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Modelling of thermo-mechanical behaviour of tunnels under fire conditions

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ABSTRACT: Major fire events in tunnels have shown severe consequences in their structural performance, as reported in several incidents in the last decades. Particularly, due to the enclosure geometry of tunnels, intense thermal loading may develop in comparison to typical fires in buildings, which may result in more damage. Even though tunnel linings are an important component in transportation infrastructure, the understanding of their behaviour under high temperatures is limited. This paper aims to study the thermo-mechanical response of tunnels subjected to fire using nonlinear finite element analysis (NLFEA). For this purpose, recent experimental tests of large-scale reinforced concrete tunnels with and without fire protection are simulated. Different modelling approaches and strategies are discussed, and a detailed description of the constitutive model employed is presented. The results obtained in the simulations indicated that the numerical analysis can be used to investigate the thermo-mechanical behaviour of concrete structures.

1 INTRODUCTION

Traditionally, reinforced concrete (RC) structures have been regarded as good in terms of fire resistance due to the non-combustibility and low conductivity of concrete. For that reason, in many cases, RC elements do not require any additional fire protection. However, in the case of tunnels, due to the enclosure geometry, more intense fire scenarios can develop in comparison to typical fires in buildings, leading to more severe damage. In the past, several incidents related to fire events in tunnels have occurred (Lin and Chien, 2021). Even though tunnel linings are an important component in transportation infrastructure, the understanding of their behaviour under high temperatures is limited. The research on tunnels under fire has focused more on fire dynamics, smoke simulations, and evacuation strategies, and less research has been done in the areas of structural risk assessment, damage quantification, and robustness (Hua *et al.*, 2021a). As pointed out by Gernay *et al.* (2023), there are no general guidelines regarding the structural modelling of tunnels exposed to fire. In practice, a model representing the section with a 2D beam finite element is probably the most commonly used approach.

This work investigates the thermo-mechanical response of tunnels subjected to fire using nonlinear finite element analysis (NLFEA). This investigation aims to gain additional insight into the structural behaviour of tunnels in the case of a fire using advanced calculation methods. A detailed description of the constitutive model employed is presented in the

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following sections. Then, different modelling approaches and strategies are discussed and evaluated for the representation of fire scenarios, including (i) 2D model representation (plane strain and plane stress assumptions), (ii) influence of protective coating and (iii) representation of concrete spalling. The assessment of these parameters is conducted through the simulation of recent experimental tests based on the Hong Kong-Zhuhai-Macao Bridge (HZMB) project conducted by Huaqiao University. The experimental and numerical results are compared in terms of temperature distribution and crack patterns. Finally, key outcomes and conclusions are derived about the capabilities and limitations of the simulations and the feasibility of using finite element-based analyses in the design and post-fire evaluations of tunnels.

2 CONSTITUTIVE MODEL

2.1 *Concrete model*

The concrete nonlinear behaviour is represented by the total strain fixed crack model (Feenstra *et al.*, 1998) implemented in DIANA (release 10.5). Additional constitutive relationships are used to account for the material degradation due to high temperatures based on NEN-EN -1992-1-2:2005. The tension softening is described using the relationship proposed by Hordijk (1991), which assumed that the concrete tensile behaviour depends on the tensile strength (f_{ct}) and fracture energy (G_F). The concrete behaviour in compression is based on the fracture energy-based model proposed by Feenstra (1993). This model describes the compressive stresses by a parabolic stress diagram followed by a softening branch. Similarly to the tensile behaviour, the maximum compressive strain is scaled according to a fracture energy and a crushing-band width to reduce mesh size sensitivity. In addition, the model proposed by Vecchio and Selby (1991) is considered to represent the increment in strength and ductility due to confinement. Furthermore, the influence of large strains perpendicular to the compressive direction is included using the formulation proposed by Vecchio and Collins (1993), with a minimum reduction limit of 0.4 (Hendriks and Roosen, 2022). The concrete shear behaviour is represented using the so-called damaged-based shear retention factor proposed by DeJong *et al.* (2008).

For the thermal analysis, the thermal conductivity and the volumetric specific heat are used to model the concrete heat transfer. These properties are considered to be temperature-dependent using the relations recommended in NEN-EN-1992-1-2:2005. The thermal concrete expansion is defined according to the thermal expansion coefficient. The transient creep strain (also known as load-induced thermal strain, LITS) is modelled implicitly following the approach described in NEN-EN-1992- 1-2:2005.

2.2 *Reinforcing steel model*

The reinforcing steel constitutive model assumes an elastic-perfectly plastic behaviour. Similar to concrete properties, degradation as a function of the temperature is adopted. For this purpose, the modulus of elasticity and the yield strength are assumed to depend on temperature according to NEN-EN-1992-1-2:2005. It is noteworthy to mention that since the rebar is embedded and its area is relatively small compared to concrete, the steel rebar is not explicitly considered in the thermal analysis. It is assumed that the temperature is uniform within the steel area and equal to the temperature of the surrounding concrete.

Preliminary simulations showed an unrealistic full recovery of the steel mechanical properties during the cooling phase (i.e. the period when the temperature decreases to ambient temperature), even after reaching temperatures higher than 1000ºC. To address this issue, the original constitutive model in DIANA was adapted using a user-supplied subroutine written in Fortran. The subroutine restricted the yield strength and the Young modulus values based on the maximum temperature reached during the heating phase.

3 CASE STUDY: TUNNEL TESTS BY HUAQIAO UNIVERSITY

The large-scale fire tests on tunnel segments performed by Dong *et al.* (2023) and Duan *et al.* (2021) are used as reference in this study to compare and validate the modelling choices for the fire resistance assessment in tunnels. The experimental specimens represent a reduced scale (1:5) model of an immersed tunnel segment of the Hong Kong-Zhuhai-Macao Bridge (HZMB) project. The cross-section of the scale tunnel is shown in Figure 1. The total length of the tested tunnel segment was 5.8 m, and the overall width and height were 7.59 m and 2.28 m, respectively.

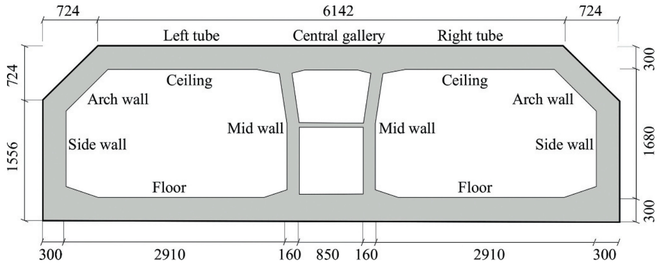

Figure 1. Cross-section of the scale tunnel, measures in mm. Adapted from Dong et al. (2023).

Before applying the thermal loading, vertical and horizontal loads were applied to the tunnel segments to represent the service load from the sediment and seawater. The total vertical load applied on the ceiling was 2427.4 kN, and the horizontal load on both sides of the walls of the tunnel was 437.2 kN. The test setup is shown in Figure 2a. During the fire test, the mechanical load was maintained constant during the heating and cooling phases. The fire scenario in the experiment conducted by Dong *et al.* (2023) considered a severe situation in which both tubes of the tunnel were exposed to fire at the same time. In the tunnel segment tested by Duan *et al.* (2021) the fire was applied in only one tube and a 30 mm sprayed coating was used.

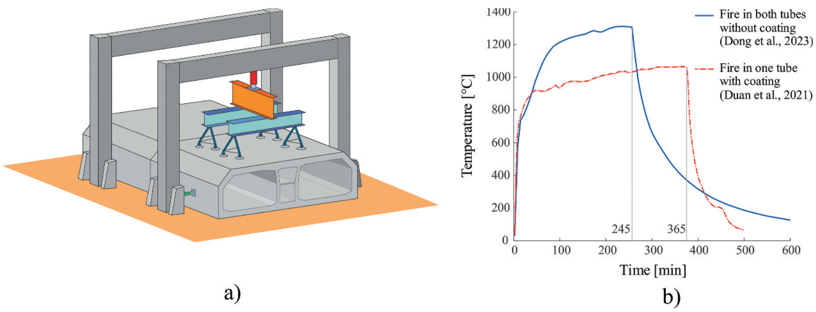

Figure 2. A) Setup during testing; b) fire scenarios for the tunnel segments tested by Dong et al. (2023) and Duan et al. (2021).

At the end of both tests, the authors reported that the capacity of the tunnel segments was sufficient to withstand the applied thermal load (during the heating and cooling phases). Despite that, the damage on the heated and unheated side of the tunnel was severe for the fire scenario on both tubes. Dong *et al.* (2023) reported that almost 100% of the exposed fire surface was affected by spalling. In the test conducted by Duan *et al.* (2021), the fire-resistive

Table 1. Fire scenarios considered in the tunnel segments.

Reference	Number of tubes exposed to fire	Fire protection	Maximum temperature	Spalling
Dong *et al.* (2023)	2	No	1344.8°C	Yes
Duan *et al.* (2021)	1	Yes	1113.7°C	No

coating performed well, and concrete spalling did not occur. However, concrete cracking was observed in the unheated side of the tunnel and even in the mid gallery and the left tube that were not exposed to fire at all. The fire scenarios are summarised in Table 1 and the recorded average furnace temperatures are shown in Figure 2b.

4 NUMERICAL MODELS FOR THERMO-MECHANICAL ANALYSIS

4.1 *Numerical models considerations*

In this work, a 2D approach was used to simulate the tunnel tests conducted by Huaqiao University. In the following sections, the fire scenarios are designated with the letters A and B, as given in Table 2. Two different modelling assumptions were compared for each tunnel test, namely, plane stress and plane strain idealisations. The former approach implies that the out-of-plane stresses are not accounted for in the simulation. In the case of the plane-strain assumption, the out-of-plane displacement (and the total strain in that direction) is considered null. It should be noted that both 2D approaches are approximations to represent the tested tunnel segments behaviour. A 2D plane-stress idealisation can be regarded as conservative since it ignores any possibility of force redistribution in the longitudinal direction of the tunnel (Gernay *et al.*, 2023). On the other hand, a 2D plane-strain idealisation can be regarded as a more reasonable alternative for the modelling of a real tunnel with several segments in the longitudinal direction. Yet, the plane strain assumption can also be regarded as conservative since it considers a uniform fire scenario along the tunnel full-length. As indicated in Table 2, additional simulations were also carried out to investigate the influence of spalling effects on the fire scenario in both tubes, leading to six simulations in total.

Table 2. Numerical models conducted for each fire scenario.

Fire scenario	Numerical model
Scenario A: fire in one tube with coating tested by Duan *et al.* (2021)	Plane strain (without spalling) Plane stress (without spalling)
Scenario B: fire in both tubes without coating tested by Dong *et al.* (2023)	Plane strain without spalling Plane stress without spalling Plane strain with spalling Plane stress with spalling

The tunnel cross-section is discretized using quadrilateral elements. For the staggered analysis, first-order isoparametric quadrilateral elements (denoted as Q4HT) are used during the thermal transfer calculations. In the subsequent mechanical analysis, these elements are automatically upgraded to second-order isoparametric elements denoted as CQ16M (for plane stress) and CQ16E (for plane strain). An average mesh size of 30 mm was adopted in the analyses (refer to Figure 3). This value was established with the aim of obtaining a sufficiently

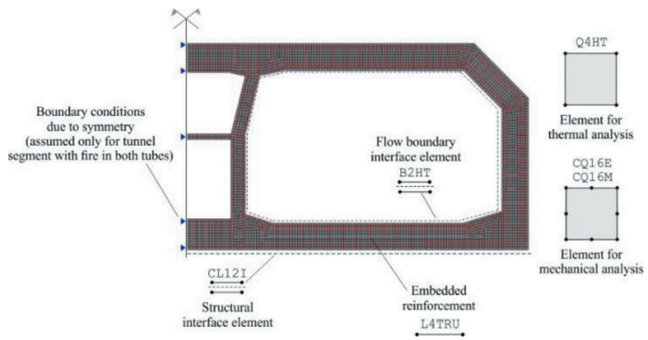

Figure 3. Typical mesh used for the numerical simulations for scenario B.

accurate nonlinear temperature distribution on the concrete cover. In the thermal analysis, the heat transfer is considered by radiation and convection mechanisms using boundary interface elements (B2HT). The furnace temperatures recorded during the test were used as an input in the thermal analyses. The interaction between the tunnel and the ground was modelled with interface elements (CL12I) with no support in tension and a high compression stiffness (75 MN/m^3), as assumed in Guo et al. (2016). No additional soil settlements or concrete primary creep strains were included in the simulation. The concrete nonlinear behaviour is represented using the constitutive model described in Section 2.

4.2 *Spalling simulation*

The loss of concrete cover due to spalling has two effects: (i) it reduces the stiffness of the member due to the reduction in the cross-section and mechanical properties, and (ii) in case of severe spalling, the reinforcement is directly exposed to fire and the heat flow inside the structure also increases. In this study, the spalling is simulated by considering a modification in the concrete thermal properties of the elements located at the exposed inner face. A 60 mm layer was defined as the spalling region at the ceiling and lateral walls. The region was created according to the average spalling depth obtained in the test. For the elements at that region, the thermal properties were adjusted during the analysis with a high value of thermal conductivity, 5 W/(mºC), and a relatively low value of thermal capacity, 10000 J/(m^{3}ºC;). The previous values were selected based on the criteria developed by Hua *et al.* (2021b), aiming to obtain a spalling rate close to 2.6 mm/min.

4.3 *Staggered analysis considerations*

A staggered procedure is performed in DIANA to couple the thermal and mechanical analyses. Therefore, the analysis is executed in two consecutive steps with the main assumption that the temperature distribution in the structure is independent of the structural behaviour. The calculated temperature fields are then incrementally applied in a separate mechanical analysis that takes into account the thermally induced strain as well as the material degradation due to high temperatures. The regular Newton-Raphson method is employed for the iterative procedure, and the convergence at each iteration is evaluated in terms of unbalanced force, incremental displacement and relative energy. The time step increments are updated with a cutback-based automatic incremental loading procedure. This algorithm reduces the number of steps required to apply the total time (i.e. the fire duration) and automatically decreases the time increments to recover from non-convergence steps in the iterative procedure.

5 RESULTS AND DISCUSSION

5.1 *Thermal analysis results*

The computed temperature field is independent of the 2D assumption. Thus, the same temperature distributions are obtained for plane stress and plane strain idealisations. Figure 4 shows the

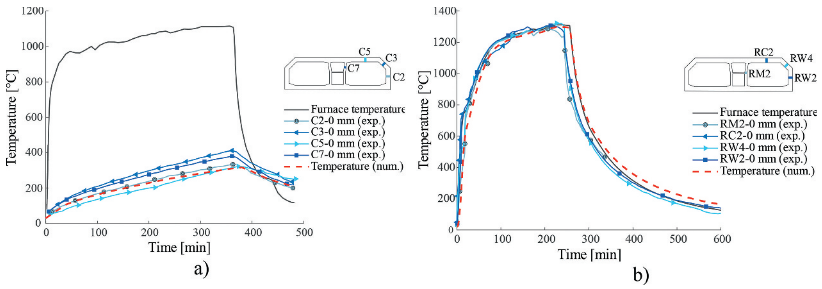

Figure 4. Temperatures at exposed surface, simulation (num.) results versus experimental (exp.) results: a) scenario A; b) scenario B.

temperature at different locations at the exposed surface for both experiments. For the tunnel with a protective coating (scenario A), the numerical simulation was able to represent the average temperature at the exposed side of the tunnel (Figure 4a). It can be noted that the coating significantly reduced the temperature in the concrete in comparison with the applied fire scenario. In the case of the tunnel without protective coating (scenario B), the temperature at the exposed concrete surface, followed almost the same furnace temperature (Figure 4b).

Figure 5 shows the temperatures measured by the thermocouples RC2 (ceiling) at 0 mm, 30 mm, and 60 mm in the fire scenario B. It is important to note that the temperature inside the concrete cross-section followed a temperature similar to that recorded at 0 mm. As stated by Dong *et al.* (2023), this result can be explained by the large spalling observed during the test. Furthermore, it is also noted that the temperature at 30 mm and 60 mm is underestimated if the spalling effect is not considered in the simulations.

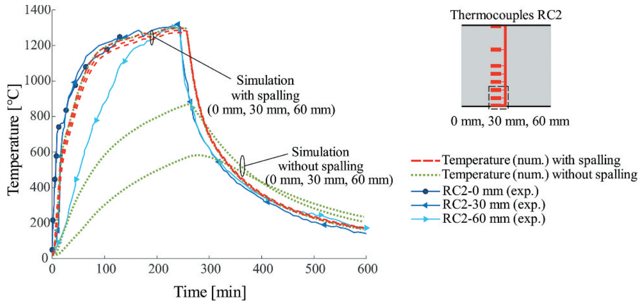

Figure 5. Temperature evolution inside the concrete section for scenario B.

5.2 *Mechanical analysis results*

According to the experiments, the tunnel capacities were sufficient to resist for both fire scenarios. This observation also holds for the numerical simulations with fire coating, in which the iterative procedure converged for all the time steps (Figure 6a). In contrast, the numerical simulations for the most severe fire scenario finished earlier due to convergence problems. The consideration of spalling contributed to reducing the fire resistance time, especially for the plane strain idealisation with spalling, in which the analysis only lasted 72 minutes. The other simulations ended during the cooling phase, as depicted in Figure 6b. It should be noted that the constitutive model assumes that damage recovery due to cracking is not possible. However, a full recovery of the thermal strain was observed during the cooling phase. Consequently, some reloading behaviour was noticed in the concrete stress evolution. It can be inferred that this inaccurate representation, together with the large number of active cracks during the cooling phase may have led to convergence problems in the numerical analyses.

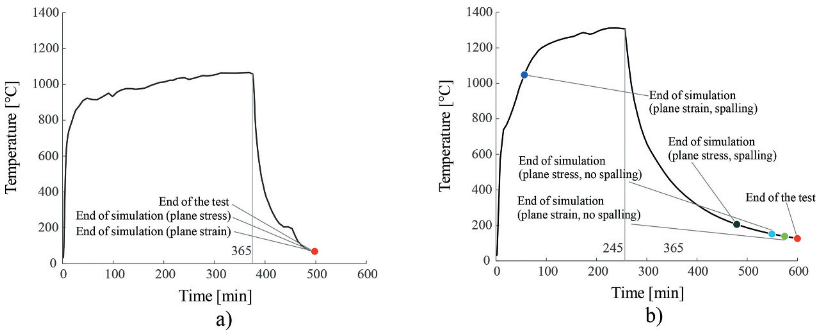

Figure 6. Applied fire scenarios and the last step converged in each numerical analyses: a) scenario A; b) scenario B.

Figure 7 shows an example of the crack pattern obtained from the simulations at the end of the heating phase for both fire scenarios. For the sake of brevity, the results are only shown for two plane stress models. According to Duan *et al.* (2021), for the fire scenario with a protective coating, the cracks were smeared over the unheated side of the tunnel and the mid-wall. This observation is in agreement with the numerical results. For the fire simulation on both tubes, the crack strain increased considerably compared to the previous fire scenario, and the maximum strain values were localised at the tunnel corners and mid-walls.

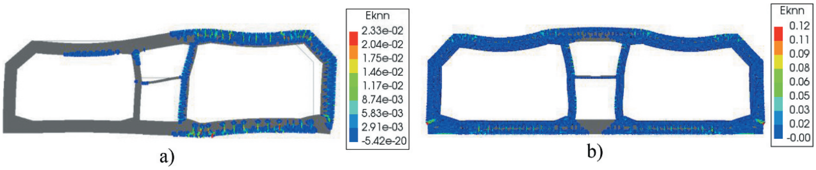

Figure 7. Comparison of crack patterns at the end of the heating phase for plane stress representation: a) scenario A; b) scenario B with explicit spalling representation. The numerical crack width can be estimated as $w = \varepsilon \cdot h$, with h = 30 mm.

The analysis of the principal compressive stresses in the simulations showed the development of a compressive ring in the tunnel cross-section. This ring was located in the concrete cover region, and it was induced by the concrete thermal expansion. For both 2D representations, it was observed that when the spalling representation was included, the compressive stresses at the ring region were visibly reduced. As shown in Figure 8, the comparison between the models with and without spalling for the plane strain idealisation provided a good understanding of the early failure when the spalling effects were included.

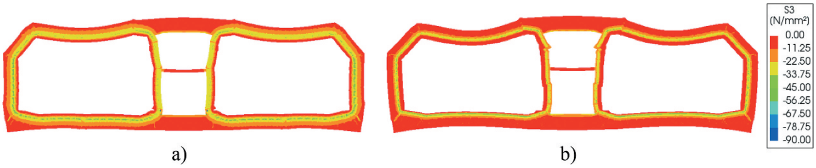

Figure 8. Principal compressive stresses for scenario B: a) plane strain and no spalling at 365 minutes; b) plane strain with explicit spalling at 72 minutes.

Overall, the moment diagrams, values and redistribution were similar in both 2D representations. It was noted that even with the coating application, the maximum moment values were similar to the ones calculated for both fire case scenarios without protection. It is pointed out that the maximum moments at the loading stage represented approximately only 10% of the maximum moment obtained during the fire event as shown in Figure 9.

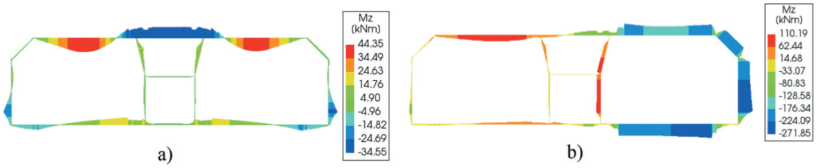

Figure 9. Bending moment diagrams for 2D plane strain representation: a) from mechanical loads b) at 365 minutes in fire scenario A.

6 CONCLUSIONS

This paper investigated the thermo-mechanical behaviour of reinforced concrete tunnel segments under fire using NLFEA. The results from different modelling strategies were compared against experimental data reported in the literature. The main modelling assumptions included the 2D idealisation (plane stress/strain representation) and the explicit spalling simulation based on the simplified model proposed by Hua *et al.* (2021b). The numerical results from the thermal analysis indicated that the temperatures can be significantly underestimated if the spalling effects are not considered in the simulation of a tunnel without fire protection. Moreover, plane stress and plane strain idealisations provided similar numerical responses for both fire scenarios when compared to the experimental results and also in terms of internal forces. The main difference was found when the spalling effect was included. In that case, more damage was observed in the plane strain representation, and consequently, the simulation finished earlier than expected, during the heating phase. These conclusions are limited to the case study examined in this paper. Future works will focus on the developing more reliable constitutive models for the representation of concrete during the cooling process. These developments will be important for assessing the structures remaining capacity after a fire event and in the design optimisation of fire protection systems.

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