	Linear weight	3 kg/m
5	Heavy duty section	346 mm

Table 3 Mechanical characteristics of GRFP tubes

Rigidity	Temporary support	Anchor bolt
Normal stiffness EA (kN/m)	5.18E+06	10.3E+04
Flexural rigidity EI (kN m ² /m)	6.9E+04	4.02

Calculation phasing

The calculations were conducted considering the following phasage:

- Phase 0: initialization of constraints (geostatic constraints)
- Phase 1: total digging of the left tunnel and temporary retaining structure over a length of 6 m ($\lambda = 1$). Flat front, SLICE 1
- Phase 2: excavation of 2-m cap; SLICE2, SLICE3, SLICE4 with deconfinement ($\lambda = 0.4$).
- Phase 3: temporary support installation hanger + shotcrete + anchor bolt (for the excavation part—calotte); Plane A slice 2, Plane B slice 3, Plane C slice 4; deconfinement will be ($\lambda = 1$).
- Phase 4: excavation of 6-m stross; SLICE2, SLICE3, for a deconfinement ($\lambda = 0.4$).
- Phase 5: installation of temporary support hanger + shotcrete + anchor bolt + reinforcement of face by the application of the force of pressure (For the excavate part—stross); Plane A slice 2, Plane B slice 3; deconfinement ($\lambda = 1$).
- Phase 6: excavation of Radier at 6 m; SLICE2, with a deconfinement ($\lambda = 0.25$).
- Phase 7: support to strike off; Plane A slice2, ($\lambda = 1$).

- Phase 8: partial creep corresponding to the first 10 years of service; we take a terrain module equal to 75,000 kN/m² (is $E1o/Eo = 0.75$) (according to Bultel 2001), and for cladding, since the creep loading from the ground is slow and permanent, it is assumed that the stresses in the concrete are determined with the long-term modulus (BAEL91 1992).
- Phase 9 to phase 16: repeated the same phases from 1 to 8 for the right tunnel.

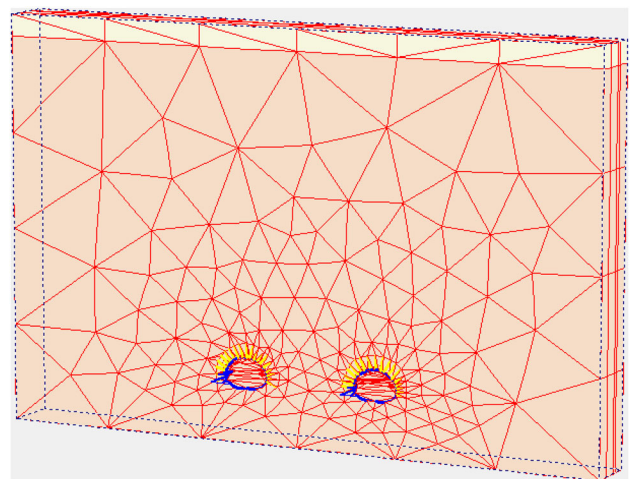
Note: Only partial creep, and not final creep, is taken into account because the collapse of the tunnel takes place during the first 10 years of commissioning (Fig. 11).

Analysis of results achieved

State of induced stresses in the soil around the wall of the tunnel cavity

The profiles of the stresses developed in the soil around wall of the cavity of the two tunnels are presented in Fig. 12.

The high depth of the tunnel generates high constraints around a tunnel cavity, it is observed that these constraints are released gradually in accordance with the method “convergence–confinement” (Panet and Guellec 1974), (Panet and Guenot 1982), the convergence is linked to soils, and this connection is explained by the phenomenon of soil swelling.

**Fig. 11** The geometry and the mesh of model in 3D

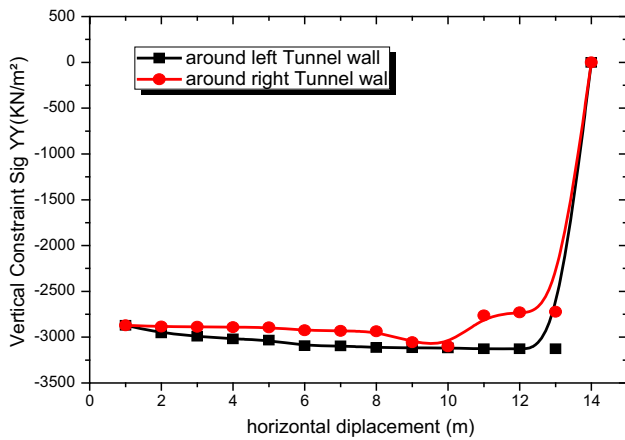


Fig. 12 The state of the constraints induced in the soil around of the cavity wall of the two tunnels

State of the deformations induced in the soil around wall of the tunnel cavity

The deformation profiles developed in the soil around wall of the cavity of the two tunnels are shown in Fig. 13.

We notice an evolution of the deformations of contact between the support and the ground. These deformations of soil are very high at the level of right tunnel, the strong deformation is resulting from several factors such as the swelling, and the weak resistance of the compression of the argillite (AFTES 1 2003; AFTES 2 2003), these deformations that can translate the opening of a crack in the soil mass.

State of the stresses induced in the wall of the tunnel cavity

Figure 14 shows the evolution of the stresses in the wall of the cavity of two tunnels. The wall stresses of the

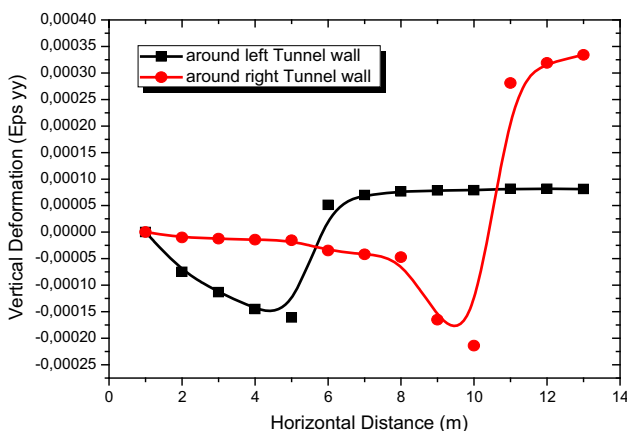


Fig. 13 The state of the deformations induced in the soil around of the cavity wall of the two tunnels

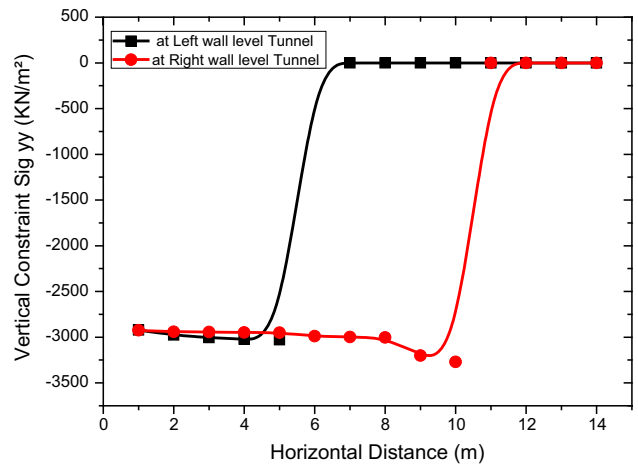


Fig. 14 The state of the constraints induced in the support of the cavity wall of the two tunnels

excavation are progressively released due to the effect of soil convergence around the tunnel cavity. The pressure obtained becomes negative, which highlights tensile stresses in the concrete wall; these high stresses cause the destruction of the support.

State of the deformations induced in the concrete wall of the tunnel cavity

Figure 15 shows the evolution of the deformations in the supports of the two tunnels. There is a significant increase in deformations in the left tunnel retaining concrete. The strong deformation due to the convergence which is linked to soils, this connection explained by the phenomenon of soil swelling. These deformations lead to the opening of a crack in the wall and in the tunnel roof.

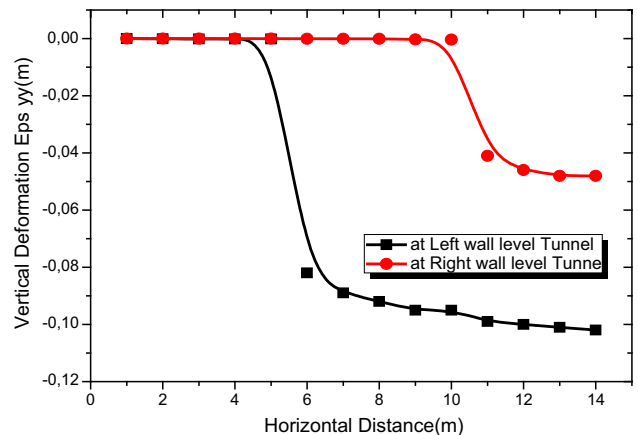


Fig. 15 The state of the deformations induced at the level concrete wall of the cavity of the two tunnels

Settlement of the massif at the free surface of the soil

Figure 16 shows the settlement variation of the massif above the tunnels along three points (at the left tunnel axis, between the two tunnels, and at the right tunnel axis).

According to the figure shown above, we notice a sudden uprising of the ground above the left tunnel, and against a settlement on the ground surface corresponding to the area above right tunnel and between the two tunnels. Soil uplift is due to the phenomenon of swelling of argillite.

Movement of the massif under the raft

Figure 17 shows the vertical displacements at the raft/ground interface along a line under the rafts of the two tunnels.

We note well a weak settlement of soil under the rafts of the two tunnels in the first time, then a very sudden lifting of ground under the rafts of the two tunnels. On the other hand, there is a settlement of the soil corresponding to the zone between the two tunnels.

There is a noticeable influence of the swelling on the vertical displacement under the rafts of the two tunnel tubes, which corresponds to the zone of the raft/ground interfaces, that is to say the swelling is entirely directed towards the interior of the tunnel.

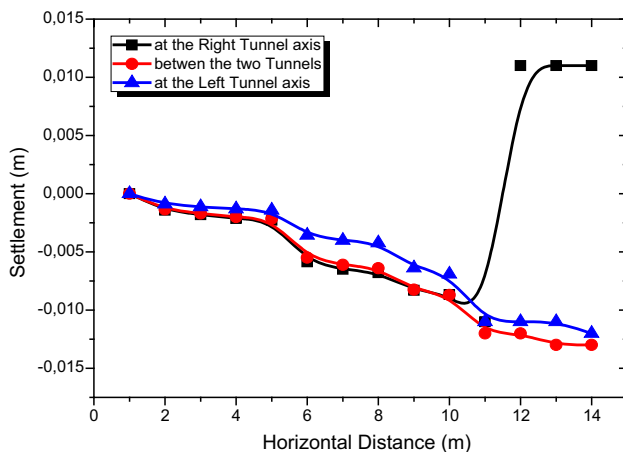


Fig. 16 Settlement on the soil surface according to the different points above the tunnels

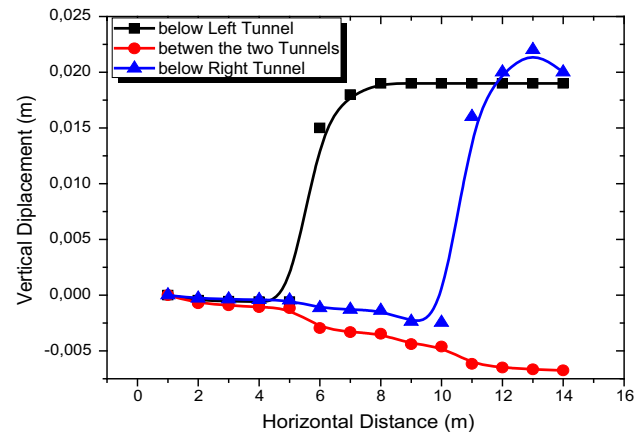


Fig. 17 Vertical displacement next a line under the rafts of the two tunnels

The effect of taking into account swelling in numerical modeling

The inclusion of swelling has been incorporated into the numerical calculation in phases 7 and 8, which amounts to making the assumption (according to Bultel 2001) that the phenomenon starts from the commissioning of the system under floor drainage capable of bringing the water from the hydrophilic zone of the argillite into contact with the initially dry zone (by effect of capillarity). After the excavation of the raft and the total deconfinement of the ground (phase 6), the stresses under the raft become weak and favor the development of the swelling. This process can take several days (or even months). As the water propagates slowly in the massif, it is reasonable to associate the swelling with the two delayed-effect calculations. The phenomenon of swelling was, therefore, introduced simultaneously with creep. The swelling zone is located below the slab (below the water table); it is assumed that the water flows from the drainage system along the base and at depth.

Conclusion

This work presents a pathological analysis of a real case of a part of the tunnel T1 Djebel El-Ouahch which suffered a collapse of the right tube and a very important damage of the second tube. A numerical computation model of the damaged tunnel section has been established; this calculation may not be the solution to the problem, but a reasonable approximation of reality.

After the analysis of the results, it is noticed that the swelling of ground can entail following consequences:

- Cracking in the soil mass around straight tunnel tube.

- Destruction of tunnel tube liner.
- Soil uplift above tunnel left tube.
- The lifting of soil below the rafts of the two tunnel tubes.

The difficulties caused by swelling soils for tunnel construction have already been widely discussed in the literature (Einstein and Bischoff 1975; Chen 1965, 1975; Mou roux 1988; Robert and Fabre 1987; Bellwald 1990; Steiner 1993 and many others). Some detailed and significant feedbacks provide information related to the swelling phenomenon observed in situ, for example, Bözberg Tunnel (Switzerland) excavated in the marl Built between 1871 and 1875, Belchen Tunnel (Switzerland) dug into the marl Anhydrite and shale built between 1963 and 1970, Tunnel San Donato (Italy) excavated in the Macigno formation (sandstone, silt) in the center, surrounded by the argillaceous complex Scagliose (shale), then the Alberese formation (marly limestone) Tunnel on the dam of the Saskatchewan River (Canada) dug in the shale, and the tunnel of recognition of the Chamoise tunnel (France) dug in the marl, built between 1979 and 1980, this swelling is manifested mainly by a large uprising of the to strike off associated with strong convergences in pedestrian.

The recommendations

In the field of underground excavations, the swelling of the ground can cause serious problems, both during construction and after the commissioning of the work; many engineers are studying the problem of swelling (Mou roux 1988, Frédéric Bultel 2001).

In our study to limit the swelling effects of the land, it is necessary to provide:

- The improvement of the wall reinforcement section from the increase of the section of the profiles of the hangers, and thus the thickness of the shotcrete, to limit the deformations of the soil around the wall cavity to a permissible value.
- The repair of coatings by the use of masonry binders with good mechanical properties.
- Re-excavation of the raft thus becomes a regular maintenance operation; it is necessary to build inverted vault rafts intended to limit the movements to an acceptable value.
- Draining the water flowing into the tunnel is a good initiative against swelling. Indeed, channeling the water inflow reduces the contact of the water with the terrain likely to swell. However, the drainage system must remain effective after the construction of the tunnel because a simple leak can quickly put the water in

contact with the terrain that may inflate and destroy the tunnel.

Compliance with ethical standards

Conflict of interest On behalf of all authors, the corresponding author Mrs Berkane states that there is no conflict of interest.

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