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MODELING OF THE CONSTRUCTION OF ANUNDERGROUND TUNNELIN A SATURATED GROUND

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ABSTRACT

The construction of tunnels in aquifer areas has always posed problems for geotechnicians during digging or during operation of the structure.The risks linked to the stability of the contour of the tunnel are essentially due to the poor characteristics of the ground in place and a danger of loosening under strong hydraulic gradients is to be feared.The development of digging methods permitted to engineers to be able to better anticipate the movements generated in the surrounding soils, and the construction of tunnels in this type of terrain has become possible.The purpose of this workis to model the behavior of a circular tunnel built in saturated soil with the aim of investigating the method that would be suitable for its construction. Two digging methods are proposed and consist of using the tunnel boring machine and the new Austrian method (NATM). The analysis is carried out while respecting the construction stages to compute the forces and moments that develop in the final concrete liner over the long term.

Keywords: Tunnels; Aquifer Zones; Drying Wells; NATM method; Tunneling.

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1. INTRODUCTION

The construction of underground is in permanent expansion because of their use in various fields. Today, this field of civil engineering is supported by a series of modern technological innovations making it possible to meet the growing needs for communication and transport (goods, water), but also to ensure the storage of dangerous materials (oil, gas), to relieve congestion on the surface of cities (underground car parks) or to accommodate energy production units (buried power plants). The cumulative length of underground structures built around the world shows the importance of the number of underground structures built over the years [1].

Underground construction generates specific risks during all stages of the project; during its construction, its operation and even after its abandonment. The history of the tunnels indeed mentions spectacular collapses and other disasters whichdemonstrate the existence of a potential for large-scale accidents during the digging and operation of these structures. The main risks encountered have several origins. They are linked to geotechnical and geological risks, hydrological risks and study risks. The risks linked to instabilities in these structures are classified according to a scale of severity of the corresponding impacts [2], [3].

Sizing methods, calculation tools, digging techniques and support methods have evolved significantly over time, but the history of underground construction reserves many surprises and is full of tragic accidents with losses of human lives. These accidents can also have economic repercussions and social impacts which must not be neglected [4]. In urban sites, engineers are faced with a major problem concerning the protection of existing structures against the risks induced by digging. Also, in the first time, many researches in this area were oriented towards determining the soil movements induced by digging in the absence of these structures. For this, various approaches have been developed and are based on auscultation and in-situ measurements to provide valuable information on the ground movements induced by digging and have made it possible to constitute a good knowledge base in this area [5], [6], [7], [8], [9], [10].

In aquifer areas, the construction of a tunnel has always posed enormous problems which have not been fully resolved at present. The difficulty is all the greater as the soil has low cohesion and there is a risk of loosening under strong hydraulic gradients, inevitably causing the gallery to flood during tunnel digging.

Excavation and support methods continue to evolve to respond to the various problems that arise during all stages of the project. In soils with good characteristics, excavation is generally carried out with conventional methods (explosives or with point attack machines) which rarely require temporary supports such as shotcreteand anchor bolts. In soils with poor characteristics, frontal and lateral conservation of the excavation and control of hydrostatic pressures are essential. The use of the New Austrian Method (excavation in parts) or the tunnel boring machine are possible. The use of the Austrian Method makes it possible to limit the deconfinement of the soil around the excavation but it does not make it possible to control the hydrostatic pressures, which requires the drying of the soil. The use of a tunnel boring machine (TBM) ensures the stability of the working face and theconservation of hydrostatic pressures due to the application of frontal pressures, and to minimize the convergence of the ground due to the presence of the metal skirt as well as the possibility of injecting a filler mortar on the upper surface of the concrete liner.

For the construction of deep underground tunnels, one of the important problems is thedesign of the lining taking into account the flow of groundwater in the tunnel especially when the infrastructure is located underwater [11]. The effective stresses around the tunnel are affected by the resulting seepage forces. Determining the stress distribution around the tunnel is essential for its influence on worker safety and daily preservation, particularly when underground openings are excavated in areas with high seismic potential [12]. Analytical models can provide stress and displacement fields across the entire domain through a rigorous mathematical derivation that takes into account the relevant physical parameters [13]. Analytical solutions are considered more advantageous than other methods because they allow the governing physics to be explored more effectively and the accuracy of the results to be verified. However, numerical methods make it possible to study the poro-mechanical instabilities of rocks and the nonlinear behavior of geomaterials which are difficult to achieve analytically [14], [15].

The water supply in a tunnel has been widely studied in recent years [16], [17], [18], [19], [20], [21]. In these studies, water tables were considered using conformal mapping, which mapped the fully saturated semi-infinite hydraulic domain onto a circular ring. [22] showed that the variation of pore pressure has a great influence on the distribution of stresses and displacements around tunnels.

This work involves modeling the construction of a circular tunnel located at a depth of 65 m below the ground surface. The major problem in realizing this task is the presence of a water table 5 m from the ground surface. Two solutions are being investigated for the digging of this tunnel in the particular conditions; the tunnel boring method and the new Austrian method. For each solution considered, thefollowed approach for the numerical modeling and the main obtained results are presented. We will be particularly interested in determining the forces and moments which develop in the concrete liner when the temporary shotcrete loses its strength.

2. PRESENTATIONOF THE CASE STUDY AND GEOTECHNICAL CONTEXT

The case considered in this study is a simple representation of a deep tunnel in saturated soil [23]. It is a circular tunnel an excavated radius of6 m, located 65 m below the ground surface as shown in Fig. 1. For the construction of the tunnel, the first suggested solution consists of the use of the tunnel boring machine (TBM) for the excavation and the installation of the final cast-in-place thick concrete liner.

A filler mortar is injected to fill the gaps between the final concrete liner and the surrounding ground. The use of this method does not require the drawdown of the water table, but depends on the availability of the tunnel boring machine. The second solution consists of the use of the New Austrian Method which uses a half-section excavation method and temporary support reinforced by anchor bolts before the installation of a final cast –in- place concrete liner. The use of this method requires the drawdown of the water table using dewatering wells, which will be active during the construction of the structure.
The soils in place are made up of a homogeneous soft rock made up of limestone whose characteristics, in drained conditions, are summarized in Table 1. [23]

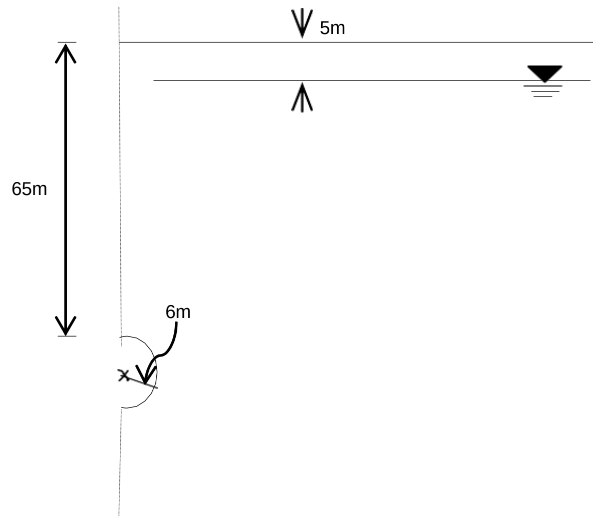


Fig.1. Conditionsof the studied case

Table 1. Properties of rock in place [23]

	Units	Rock
Unsaturated specific weight(γ_{unsat})	(kN/m^3)	21.40
Saturated specific weight(γ_{sat})	(kN/m^3)	22.40
Young Modulus (E)	(MPa	1375
PoissonCoefficient (ν)		0.3
Cohesion(C)	(kPa)	20
frition angle(φ)	(Degree)	20
Permeability	m/s	10^{-8}

3. NUMERICAL MODEL AND BOUNDARY CONDITIONS

The numerical analysis is carried out using the PLAXIS-2D finite element software. The mesh adopted for the numerical model is shown in Fig. 2. The symmetry of the problem allows us to treat half of the model by considering a line of symmetry which corresponds to X

= 0. The dimensions of the considered model are 0.0 to 120.0 m in the X direction and from -80.0 m to 65.0 m in the Y direction.

The boundary conditions are imposed in such a way as to prevent any horizontal displacement on the lateral boundaries and the base of the model is considered fixed.

For modeling the behavior of the soil in place, the perfectly plastic elastic behavior law of Mohr-Couloumb is used. The parameter values needed for this model are shown in Table 1.

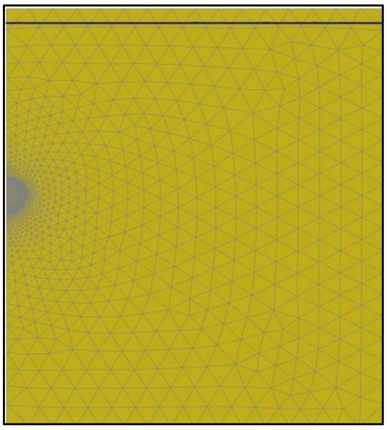


Fig.2. Adopted mesh for the numerical model of the tunnel

4. INITIAL STATE OF STRESS AND PORE PRESSURESIN THE EXISTING SOIL

The initial stress distribution of the soils in place before any loading is obtained using the K_o procedure. In Fig. 3 are presented the vertical and the horizontal stresses in the soils before the construction of the tunnel.

In this project, the water table level is 5m from the surface, the initial distribution of pore pressures in the existing soils is illustrated in Fig. 4.

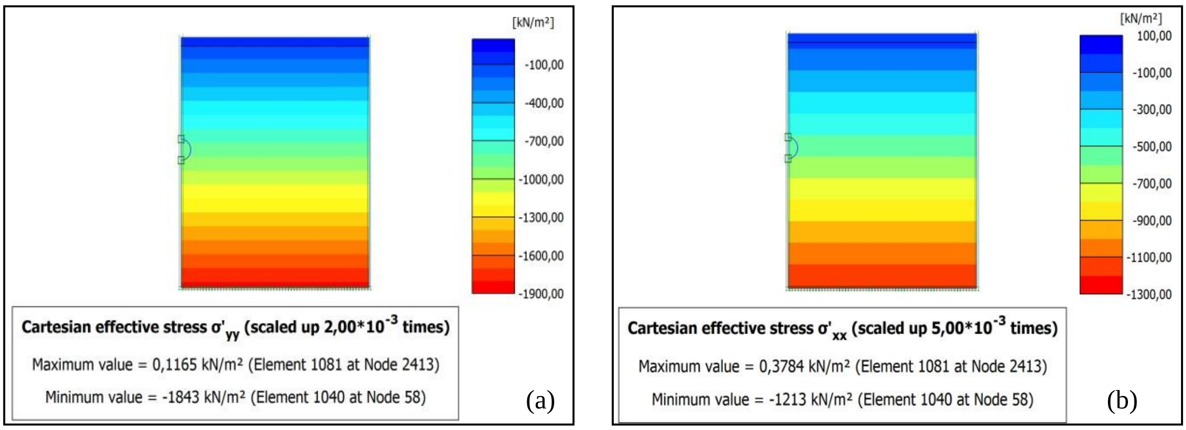


Fig.3. Distribution of stresses in the ground in the initial state: a- vertical stresses, b- horizontal stresses

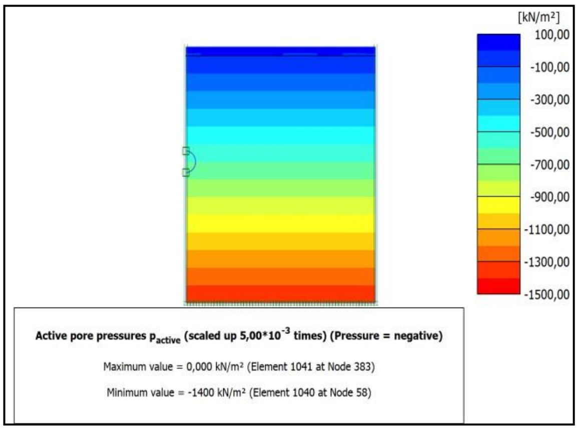


Fig.4. Distributionpore pressures in the soil in the initial state

5. MODELING OF THE CONSTRUCTION OF THE TUNNEL USING THE TUNNELING METHOD

The simulation procedure consists of checking the stability of the tunnel during excavation with a tunnel boring machine and after installation of the final liner and injection of the filler mortar while activating a contraction of Cref =0.5. The structural elements are modeled in the form of quadrilateral plates with linear elastic behavior, having a bending rigidity EI and a normal stiffness EA. The properties of the concrete liner used are summarized in Table 2.

Table2. Properties of final cast-in-place concrete liner

	Units	Concrete liner
Specific weight	(kN/m^3)	25
Young Modulus (E)	(GPa)	15
PoissonCoefficient (ν)		0.2
Normal Rigidity (EA)	(kN/ml)	6×10^6
Bending Rigidity (EI)	(kN.m²/ml)	79500
Thickness	m	0.4

In Fig. 5 are presented the displacements obtained after digging the tunnel and installing the final liner. We note that the maximum vertical displacement of 1.67 cm is located at the top (tunnel vault) (Fig. 5a). A maximum horizontal displacement of 2.33 cm is recorded at the side part of the tunnel (Fig. 5b).

In Fig. 6 are indicated the displacements induced once the packing mortar degrades and the ground relaxes on the final concrete liner. We note an insignificant increase in these displacements compared to the previous situation (0.02 cm for the vertical displacement and0.05 cm for horizontal movement). This means that the packing mortar does not play a role in terms of strength. A vertical displacement of around 1.7 cm is induced in the final concrete liner as shown in Fig. 7.

The distribution of internal forces in the final concrete liner is shown in Fig. 8 for the axial forces and for the bending moments. A maximum force of 4615 kN/ml and a maximum moment of 39.81 kN.m/ml are induced. These requests are necessary to calculate the appropriate reinforcement for the concrete linerused in this tunnel.

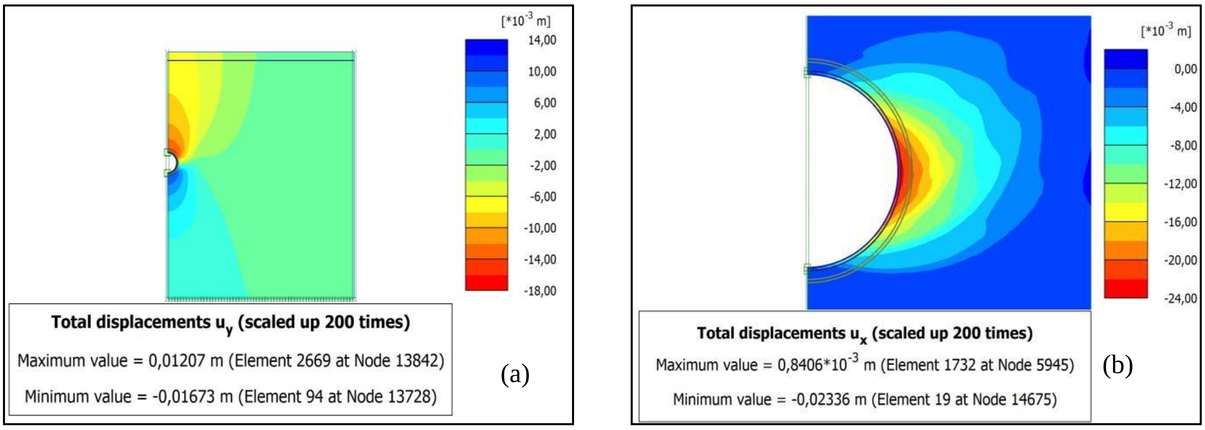


Fig.5. Displacements induced in the ground after digging the tunnel and installing the final concrete liner: a- vertical displacement, b- horizontal displacement

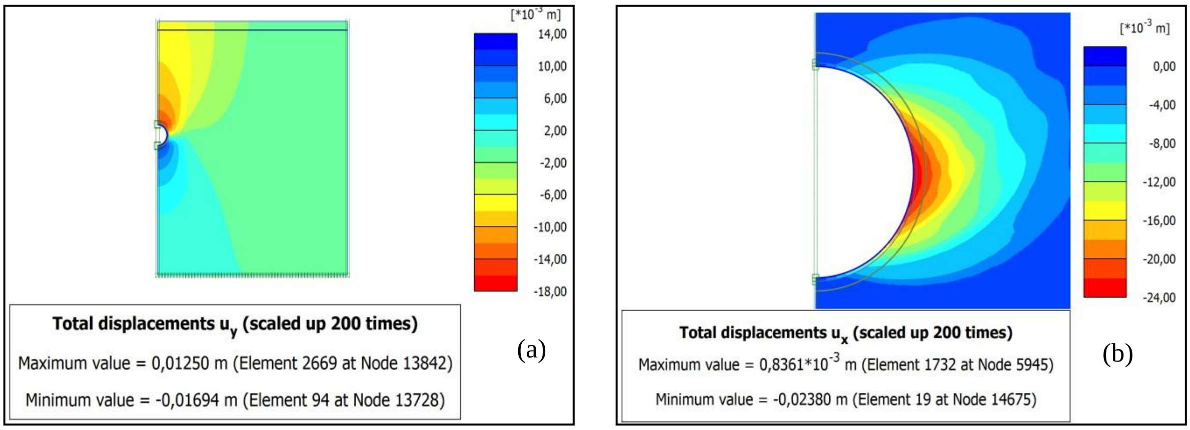


Fig.6. Long-term induced displacements in the ground: a- vertical displacement, b- horizontal displacement

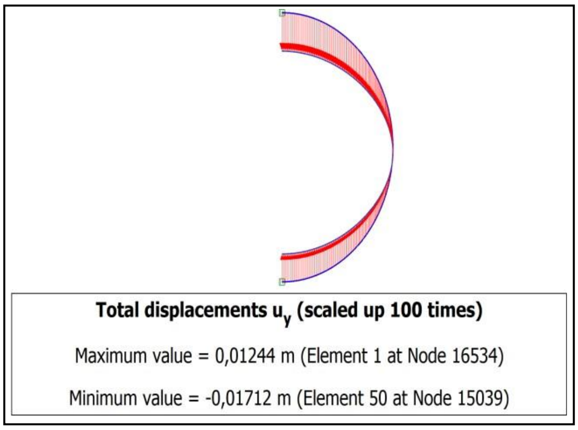


Fig.7. Vertical displacement induced in the final concrete liner

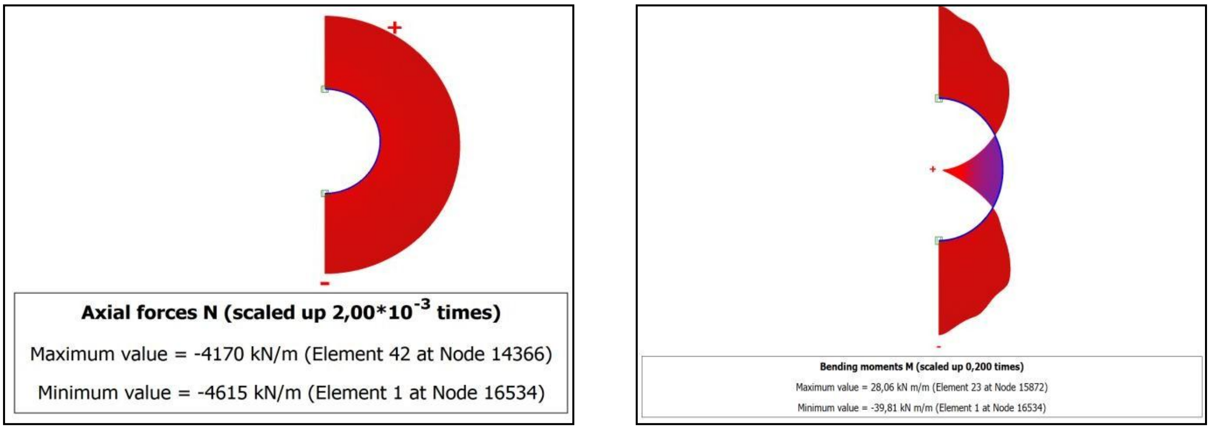


Fig.8. Axial force and bending moments in the final concrete liner

6. MODELING OF THE TUNNEL CONSTRUCTION BY THE NEW AUSTRIAN METHOD (NATM)

The second solution consists of creating the tunnel in question by half-section excavation. The structural elements (temporary support and final concrete liner) are modeled in the form of quadrilateral plates with linear elastic behavior, and having a bending rigidity (EI) and a normal stiffness (EA). The required characteristics are presented in Table 3 for the shotcrete and anchor bolts and in Table 4 for the final cast-in-place concrete liner. An equivalent thickness for the structural elements d_{eq} is obtained by the following relation (1):

$$d_{eq} = \sqrt{12\frac{EI}{EA}} \tag{1}$$

The adopted approach to model the construction of the tunnel using the NATM method comprises seven stages. Each of them represents a stage of in-situ construction:

1- Reduction of the water table level using five dewatering wells placed every 20 m, with a flow rate of 20 m^3/day.
2- Excavationof the upper part of the tunnel with soil deconfinemento f *(1-β)=60%*.
3- Application of shotcretesupport and installation of anchor bolts with end of deconfinement *(1-β) =100%*.
4- Excavationof the lower part, with soil deconfinementof *(1-β)=60%*.
5- Application of the support of the lower part and installation of the anchor bolts with application of the end of deconfinement *(1-β) =100%*.
6- Installationof the final cast-in-place concrete liner.
7- Long-term calculation, the level of the water table is at its original level and the shotcrete loses its resistance.

Table 3. Properties of shotcrete and anchor bolts

	Units	shotcrete	anchor bolts
Specific weight	(kN/m^3)	24	29.40
YoungModulus (E)	(GPa)	10	210
PoissonCoefficient (ν)		0.2	0.2
Normal Rigidity(EA)	(kN/ml)	2×10^6	-
Bending Rigidity (EI)	($kN.m^2/ml$)	6600	-
Thickness	(m)	0.2	-
Diameter	(m)	-	0.022
Section	(m^2)	-	0.379
Anchor length	(m)	-	6

Table 4. Properties of the final cast-in-place concrete liner

	Units	final cast-in-place concrete liner
Specific weight	(kN/m^3)	25
Young Modulus(E)	(GPa)	25
PoissonCoefficient (ν)		0.2
Normal Rigidity(EA)	(kN/ml)	10^7
Bending Rigidity (EI)	($kN.m^2/ml$)	132500
Thickness	m	0.4

6.1. Reducing the Water Table Level Using Dewatering Wells

As already mentioned, five dewatering wells arranged every twenty meters are used to lower the water table to a level of 10 m below the tunnel with a flow rate of 20 m^3/day. After the water table has drawn down, the dewatering wells are active during all stages construction in order to keep the same level of the water table. Fig. 9 allows to visualize the operation of the shafts before digging the tunnel. The degree of saturation of the soils in place before and after the drawdown of the water table is shown in Fig. 10. This figure shows the good functioning of dewatering wells. The reduction in the level of the water table generated surface settlements of approximately 5 cm as shown in Fig. 11.

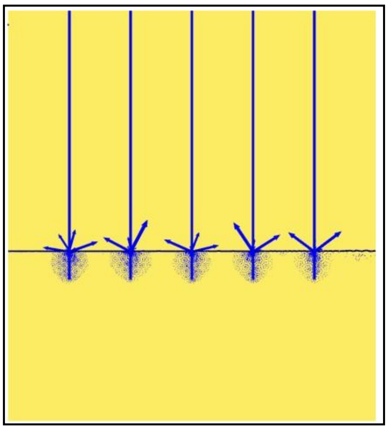


Fig.9. Operation of dewatering wells

6.2. Excavation of the Upper Part and Installation of the Support

The induced displacements after excavation of the upper part of the tunnel are shown in Fig. 12 and in Fig. 13 after the installation of the temporary support and the anchor bolts. We note that the induced displacements are a few millimeters during the excavation and application of 60% of deconfinement. These displacements reach a value of around 5.1 cm after the application of 100% of deconfinement and the installation of the support.

(a) (b)

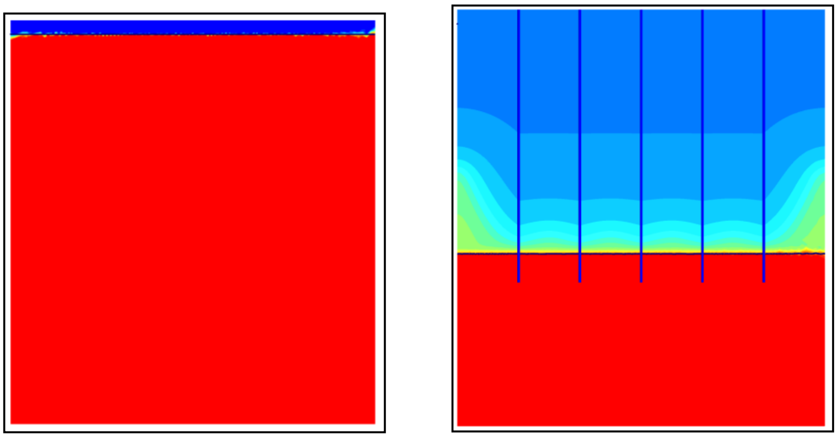


Fig.10. Degree of soil saturation: (a) before the drawdown of the water table – (b) after the drawdown of the water table

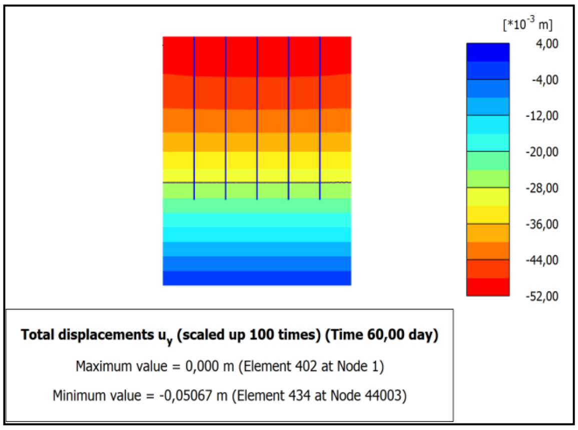


Fig.11. Induced vertical displacements in the ground by the drawdown of the water table

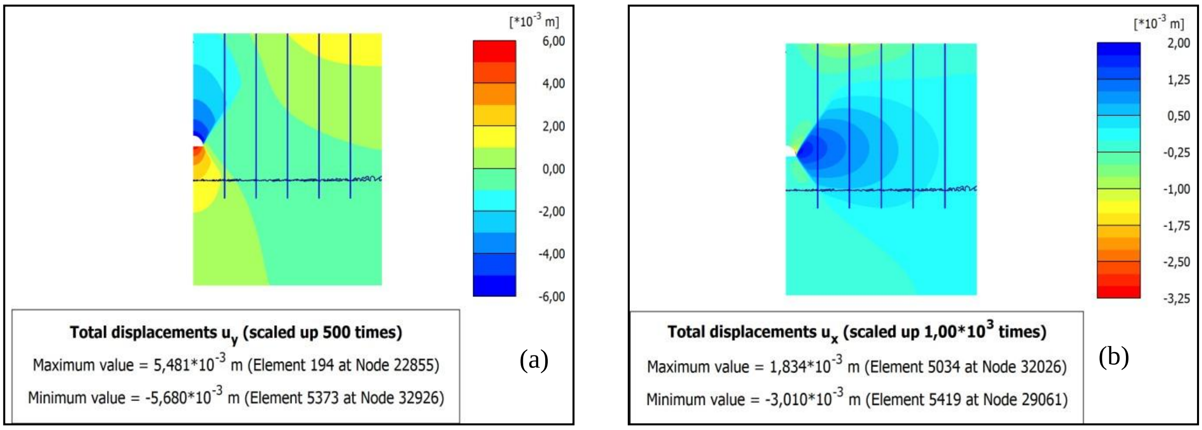


Fig.12. Induced displacements in the ground after excavation of the upper part: a- vertical displacement, b- horizontal displacement

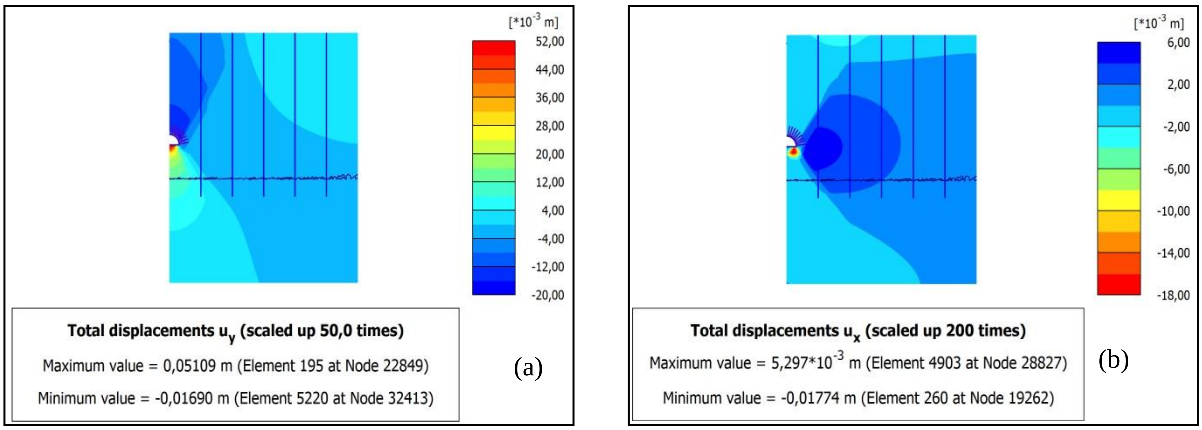


Fig.13. Induced displacements in the ground after installation of the support of the upper part: a- vertical displacement, b- horizontal displacement

6.3. Excavation and Support of the Lower Part

The obtained displacements after the excavation and the installation of the support of the lower part are shown in Fig. 14 and in Fig. 15 respectively. At the end of the deconfinement, a maximum of settlement of around 3.61 cm is recorded.

6.4. Final Cast-in-Place Concrete Liner Application

The displacements recorded after the placement of the final cast-in-place concrete liner are shown in Fig. 16. The state of the deformations remains almost unchanged compared to the previous step. This verifies the good short-term resistance of the applied support.

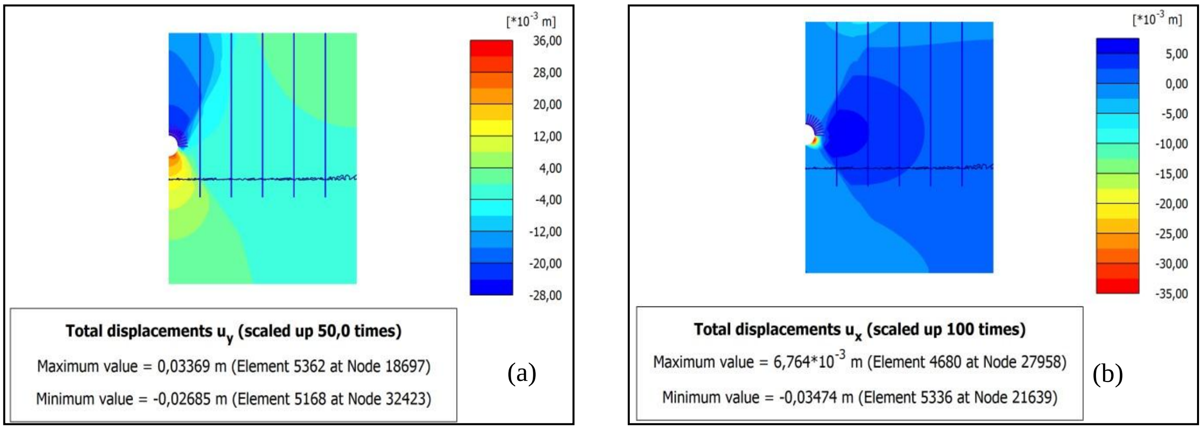


Fig.14. Induced displacements in the ground after excavation of the lower part: a- vertical displacement, b- horizontal displacement

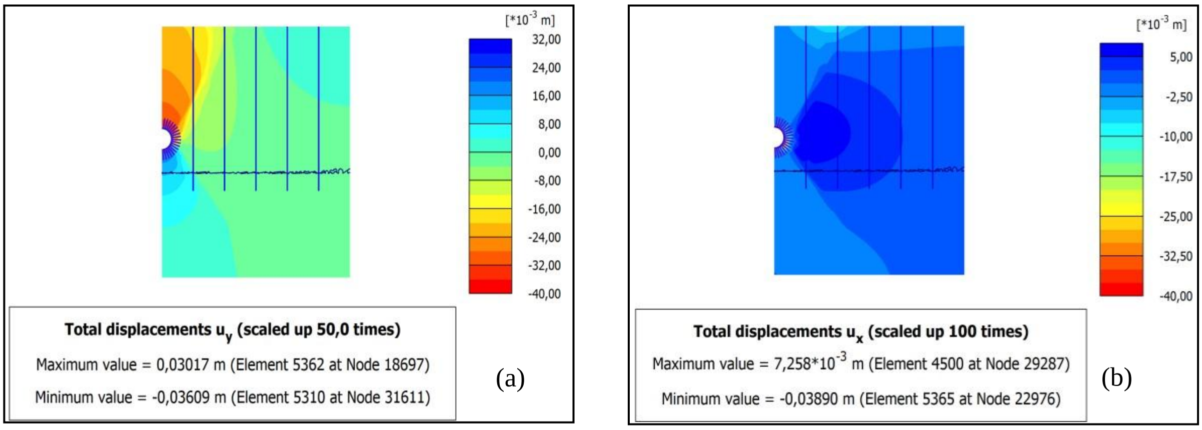


Fig.15. Induced displacements in the ground after installation of the support of the lower part: a- vertical displacement, b- horizontal displacement

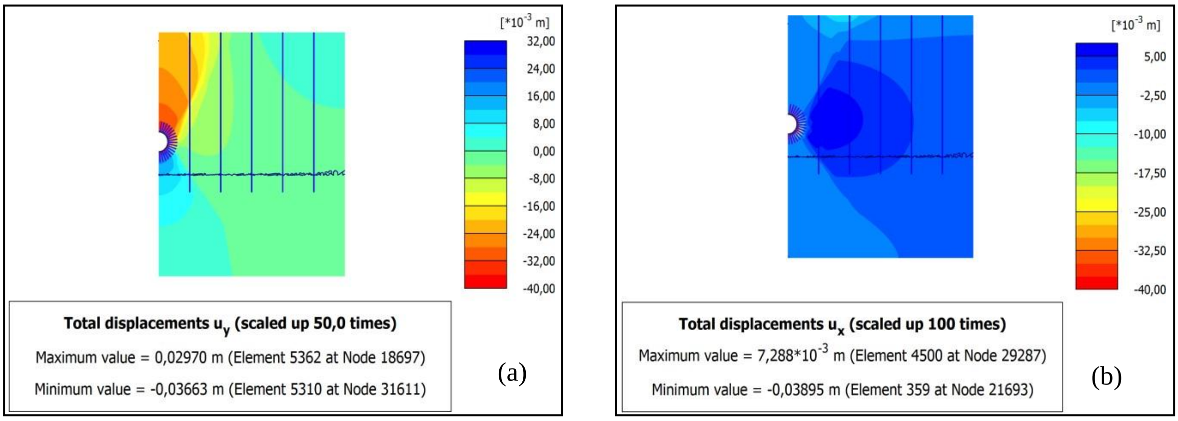


Fig.16. Induced displacements in the ground after installation of the final –in- place concrete liner: a- vertical displacement, b- horizontal displacement

6.5. Short-Term Behavior Once the Water Table is Restored to its Initial Level

Once the final cast-in-place concrete liner is installed, the dewatering wells are stopped and the water table rises to its original level. Pore pressures are re-established in the existing soil, which generates axial forces in the final concrete liner. The induced displacements after stopping the dewatering wells are shown in Fig. 17. A maximum displacement of 4.8 cm in the form of uplift is recorded on the surface.

6.6. Long-Term Behavior

In this computational step, the state of the displacements is checked in the long term once the shotcrete loses its strength and the level of the water table is still at its original level. We note that the deformations are very insignificant, which indicates good long-term behavior of the structure and the soils in place as shown in Fig. 18. The vertical displacement recorded on the final concrete liner is of the order of 1.77 mm as shown in Fig. 19. The distribution of internal forces in the final concrete liner, shown in Fig. 20 for the resultant of the axial forces and for the bending moments, gives a maximum axial force of 4614 kN/ml and a maximum moment of 60.77 kN.m/ml. These values are useful for calculating the reinforcement of this structural element.

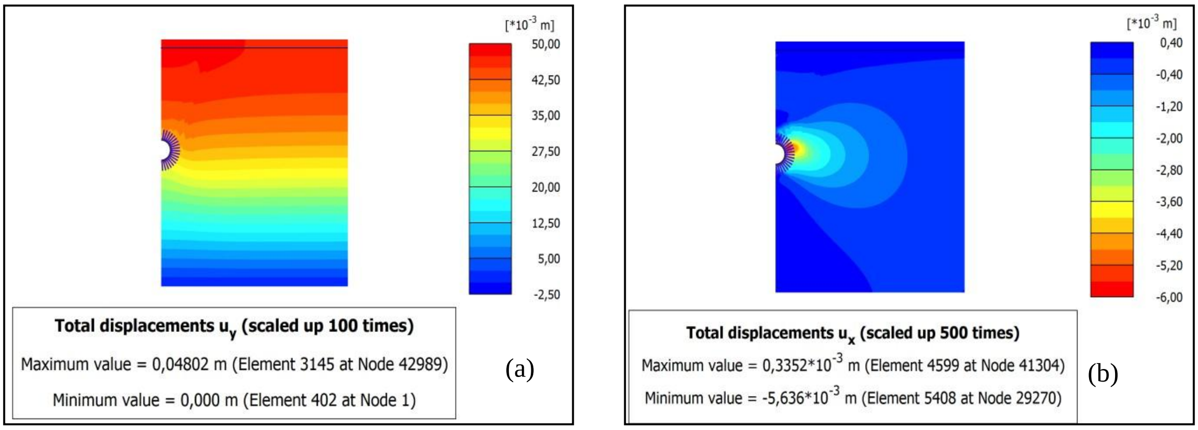


Fig.17. Induceddisplacements in short term after stopping the dewatering wells: a- vertical displacement, b- horizontal displacement

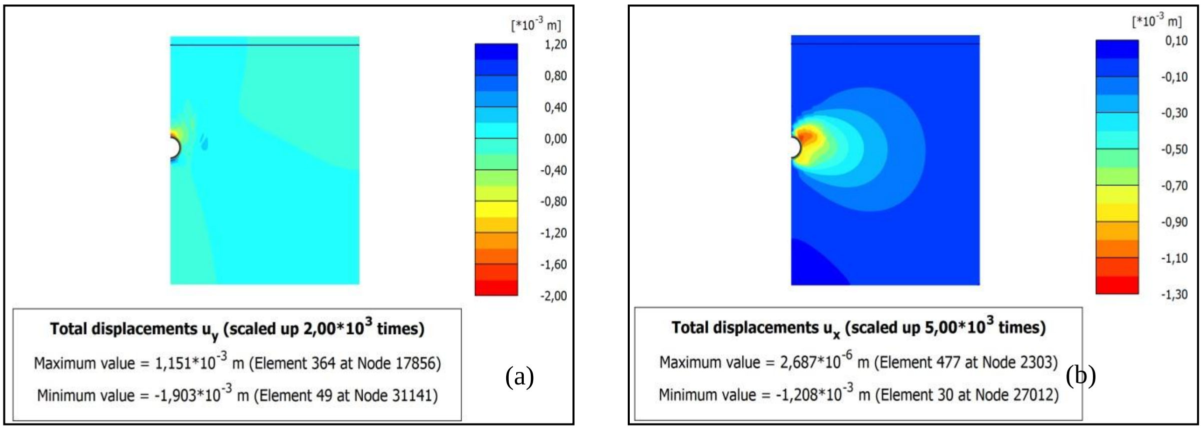


Fig.18. Induced displacements in the ground in long term with the level of the water table at its original level: a- vertical displacement, b- horizontal displacement

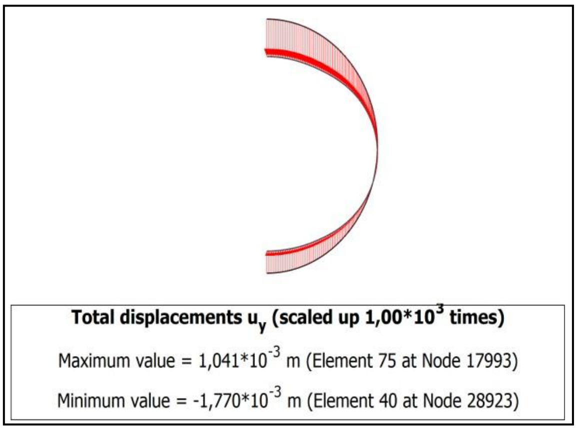
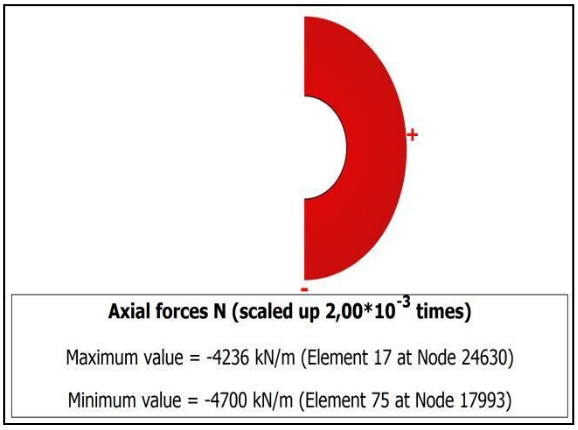


Fig.19. Induced vertical displacement in the final concrete liner



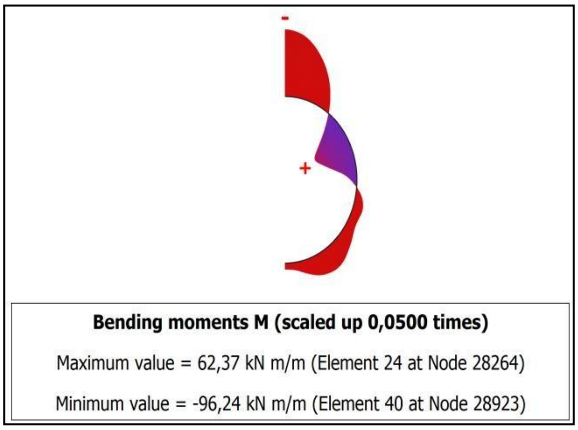


Fig.20. Axial force and bending moments in the final concrete liner once the shotcrete loses its strength

7. CONCLUSION

The choice of tunneling method generally depends on the type of rock or soil to be excavated in conjunction with the water table situation (water table level), the depth of the tunnel, and the geometry of the tunnel.

The tunnel boring method makes it possible to achieve projects in all types of geology, from very permeable soft ground to the hardest rocks, under heavy water load, and when stability conditions are not assured.The excavation of the section of the tunnel to be carried out is done in one step (global attack) and includes a system for protecting the walls of the excavation between the working face and the lining. The very rapid and efficient execution makes this method preferred but its availability, its very high cost and the section which must always remain circular with the same diameter constitute no less important limits to its use.

The new Austrian method (NATM) can be carried out using different methods of attack, depending on the quality of the soil encountered. Its use makes it possible to limit the deconfinement of the soil around the excavation but it does not make it possible to control hydrostatic pressures. which requires drying of the soil.

In the case of this study, the digging of the tunnel gave rise to disturbances in the initial state of stresses in the ground. The redistribution of the latter varies depending on the digging method and phase. In soil with poor characteristics as presented in the studiedcase, the frontal and lateral conservation of the excavation and the control of hydrostatic pressures are important.

Numerical modeling showed that excavation with the Tunnel Boring Machine gives rise to less significant settlements than those caused by excavation with the New Austrian method. The displacements obtained using the latter, which requires variation in the level of the water table, highlight the importance of hydrogeological conditions on the displacements likely to be induced when digging a tunnel in saturated massifs. It is possible to use other methods which make it possible to deal with the problem of the water table such as the use of the method of freezing the water around the tunnel or the use of the Jet-grouting method to consolidate and seal the ground around the tunnel. The choice between the two digging methods will necessarily depend on the availability of tunnel boring machines and the technical-economic study.

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