FIRE SAFETY ASSESMENT OF TUNNEL STRUCTURES

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1. ABSTRACT

Tunnels and underground structures are becoming more and more essential these days, when installing new transportation infrastructures in congested areas as well as when raising the qualities within the existing urbanizations. The realization of such structures calls for specific measures regarding durability provisions, commitment to environmental aspects, issues of sustainability and safety assurance, for their whole lifecycle. The design for safety of tunnel infrastructures is a multifaceted process, since there are many aspects that need to be accounted for, regarding different aspects (e.g. structural and non structural, organizational, human behavior). This is even more truth for the fire safety design of such structures. Fire safety in tunnels is challenging because of the particular environment, bearing in mind also that a fire can occur in different phases of the tunnel's lifecycle. Plans for upgrading fire safety provisions and tunnel management are also important for existing tunnels. In this study, following a brief introduction of issues regarding the above mentioned aspects, the structural performance of a steel rib for a tunnel infrastructure subject to fire is assessed by means of nonlinear analyses for different fire exposures.

2. INTRODUCTION

Over the last 20 years, there has been an increase in the construction of both road and rail tunnels. Europe in particular, not only has some of the world's longest tunnels in operation, but many more are under construction. The Brenner base railway tunnel (55 km) and the Mont d'Ambin base railway tunnel (57 km) are two noteworthy examples.

On the other hand, again with reference only in Europe, over the period of just 2 years, some notable disasters brought the problem of tunnel fire safety in evidence [1]:

- Mont Blanc tunnel, Italy/France, 1999: 39 victims.
- Tauern tunnel, Austria, 1999: 12 victims.
- Gletscherbahn 2 (Kitzsteinhorn funicular tunnel), Austria, 2000: 155 victims.
- Gotthard Road Tunnel, Switzerland, 2001: 11 victims.

Added to this human tragedy, is the damage to the tunnel structures and installations and the service disruption, with significant economic consequences. The Great Belt fire (during construction) and Channel Tunnel fires (three fires, the last occurring in 2008, were significant enough to close the tunnel) while resulting in no loss of life, have nevertheless caused major structural damage and financial loss.

After this series of dramatic accidents, the public opinion has turned its attention to tunnel fire safety. The European Union in particular, issued the Directive 2004/54/EC concerning the minimum safety requirements for tunnels in the trans-European road network [2] and as a consequence, several research projects on the topic were commenced [3], something that eventually led to the revision of national safety codes and standards. More recently, in a report commissioned by the European Parliament [4] twenty five recommendations are given for the safety assessment of tunnels.

Nowadays, the implementation of fire safety engineering (FSE) concepts [5] in the design of this kind of projects significantly enhances the design process by adding flexibility to the design parameters used in the project such as occupant egress facilities, ventilation requirements and material selection.

In a FSE complying strategy, a number of objectives are identified (safety of life, conservation of property, continuity of business operations, preservation of heritage, etc.). These (qualitative) objectives must be characterized by setting specific performance criteria. Regarding in particular the safety of life, the principal aim is to ensure the necessary time for the safe evacuation.

The structural performance of the structure under fire can be assessed with the implementation of analytical and computational tools, tools that require a very good understanding of the fire phenomenon.

Focus in this study is given to the consequences of fire to the structural elements, and more specifically, to the steel rib for a tunnel construction. Even though it is known that direct structural consequences of structural failure due to fire are not the primary safety reasons for tunnel occupation, and most research on the topic is focused to other issues (evacuation, smoke evolution, system response, mitigation etc), it is still an important aspect. This is due to several facts, primarily:

a. economic losses and social impact as a consequence of structural failure are nevertheless important, especially for critical transportation infrastructures;
b. structural safety during the fire evolution can be important for the evacuation.

Thus, numerical simulations for the load bearing capacity of tunnel structures under fire (see for example [6], for RC tunnels) and of the spalling effect (see for example [7]), are an important part to the fire safe design of a tunnel structure.

In the next paragraph, and in order to frame the problem in an appropriate manner, an overview of the system approach to tunnel fire safety is provided.

3. THE SYSTEM APPROACH TO TUNNEL FIRE SAFETY

In order to address the tunnel fire safety in a holistic manner, it is important to operate within a system engineering methodology.
Following Figure 1, it is essential to divide the safety objectives in different phases:

- Prevention. All those measures aiming at reducing the risk for a tunnel fire, including risk mitigation measure.
- Suppression. Including measures aiming at the tunnel evacuation (detection, warning, control, evacuation).
- Fire mitigation, should the fire is not suppressed.
- Measures for the fire safety design, aiming at:
  - The structural integrity of the structure, intended as the absence of structural failure. Therefore this step concerns the structural state, in the sense that the maximum grade of structural integrity is related to the nominal configuration of the structure, i.e. the undamaged one [8].
  - The global safety of the structure, related with no load bearing parts of the structure (e.g. to avoid the collapse of the tunnel lining due to spalling).
- Robust design of the structure, intended as the ability of a structure to sustain local failure [9] without developing a major collapse.

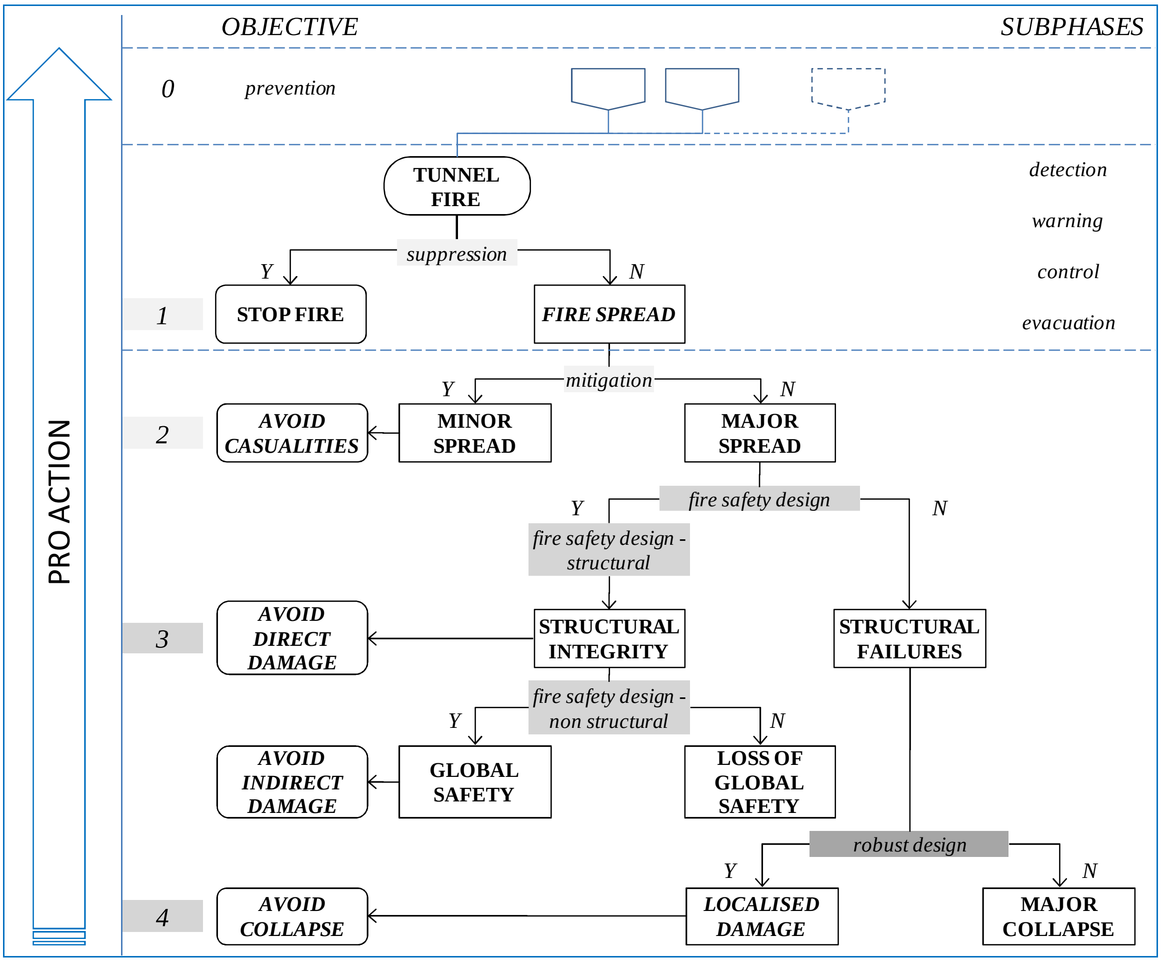

Fig. 1 System approach to tunnel fires

In the following paragraph, the structural integrity of a steel rib for a tunnel is inquired.

4. STRUCTURAL ANALYSIS OF A STEEL RIB UNDER FIRE

In this paragraph, the structural behavior of a steel rib for a tunnel construction is investigated, accounting for different adverse fire events. The performed analyses (implemented in a commercial FEM code) account for the material and geometry nonlinearities, thus being able to accurately describe the actual behavior of the structure.
Different time-temperature curves are implemented, representing different exposure conditions, developed over the last decades, both in real (disused) tunnels and laboratory conditions. The idea is to model various types of fires for underground structures that would result in different combustion rates, duration, and peak temperature. A brief overview is given in [10] while design curves are methodically discussed in [11].
In this study, four different curves are implemented in particular (Figure 2), a. the cellulosic curve, b. the RWS curve and c. the RABT ZTV car and train curves.

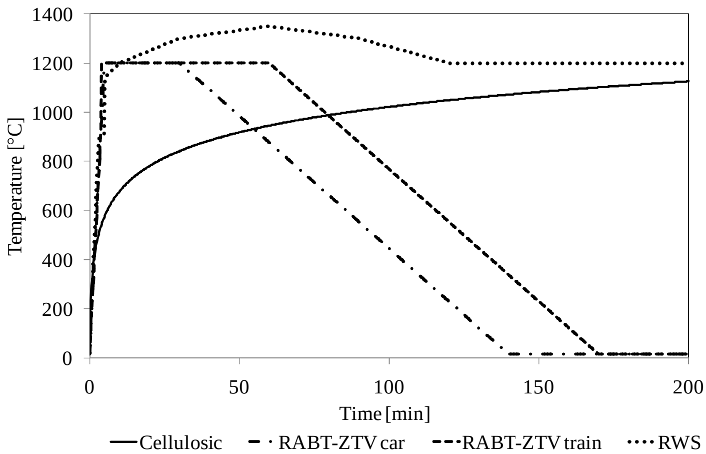

Fig. 2 Fire exposure curves

For the purpose of the analyses, the time-temperature load has been protracted from 180 min. to 200 min, while the temperature load is applied uniformly to the entire steel structure, without accounting for site specific fire loading effects [12], e.g. ventilation.

4.1 Details of the steel rib and FEM modelling

The steel rib for the construction of a tunnel consists in assembled steel HEB section elements. Given the geometry of the structure, the steel rib is modelled using one-dimensional isoparametric finite elements. The mechanical characteristics assumed are consistent with the HEB200 steel section specifications, while the sustained ground is modelled with frame elements with a thickness equal to a nominal stripe of two meters.
The steel rib is made of Fe360 steel. In order to take into account the degradation of the material with the rise of the temperature, a thermo-plastic model is adopted. The dimensionless parameters of the mechanical characteristics are reported in Figure 3a. In addition, for the specific analysis, the ground around the steel rib is modelled having the mechanical parameters of sand (values for the soil density, Poisson's ratio and Young's modulus are taken equal to 0.2 and 230 MPa respectively). The boundary conditions are modelled importunately. A uniform weight is applied to the ground model, equal to the weight of the terrain above the steel rib. This weight is distributed to the steel rib due to the frame elements with which the ground is modelled (for simplicity, these elements are modelled as weightless), as shown in Figure 3b.

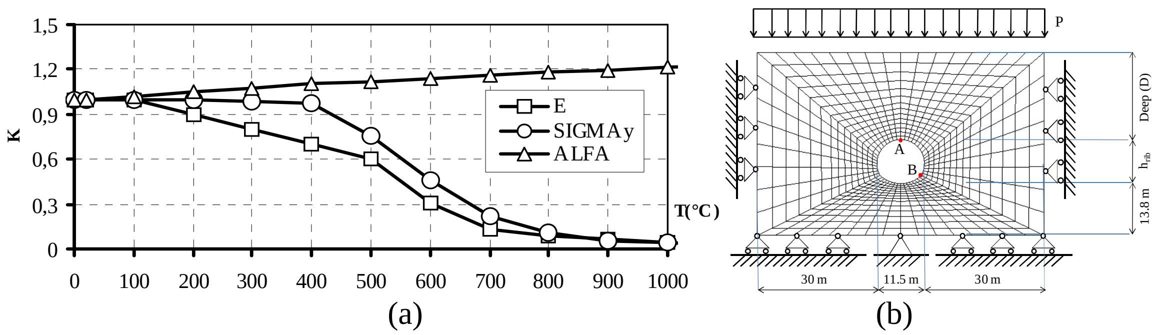

Fig. 3 Dimensionless parameters of the mechanical characteristics of the thermo-plastic steel material (a) and FEM modeling of the loads (b)

The parameters are synthesized in eq. (1) and eq. (2).

$$P = \gamma \cdot i \cdot D = 850 \frac{KN}{m} \tag{1}$$

$$\gamma = 17 \frac{KN}{m^3} \text{ (terrain specific weight)} \tag{2}$$

where, P is the linear weight, i=2 meters (distance between the steel ribs), h_{rib}=12.5 meters (steel rib height) and D=2h_{rib}=25 meters (distance between the surface and the steel rib). At each element of the steel rib is applied a temperature, rising with the rate given by the implemented time-temperature curves.

4.2 Structural response

Time-displacement and temperature-displacement diagrams on the vertical axis of a node in the top middle of the rib (node A) are obtained (Figure 4 and Figure 5).

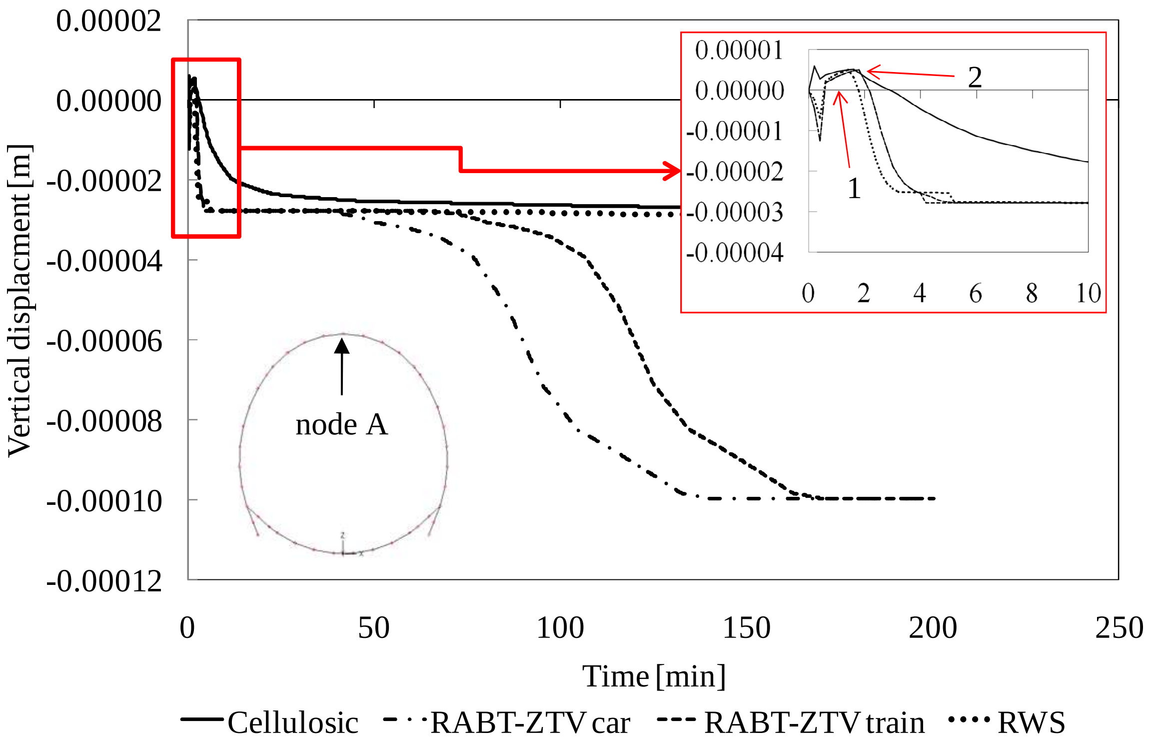

Fig. 5 Time-displacement diagram on the z-axis of node A

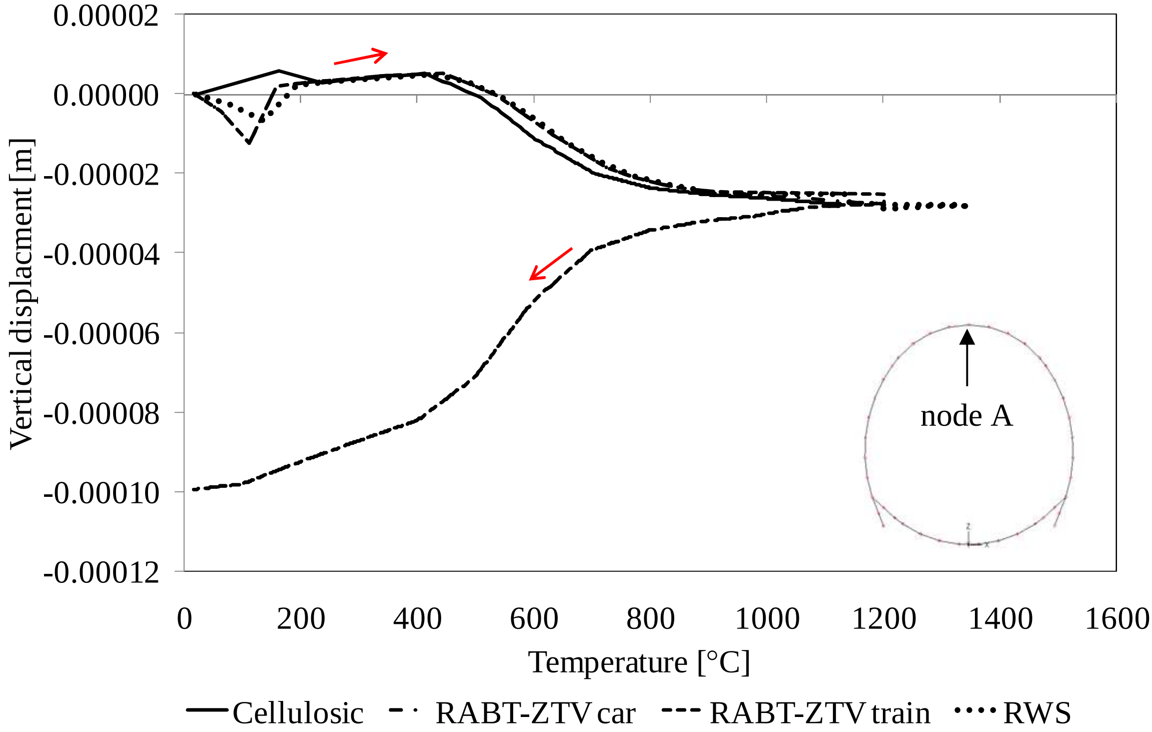

Fig. 6 Temperature-displacement diagram on the z-axis of node A

In the case of the cellulosic fire curve, at the initial temperature of 20°C a vertical displacement takes place depending only on the applied load. Consequently, in the transition from 200°C to 400°C (due to the temperature rise), the steel rib rises due to the thermal expansion of the steel. From point 1 to point 2 (Figure 5), the steel rib continuous to expand, yet in a less marked manner. This is due to the decrease of the elastic modulus of the material, starting from the temperature of 100°C. After a time of exposition that ranges approximately between 110 sec. to 150 sec. for the different fire curves, the displacement of node A of the steel rib becomes once again negative due to the degradation of the steel material at higher temperatures. Approximately before 40 min. the four fire curves produce the same behavior concerning the vertical displacement of the node A, although the response under the action of the cellulosic curve is less severe in terms of displacement gradients. When the cooling phase of the two RABT ZTV curves begins after 30 and 60 minutes, the negative displacements in these two cases become more marked. This is probably due to the fact that the cooling phase produces additional stresses, since the cooling expansion of the rib is contrasted by the terrain. This stress increment could produce the collapse of the steel rib. The (negative) displacement increase rate becomes lower as the temperature starts to drop and ends when it becomes cool again.
Similar considerations are made for the horizontal displacement of a node on the edge of the rib, while the maximum fiber stress (location of maximum absolute fiber stress for the rib) is also obtained but not reported for the sake of brevity.

5. CONCLUSION

In this paper, issues related to the fire safety design of tunnels infrastructures have been discussed. The general framework of tunnel fire safety has been introduced. In the second part, some specific structural analyses have been carried out on a steel rib for a tunnel construction.

Regarding the latter, the application of nonlinear analysis on the thermo-mechanic behaviour of materials consents to demonstrate and verify the performance of the structure in terms of resistance to fire during the design phase, and in this sense, some specific issues have been highlighted. In particular, the different behaviour of the steel material when subjected to different fire loads, something that can be observed in the computed displacements for a significant node, both in the time and the temperature domain.
The obtained results can be a starting point for further investigations (i.e. robustness assessment), and can be extended to real tunnel structures, accounting also for site specific information and fire loading aspects.

6. REFERENCES

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ΠΕΡΙΛΗΨΗ

Τα τελευταία χρόνια, με τη μεγάλη αύξηση της ιδιωτικής μετακίνησης και την κατασκευή οδικών και σιδηροδρομικών δικτύων υψηλής ταχύτητας, ο σχεδιασμός και η κατασκευή σηράγγων λαμβάνει περίοπτη θέση. Μετά από μια σειρά από δραματικά ατυχήματα στην Ευρώπη, η κοινή γνώμη έστρεψε την προσοχή της στην πυρασφάλεια των σηράγγων. Αυτό γιατί, μία πιθανή φωτιά σε σήραγγα είναι ένα ακραίο περιστατικό, το οποίο μπορεί να έχει καταστροφικές συνέπειες, με απώλεια ανθρώπινων ζωών αλλά και με μεγάλες οικονομικές ζημιές στη δημόσια περιουσία και στις οδικές μεταφορές. Στην παρούσα εργασία, το πρόβλημα της πυρασφάλεια των σηράγγων αντιμετωπίζεται με έναν ολιστικό τρόπο. Η αντιμετώπιση λαμβάνει υπόψη θέματα ασφαλούς σχεδιασμού βασιζόμενα στη μεθοδολογία της μηχανικής των συστημάτων, όσο και τους νεότερους κανονισμούς αξιολόγησης. Με βάση τα παραπάνω, παρουσιάζεται αριθμητική προσομοίωση της δυναμικής ανελαστικής συμπεριφοράς του χαλύβδινου πλαισίου ενίσχυσης της διατομής της σήραγγας, με κύριο στόχο την εκτίμηση της συμπεριφοράς του δομικού συστήματος κατά την αύξηση της θερμοκρασίας, ως επακόλουθο της εξάπλωσης της φωτιάς.