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ROUILI, A., TOUAHMIA, M. and DJERBIB, Youcef

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Numerical analysis of the partial collapse of a twin-tubes tunnel

Ahmed Rouili[1], Mabrouk Touahmia[2] and Youcef Djerbib[3]

[1] *Faculty of Sciences and Technology,*
University of Tebessa,
Tébessa, Algeria

[2] *University of Hail,*
College of Technology,
Hail, Saudi Arabia

[3] *Sheffield Hallam University,*
Sheffield, United Kingdom.

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. In this paper, a 3D numerical simulation was used to investigate the loading state and deformation pattern governing the structure during the partial collapse. The results shows that the failure 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. Practical recommendations that could be drawn from this case are suggested.

1   Introduction

Tunnels are considered as complex public work projects that require sophisticated engineering methods, tools and techniques to be designed, analysed and constructed safely. 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, analysing 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 analyze using Plaxis 3D Tunnel finite element package is used 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. The state of loading and excessive deformations governing the structure of the tunnel during the partial collapse are investigated.

2   Description of the tunnel

The twin tubes tunnel "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 layout of the tunnel, the collapse zone is highlighted.

Figure 2 and 3 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.

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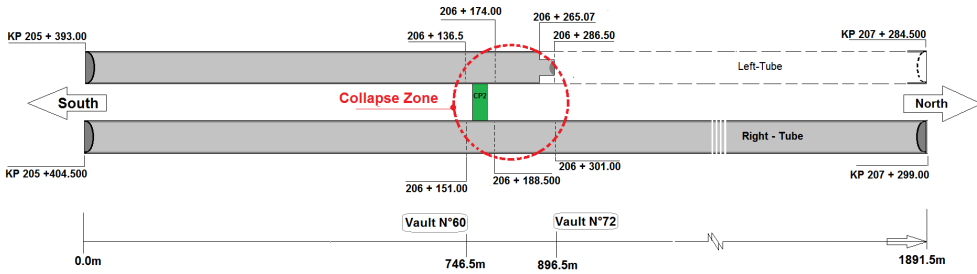

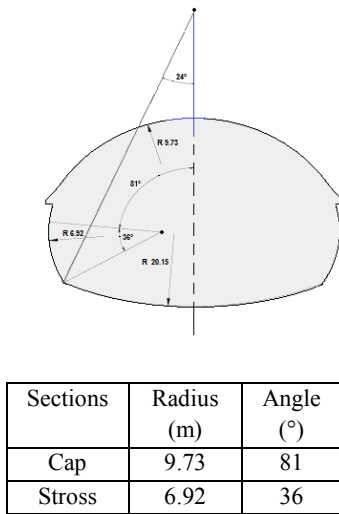Fig. 1   Layout of the twin tubes tunnel.

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

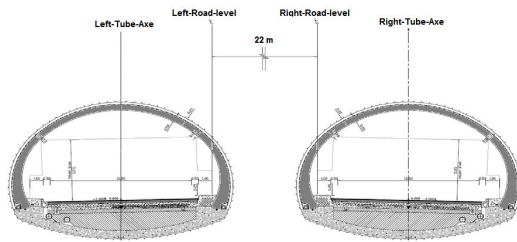Fig. 2   Geometry of the tube.

Fig. 3   Twin Tube Tunnel cross section.

3   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.

4   Description of the collapse

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. Figure 4 summarize the situation (lining) of the tunnels before the collapse, 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. After the partial collapse, the significant damages found on the right tube are characterized by the breakdown of the final concrete lining. As shown in Figure 5, 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.
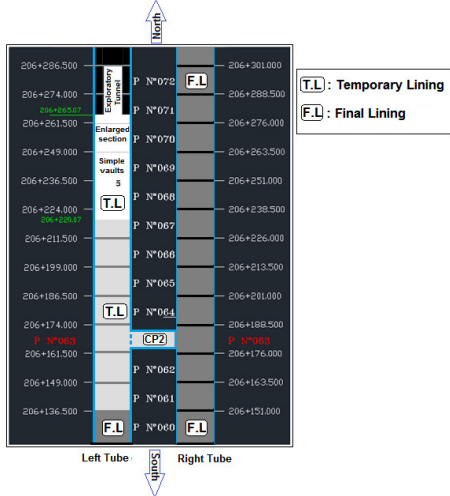

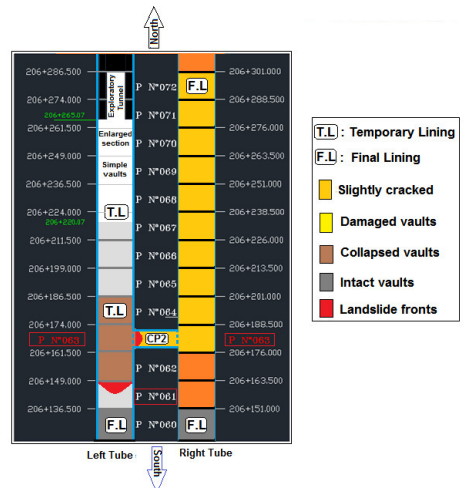
Fig. 4   Top view of the twin tubes tunnel.

Fig. 5   Top view of the Tunnel -after the partial collapse.

5   Numerical 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 [7-8]. 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. 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 investigation: starting from the vault N° 60 to the vault N°73, over a distance of about 180 m (as shown in Figure 6).

5.1   Numerical model

5.1.1   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 6. In Figure 7 a partial geometry model, with deactivated soil clusters, is shown with numbering of the slices corresponding to each vault.
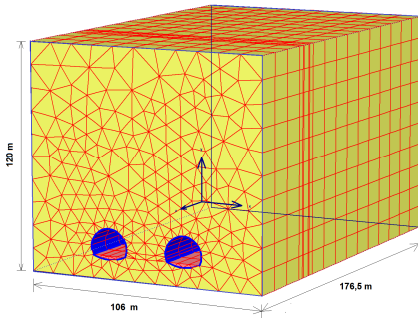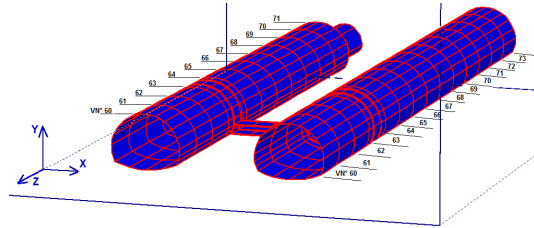
Fig. 6   3D Finite element mesh.   Fig. 7   Partial geometry model.

5.1.2   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 modeling.

Material	γ_{unsat} [kN/m³]	γ_{sat} [kN/m³]	E_{ref} [kN/m²]	C_{ref} [kN/m²]	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/m²/m)
Primary	Shotcreting + Welded-mesh + HEB200	400	Elastic	1,284*10⁷	1,706*10⁷
Final lining	Primary + Anchor bolts + Fiberglass	600	Elastic	1,680*10⁵	5,040*10⁵

5.1.3   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 8 (left and right) 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 8-right) 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.
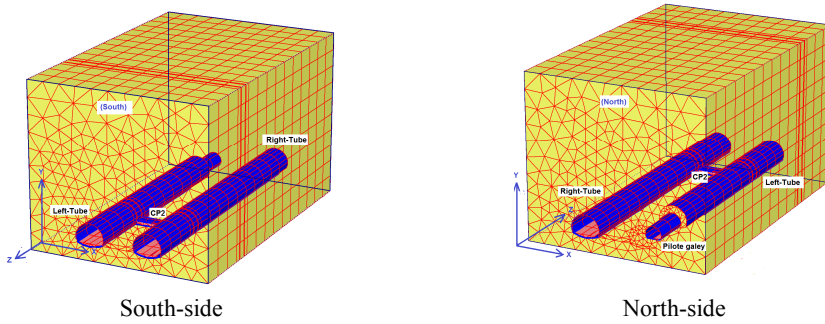

South-side   North-side

Fig. 8 Numerical model for calculation.

5.2   Numerical results

5.2.1   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 9 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 10 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.
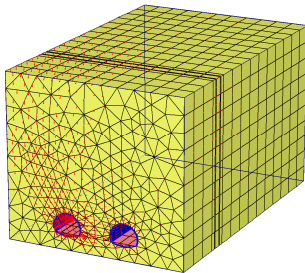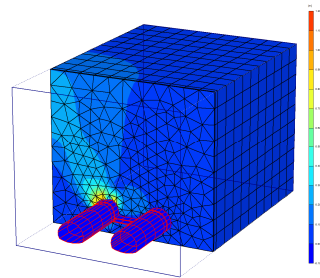

Fig. 9 Total displacements in the model.   Fig. 10 Total displacements (partial geometry).

5.2.2   Deformations (collapse) of the tubes

In Figure 11 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 presented in Figure 11(left) 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 5). In z-direction , Figure 11 (right) horizontal displacements of more than 300 mm are computed which confirms the frontal displace-ment (collapse) observed in the vault N°63.

Horizontal displacements (x)
(max: 821.83 mm)

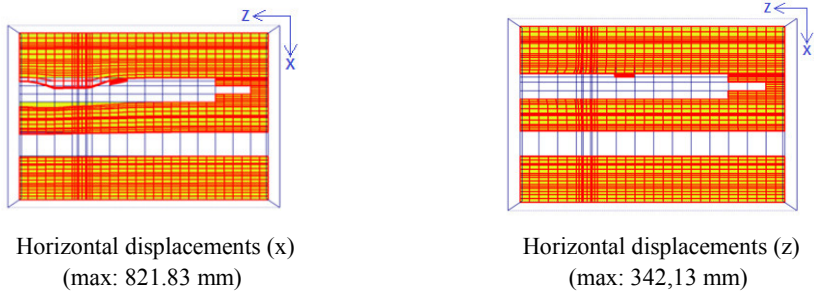Horizontal displacements (z)
(max: 342,13 mm)

Fig. 11 Horizontal displacements of the vaults in the partial collapse segment.

In Figure 12 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 12 (left)) 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 12 (right)).

Vertical displacement -left tube (y)
max=868.02 mm

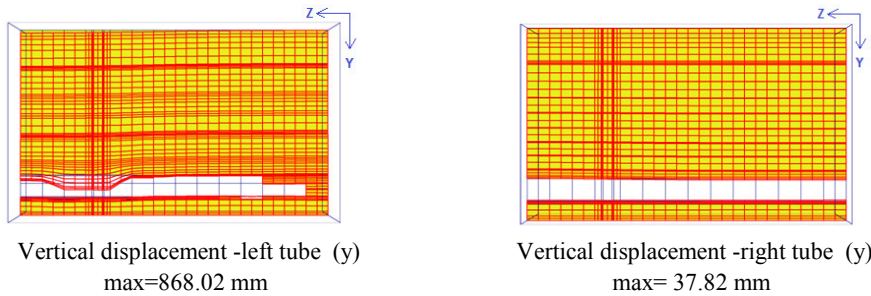Vertical displacement -right tube (y)
max= 37.82 mm

Fig. 12   Displacements (top view) deformation planes.

Figure 13 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.
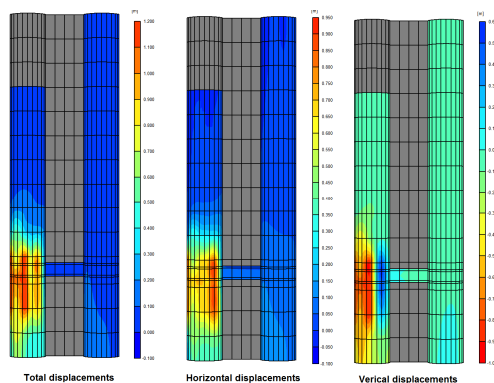

Total displacements   Horizontal displacements   Verical displacements

Fig.13   Displacements in the tunnels (top view).

5.2.3   Plastic points in the soil mass

In Figure 14 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.
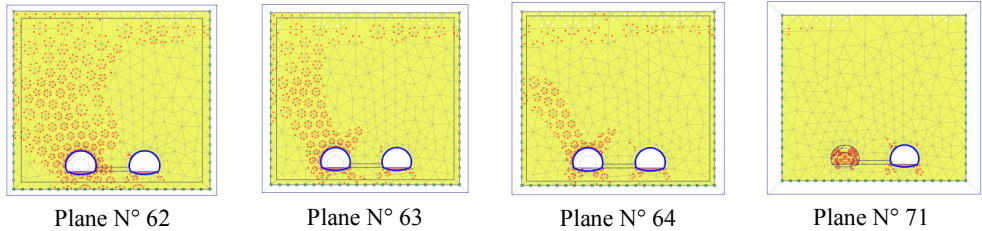

| Plane N° 62 | Plane N° 63 | Plane N° 64 | Plane N° 71 |

Fig. 14   Plastic field in the soil - selected vaults-faces.

6   Disscussion

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 [9]. 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 modeling analysis was used to simulate the state of loading and deformations governing the structure of the tunnel during the 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.

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THE 12th fib INTERNATIONAL PhD SYMPOSIUM IN CIVIL ENGINEERING

PROCEEDINGS

Continuing the tradition of the fib International PhD Symposium in Civil Engineering, the 12th edition was held at The Czech Technical University in Prague, Czech Republic, from August 29 to 31, 2018.

The event was held under the patronage of the International Federation for Structural Concrete (federation internationale du beton – fib), one of the key international associations that disseminate knowledge about concrete and concrete structures.

This document summarizes all presentations that took place at The Czech Technical University in Prague. The downloadable proceedings will be publicly available after the event.

The main topics of the symposium were related to the general field of civil engineering infrastructure, including but not limited to:

- Advanced materials
- Innovative structures and details
- Construction technology
- Structural analysis and design
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- Monitoring and structural assessment
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