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Numerical Analysis of the Partial Collapse of a Twin-tubes Tunnel: A Case study

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Abstract- On the 1st of January 2014, the left tunnel of the twin-tube situated in the eastern part of the East-West Algerian highway, still under construction, was affected by a partial collapse, which induced significant damages over a distance of 120 m. Alongside, part of the right tube was also affected by the incident and suffered a breakdown in its outer concrete lining. The tunnel, referred to in this project as "T1", is 1895.5 m long, bored through Djebel El-ouahche (South of Constantine City) under an average covering of 100 m of complex claystone material. In this paper, a three-dimensional numerical modeling analysis was used to simulate the loading state and deformation pattern governing the structure of the tunnel during the partial collapse. The numerical results show that the failure mechanism was mainly triggered by a large displacements of the primary lining which was applied to the left tunnel without final reinforcements, and was insufficient to counteract the applied stresses, when the work stopped for a period of about 18 months. The lessons that could be drawn from this event is that: if the tunnel construction process has to be stopped for technical or administrative reasons, the primary lining, technically remains a temporary reinforcement solution, therefore, it should be immediately equipped with displacements sensor to ensure timely warning so that appropriate action can be taken.

***Keywords*:** Twin-tubes tunnel, partial collapse, numerical analysis

1. Introduction

Tunnels are considered as complex public work projects. In the last decades, many failures have occurred during the tunnel construction phase as a result of surrounding rock mass instability, such as working face collapse, support failure and excessive surface settlement [1]. Tunnel collapse can occur during the construction process of the tunnel (more frequently) or after putting the structure into service. According to most reported case studies, the major causes of tunnel collapses are embedded intricately in the tunnel construction process as well the existing ground conditions [2 - 6]. Recording, analyzing and understanding the causes of past tunnels collapses remains the most reliable approach to learn lessons, gain knowledge and take measures against their recurrence.

In the present work, a numerical model was developed using the Plaxis 3D Tunnel finite element package, in order to simulate the partial collapse which occurred in the left tunnel of a twin-tube, situated on the eastern part of the East-West Algerian highway, at that time, still being under construction. All the Data related to the twin tube tunnel "'T1" was collected by Salem [7] as part of a research project. The numerical analysis is intended to simulate the state of loading and deformations governing the structure of the tunnel during the construction process and to investigate the probable causes of the observed partial collapse. The failure criteria used herein is based on the deformations of the tunnel structure observed during the collapse.

2. Description of the Tunnel

2.1. Geographic location

The twin tubes tunnel considered "T1" is of about 1990 m long, bored through the mountain named Djebel El-Ouahche, situated in the south of the City of Constantine on the Eastern part of the East -West Algerian highway. The tunnel is located between the kilometric-points KP 205 + 393 m to KP 207 + 284,5 m for the left tube and between the KP 205 + 404,5 m to KP 207 + 299 m for the right tube, each tube is divided into 152 vaults of 12.5 m length each. Figure 1 shows the geographic location of the tunnel. In Figure 2 a layout of the twin tubes tunnel at the time of collapse, the collapse zone is highlighted.

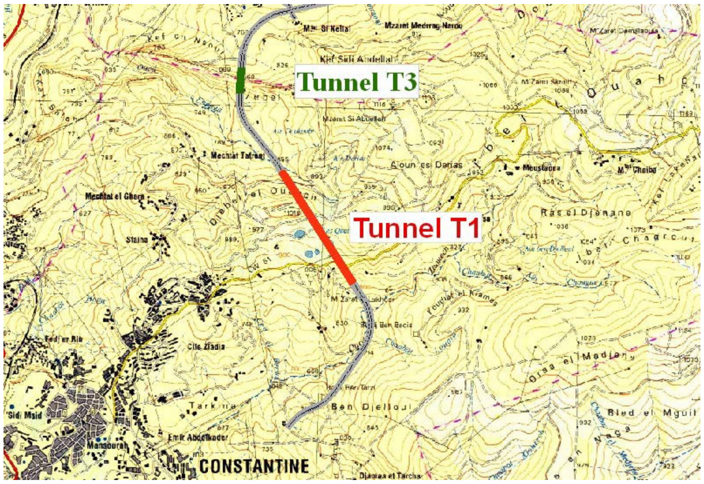


Fig. 1: Geographic location of the Tunnel "T1"

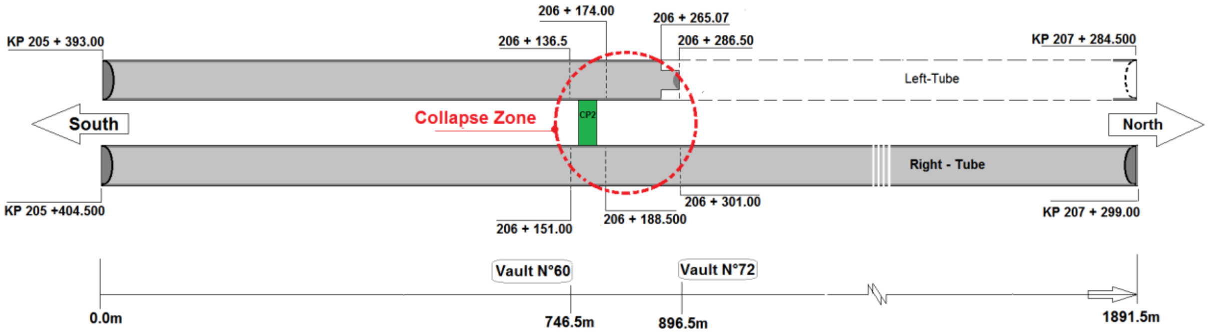
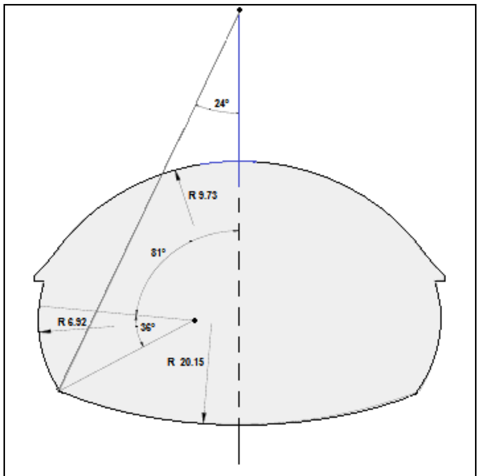


Fig. 2: Layout of the twin tubes tunnel

2.2. Geometry of the tunnel

Figure 3 and 4 show the shape and geometrical properties of each tunnel. As can be seen, the cross section of the 2 tunnels (tubes) are identical, they are oval shaped with 3 radius sections. The lateral distance between the two tubes is 22 m, with 3 communication cross passage.



Sections	Radius (m)	Angle (°)
Cap	9.73	81
Stross	6.92	36
Invert	20.15	24

Fig. 3: Shape and geometry of the tube

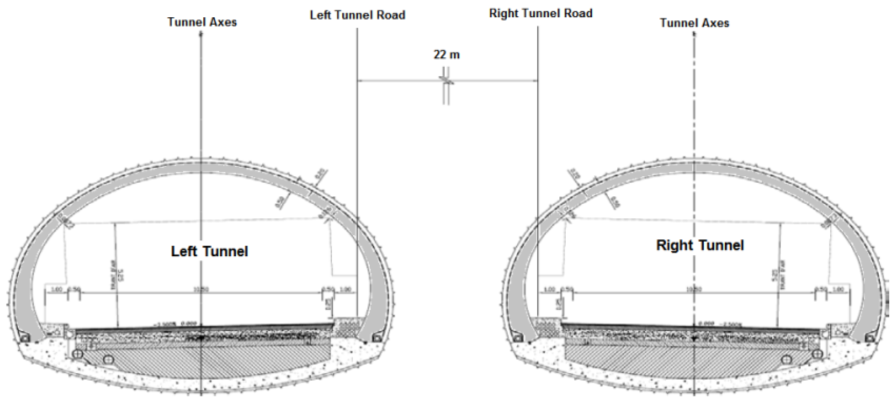


Fig. 4: Twin Tube Tunnel cross section

2.3. Stratification above the collapse zone

The stratigraphic sequence in the area of interest mainly consists of a layer of about 100 m of complex geology of predominantly claystone material. Only the layer over the collapse segment is considered in the present analysis.

2.4. Excavation method and supporting system

The excavation was carried out in accordance with the principles of the New Austrian Tunneling Method (NATM) improved by frontal and radial reinforcements. The primary or temporary support lining of the tunnels was made of a shotcreting layer of 400 mm thickness, reinforced by a netting of welded mesh and steel-HEB200 beams. The outer or final lining is made of the primary lining with additional radial reinforcement using anchors bolts and frontal reinforcement with FIT fiberglass, the resulting final reinforced lining has a thickness of 600 mm. The anchor bolts used are of the type Store Norfors (SN), secured in borehole made by a mortar fill. This reinforcement procedure is widely used in tunneling and in specialized civil engineering projects. The fiberglass is used as a permanent ground support application. Reinforced concrete with fiber glass is known as the ultimate solution to construct projects with strong mechanism against corrosive agents. Figure 5 shows the type of lining applied to the tubes vaults before the partial collapse.

3. Brief description of the collapse zone

On the 1st of January 2014, the twin-tube highway tunnel "T1" was affected by a partial collapse over a linear distance of about of 120m. The right tube was already completed and opened to the traffic for four months since September 2013, while the left tube was still being excavated after a long halt of 18 months. Fortunately, there are no reports of injuries in this event. The significant damages found on the right tube are characterized by the breakdown of the final concrete lining. As shown in Figure 2, the zone of collapse is located between the vaults N° 60 and N° 72. It includes the cross-passing N° 2 (CP2), situated in the middle of the vault N° 63. The collapse area started from the vault N°63 towards the north to the vault 72. Figure 6 summarizes the situation of the vaults corresponding to each tube after the partial collapse.

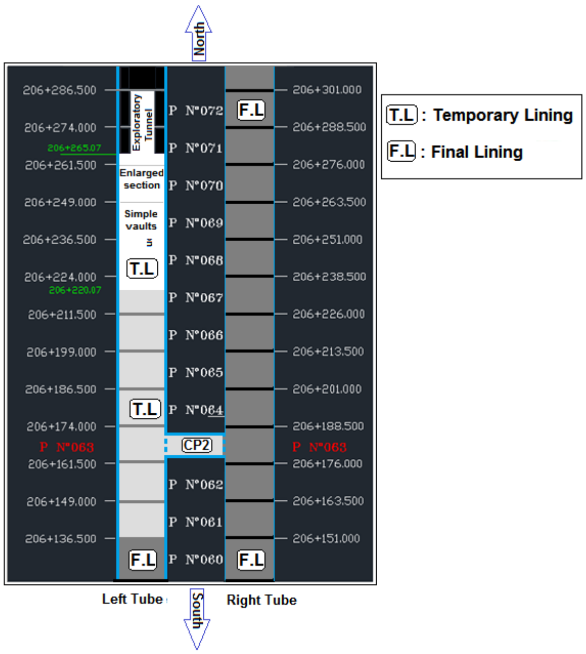


Fig. 5: Top view of the twin tubes tunnel

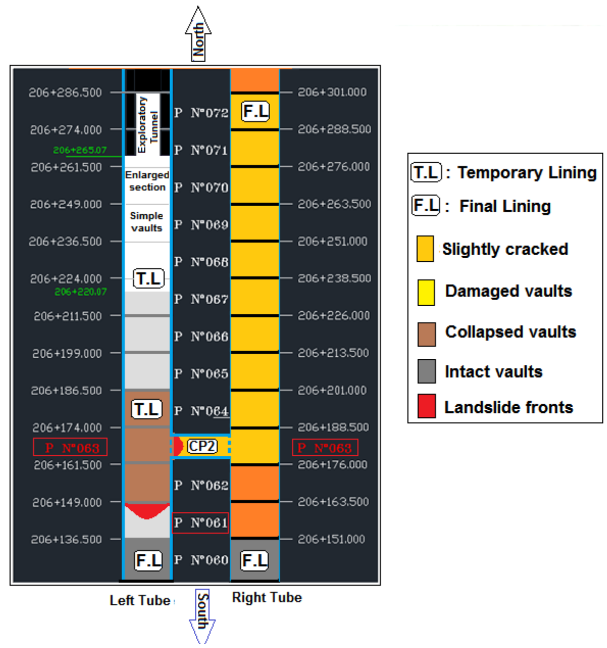


Fig. 6: Top view of the Tunnel -after the partial collapse

4. Numerical modelling analysis

Three dimensional numerical modeling of tunnels is known to be a complex and challenging task, as it involves many computational parameters, however, it remains the best approach to simulate the behavior of tunnels in an underground environment and could thus avoid the limitations of 2D modeling [8-9]. Failure or large deformations such as collapse and mass movement into or a around tunnel are difficult to simulate numerically. The failure criteria used herein to simulate the partial collapse is based on the analysis of the deformations of the tunnel structure (primary and final lining) observed. and the computation of the stability safety factor.

A numerical model was developed using the 3D Tunnel Plaxis program, taking into account the geometry and dimensions of the twin tubes tunnel, the loading conditions, material types and the boundary conditions. To limit the computation time, and instead of modeling all the length of the twin tube tunnel (1891.5 m), only the area (segment) concerned with the partial collapse has been analyzed in the present analysis: starting from the vault N° 60 to the vault N°73, over a distance of about 180 m (as shown in Figure 6).

4.1 Geometry and boundary conditions

The model dimensions are x = 106 m, y = 120 m and z = 176,5 m. The length in the (z) axis corresponds to the that of 12 vaults (12.5 m each) where the effects of collapse have been observed on both tubes. The modeling mesh data adopted in the finite element computation for the soil are based on a medium coarseness mesh, 15 nodes wedge elements leading to 9432 elements, 28175 nodes and 56376 stress points. In the (z) direction the tunnel was modeled with 15 parallel planes, and 14 slices, each corresponding to a length of a vault. In the collapse zone, the cross passage located in the vault N°63, it was modeled with 4 parallel planes. Typical 3D finite elements model is presented in Figure 7. In Figure 8 a partial geometry model, with deactivated soil clusters, is shown with numbering of the slices corresponding to each vault.

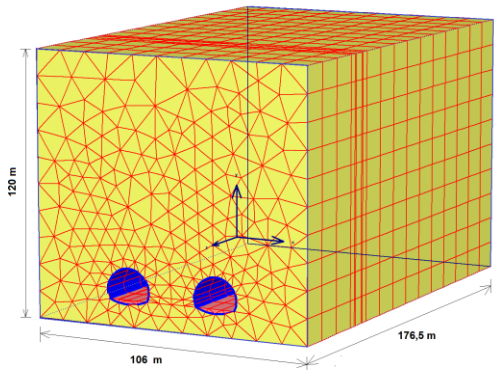


Fig. 7: 3D Finite element mesh

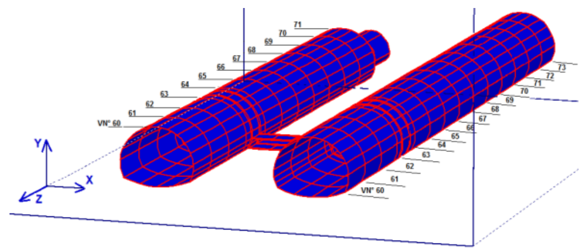


Fig. 8: Partial geometry model

4.2 Materials modelling parameters

The properties of the soil used in the Mohr - Coulomb model are shown in Table 1, where γunsat and γsat are the soil unit weights; E is Young's modulus; ϕ and c are the soil frictional angle and cohesion respectively; ν is the Poisson ratio. During the stressing displacements of the tunnel lining , it is evident that the contact of the soil with the tunnel surfaces remains permanent, therefore, interfaces elements with rigid strength apply with corresponding default value of (R_{inter} = 1).

Table 1. Soil properties for numerical modelling

Material	γ_{unsat} [kN/m^3]	γ_{sat} [kN/m^3]	E_{ref} [kN/m^2]	C_{ref} [kN/m^2]	ν -	ϕ [^{o}]	ψ [^{o}]
Soil	18.5	21.40	300000	52	0.3	22	0

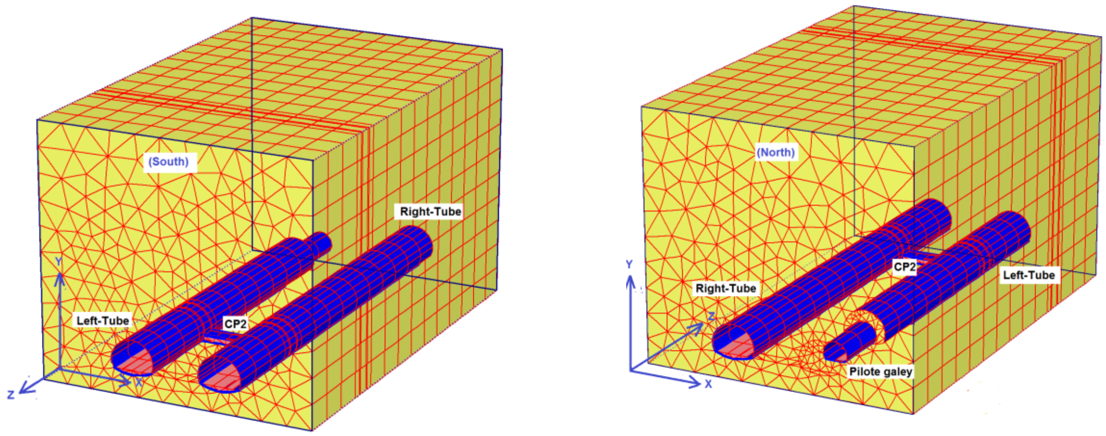
The tunnel structure was modeled using the plate-3D Tunnel option of the Plaxis and the lining was modeled as an elastic material. Table 2 presents the modeling parameters of the primary and final tunnels lining used in the numerical calculation. The materials strength of the linings is presented by the equivalent axial rigidity *EA* and the equivalent flexural rigidity *EI*.

Table 2: Modeling parameters of the tunnel linings

	Materials	Thickness (mm)	Material type	EA (kN/m)	EI (kN/m^2/m)
Primary	Shotcreting + Welded-mesh + HEB200	400	Elastic	1,284*10^7	1,706*10^7
Final lining	Primary + Anchor bolts + Fiberglass	600	Elastic	1,680*10^5	5,040*10^5

4.3 Calculation phases and types

In this analysis, 3D plastic calculations were performed and two calculation phases were defined. In the first phase a load advancement ultimate level procedure was performed until collapse of the soil or prescribed ultimate state is fully reached. In this phase all elements of the numerical model (soil + tunnels + interfaces) are activated, as indicated in Figure 9 (a) and (b) corresponding to the North and South side views. For the left tube the appropriate lining (primary) was considered in the structure, however, the outer or final lining was only attributed to the right tube. Model elements were activated in order to simulate as close as possible the state of the project (geometry, lining, etc...) at the time of collapse including the modeling of the pilot gallery (Figure 9 b). In the second calculation phase a load advancement number of steps with phi-c reduction procedure was performed to check out the global stability of the tunnel. This option is most suitable for safety analysis and the cases where a failure is expected during the loading. In this phase the safety factor is computed.



a- South-side **b-** North-side

Fig. 9: Numerical model for calculation

5. Numerical analysis results

5.1 Displacements in the soil

The first calculation phase carried out is a plastic calculation, the deformation computed in this phase shows large settlements of the soil mass in left side of the left tube. Figure 10 shows the scaled arrows of the soil displacement. It is clear that large concentration of deformation is observed in the upper part of the left tube, precisely at the vault 63 corresponding to the cross passage. In Figure 11 a partial geometry of the deformed model is presented with the forward slices of soil were deactivated. It can be seen that large total displacements (of up to 1.19 m) were concentrated in the vault N°63, which explain the failure mechanism, that was mainly triggered by a large displacements (caving in) of the primary lining which was applied to the left tunnel without final reinforcements, and at long-term, was insufficient to counteract the applied stresses.

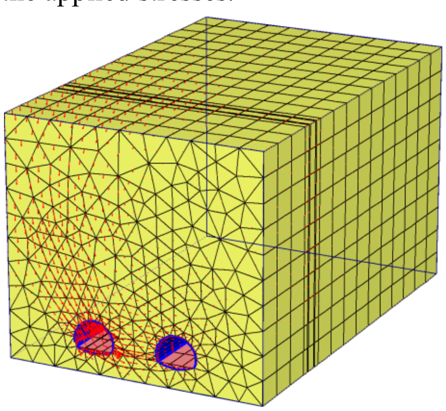


Fig. 10: Total displacements in the model

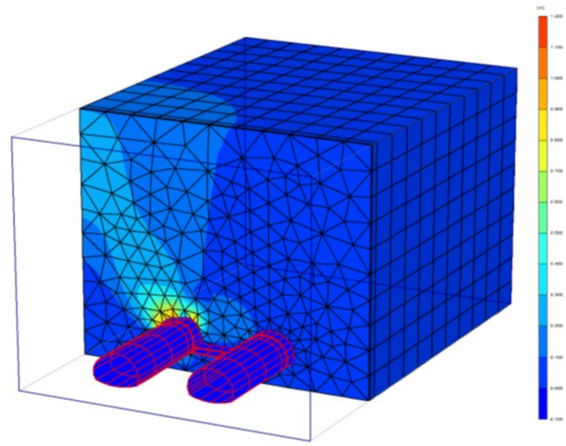
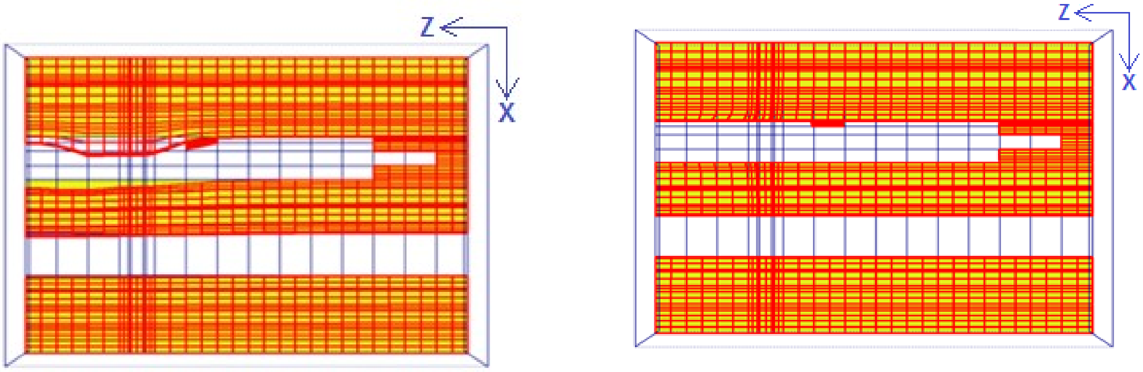


Fig. 11: Total displacements (partial geometry)

5.2 Deformations (collapse) of the tubes

In Figure 12 the tube's horizontal displacements are presented with deformation planes in z-x plane (top view), the horizontal displacements following the x- axis of the left tube (Figure 12a) comply with the partial collapse observed in the left tube, as it concerns mainly the large displacement (deformations) of the vaults N° 61 to 64 (see Figure 6). In z-direction (Figure 12b) horizontal displacements of more than 300 mm are computed which confirms the frontal displacement (collapse) observed in the vault N°63.

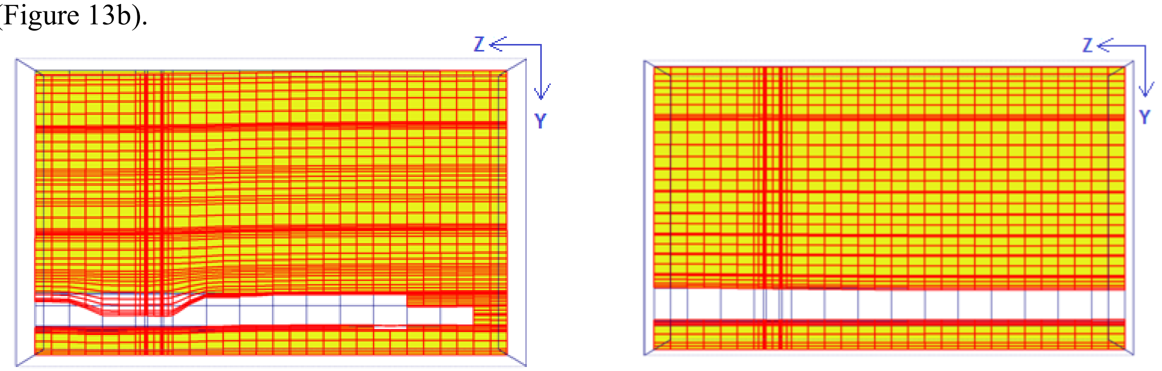


a- Horizontal displacements (x)
(x_{max}: 821.83 mm)

b- Horizontal displacements (z)
(x_{max}: 342,13 mm)

Fig. 12: Horizontal displacements of the vaults in the partial collapse segment.

In Figure 13 the vertical displacements are presented with deformation planes, following the direction z-y (side view), corresponding to two vertical cross section of the model at the top headings of the left and right tubes respectively. As for the horizontal displacements, a concentration of vertical displacement (caving in) is clearly visible if the left tube at the vaults N° 62 to N° 64 with a maximum values of 868.02 mm (Figure 13a) which confirms the collapse of the tunnel structure under the applied soil stress. However, in the right tube the vertical displacements are very small compared to the left tube (Figure 13b).



a- Vertical displacement -left tube (y)
(y_{max}=868.02 mm)

b- Vertical displacement -right tube (y)
(y_{max}= 37.82 mm)

Fig. 13: Displacements (top view) deformation planes

Figure 14 shows a top view of a partial geometry model where total horizontal and vertical displacements of the tubes (including the cross-passage) are presented in shadows form to highlight the displacements area of the tubes. It could be seen that most of the computed displacements confirm the fact that the partial collapse occurred in the segment related to vaults N° 61 to N°65 and that the cross passage has played a major role in the transmission of displacements (and overburden stresses) from the left tube to the right tube.

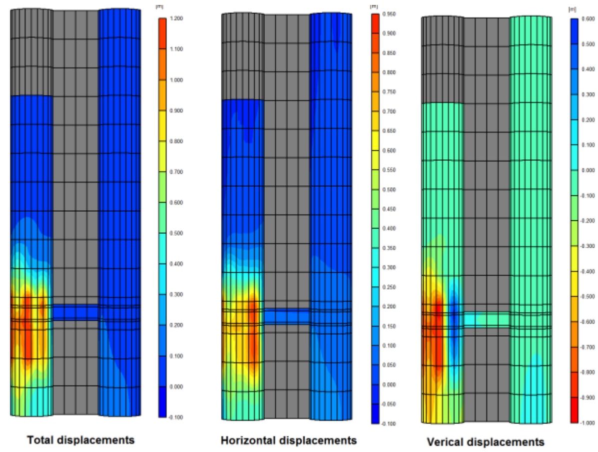
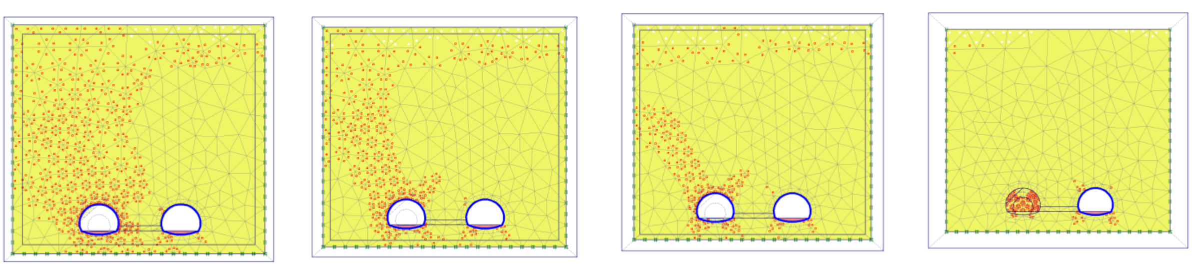


Fig. 14**:** Displacements in the tunnels (top view)

5.3 Plastic points in the soil mass

In Figure 15 selected vertical planes are presented, corresponding to the vault-faces N° 62, 63, 64 and 71 respectively. In this Figure the calculated plastic points are shown, which indicates the extension of the failure state in the soil mass at some stress points. Most of the failure is concentrated around the left tube, due probably to the large displacement observed in the soil mass in this areas and which is possibly caused by the collapse of the left tunnel shield under the load of the soil. The cracks noticed in the right tube mainly at its invert is due solely to the horizontal displacement (following x-direction) of the cross passage towards the right tube. For the vault-face N°71 plastic points indicate a possible collapse in the vault-face of the pilot gallery.



Plane N° 62 Plane N° 63 Plane N° 64 Plane N° 71

Fig.15: Plastic field in the soil - selected vaults-faces

5.4 Stability analysis

The computation methods of the safety factors is based on the strength reduction theory. Engineering application indicates that the method of underground tunnels strength reduction is very useful for analysis quantitatively the general stability of tunnels [10]. Figure 16 shows the selected nodes *A* and *B* corresponding to the top headings of both tubes and Figure 17 presents the computed safety factor corresponding to the total displacements of nodes *A* and *B*. As can be seen, calculations showed that at first stage safety factor was very close to 1.0 which indicates problems of the top heading face stability, then the safety factor decreases significantly when displacements of the top-headings occurred, which confirms the large plastic deformation of the linings, and the partial collapse.

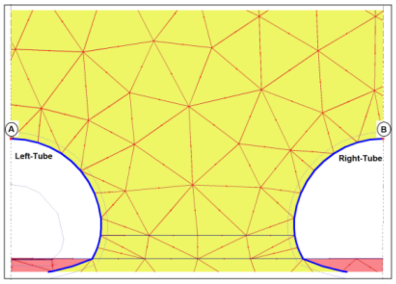


Fig.16: Selected nodes for stability analysis

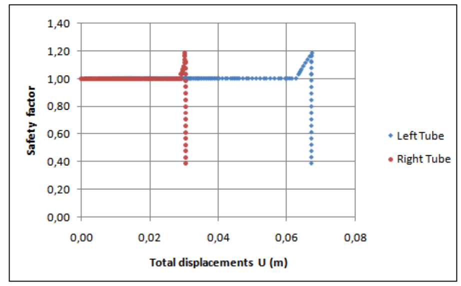


Fig. 17: Safety factor multiplier versus total displacements (point A and B)

6. Discussion

According to most guidelines for designing tunnels, an underground structure may lose its serviceability or its structural safety if there is exhaustion of the material strength of the system causing intolerably large deformations. Failure of the lining is amongst the criteria of tunnels failure causes [11]. The present numerical simulation shows that the primary lining of the left tube (over the collapse segment) erected to support the applied vertical and lateral stress from the soil, developed large deformations in a restrained manner over time, creep must have occurred leading to the collapse of the whole structure. The resulting lining failures caused excessive settlement above the tunnel which has overburden the vaults Number 62 to 64 of the left tunnel, causing large displacements of the structure elements over that segment, the cross passage has transmitted the stresses and deformations and caused cracks in the right tube.

In normal NATM applications the outer lining liner is not considered to be a load bearing element, but provides extra structural safety. In the case of the left tube of the 'T1' Tunnel, however, if the final lining, which provided ring closure and consisted of reinforcements as well, could have been constructed timely, it appears that the partial collapse could have likely been prevented. however, if the tunnel construction process has to be stopped for technical or administrative reasons, the primary lining, technically remains a temporary reinforcement solution, therefore, it should be immediately equipped with displacements sensor, and continuous monitoring of the displacements of the support system should be recorded and analyzed regularly to prevent any uncontrolled displacements (or deformations) of the temporary lining that could inevitably lead to caving in and consequent partial or total collapse of the tunnel.

7. Conclusion

In this paper a numerical approach was used to simulate the state of loading and deformations governing the structure of a twin tunnel during its partial collapse. The practical importance of this analysis is that: it could be relevant to future project, in assessing risks related to uncontrolled displacements of linings leading to structural failures which would support the contractual aspects of risk sharing and responsibilities. Out of this case study, the following conclusions are drawn:

- The behavior of the primary lining or support applied to some parts of the left-Tube over the segment where the partial collapse occurred is time dependent and its initial design- strength or rigidity ensuring the stability of the tunnel should be continuously checked during the construction stage. Primary lining might develop large deformations in a restrained manner over time leading to large settlement of the surrounding soil mass and inevitable collapse of the structure.
- If the tunnel construction process has to be stopped for technical or administrative reasons, the primary lining, technically remains a temporary reinforcement solution, therefore, it should be immediately equipped with displacements sensor, and continuous monitoring of the displacements of the support system should be recorded and analyzed regularly to prevent any uncontrolled displacements of the lining that could inevitably lead to caving in and consequent partial or total collapse of the tunnel.

References

[1] Yang, J.H., Wang, S.R., Wang, Y.G. and Li, C.L. 2015. Analysis of Arching Mechanism and Evolution Characteristics of Tunnel Pressure Arch. Jordan Journal of Civil Engineering, 9(1): 125-132.

[2] Fraldi, M. and Guarracino, F. 2011. Evaluation of impending collapse in circular tunnels by analytical and numerical approaches. Tunneling and Underground Space Technology, 26: 507-516.

[3] Ahmed, M. and Iskander, M. 2012. Evaluation of tunnel face stability by transparent soil models. Tunnelling and Underground Space Technology. 27(1): 101-110.

[4] Sousa, R. L., and Einstein, H. H. 2012. Risk analysis during tunnel construction using Bayesian Networks: Porto Metro case study. Tunnelling and Underground Space Technology 27: 86–100.

[5] Chen, R.P, Li, J.b., Kong, L.G. and Tang, L.-J. 2013. Experimental study on face instability of shield tunnel in sand. Tunnelling and Underground Space Technology. 33: 12-21.

[6] Aliabadian, Z., Sharafisafa, M., Nazemi, M. and Khamene, A. R. 2015. Numerical analyses of tunnel collapse and slope stability assessment under different filling material loadings: a case study. Arab J Geosci, 8:1229–1242.

[7] Salem, S. 2014. Etude de stabilité et de dimensionnement du tunnel autoroutier T1 (djebel ouahch - Constantine) et analyse de l'effondrement. Master Thesis. University of Tébessa.

[8] Vlachopoulos, N., and Diederichs, M. S. 2014. Appropriate Uses and Practical Limitations of 2D Numerical Analysis of Tunnels and Tunnel Support Response". Geotechnical and Geological Engineering. 32(2): 469-488.

[9] Dasari, G. R., Rawlings, C.G. and Bolton, M.D. 1996. Numerical modelling of a NATM tunnel construction in London Clay. Geotechnical Aspects of Underground Construction in Soft Ground. Mair and Taylor (editors). BALKEMA, 491-496.

[10] C. P. Shi, X. T. Feng, Q. Jiang,. 2012. Calculation of Underground Tunnels' Safety Factor Based on the Strength Reduction Technique, Advanced Materials Research, Vols. 594-597, pp. 1206-1209, 2012

[11] ITA working group on general approaches to the design of tunnels. 1988. Guidelines for the design of tunnels. Feature report. Tunnelling and underground space Technology. Volume 3, Number 3,.237-249.