Journal of Civil Structural Health Monitoring
https://doi.org/10.1007/s13349-023-00670-z

ORIGINAL PAPER

Fire risk assessment of bridges: from state of the art to structural vulnerability mitigation

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Received: 30 June 2022 / Accepted: 3 January 2023

Abstract
In recent years, due to the rapid urbanization, the fire risk in transport infrastructures is becoming more critical. These fires, typically caused by highly flammable materials, can significantly compromise the stability of the structure, as well as cause significant economic and social losses. However, in current regulations, no fire design or verification criteria are provided for bridges and the buildings prescriptions are not directly applicable due to the significant differences among the fire conditions. Therefore, starting from a deep literature review, different performance levels for bridges' structural fire resistance were proposed. These levels were linked to the fire risk classification suggested by Kodur et al., for identifying the most vulnerable bridges to fire. This methodology was applied both to the prescriptive and performance-based approaches, using nominal and natural fire curves derived by advanced zone models of several bridge fire scenarios. To better investigate the structural fire performance of bridges, parametric analyses of a typological bridge were conducted for identifying the most critical structural systems and fire scenarios. One of the most relevant finding is that the use of performance-based approach allows to consider more realistic fire conditions, to satisfy higher performance levels with an optimization of the fire protection design. Therefore, the proposed approach can be useful both for designers and industrial category to assess the bridge performances in fire, not only according to prescriptive approach but also considering the performance-based one.

Keywords Structural fire resistance · Fire risk · Bridges · Fire vulnerability · Performance-based approach

1 Introduction

According to the current international literature, the number of fires involving transportation facilities is rapidly growing in recent years due to the huge urbanization and the increased transportation of fuel and chemical materials [1]. The consequences of these fires can be very significant, endangering the lives of users and causing slowdowns of traffic flow, economic losses, and partial or total collapse of facilities. Refurbishing or replacing these structures after fires would cause a high financial investment and this implies that, in the short term, the only available choice is to extend their service life. To do this, it is necessary to recognize and assess the fire risk in bridges, reducing their vulnerability to fire through appropriate strategies.

Most of these fires occurred due to the collision of vehicles, e.g., tankers, freight trucks, and cars, with other vehicles or with structural components, generating fuel spills. In addition, these facilities are easily accessible and open to public, with minimal or any security, and therefore, they are susceptible to fires caused by vandalism [2].

Some of these fires caused significant economic and human losses; nevertheless, a lack of appropriate fire safety requirements in codes and standards is evident and the transportation facilities are designed without specific fire mitigation strategies. Thus, in case of fire, these facilities can be particularly vulnerable to fire-induced damage even to collapse, affecting the performance of the transport network and causing prolonged interruptions of the traffic flow [3].

Fires involving transportation facilities are, typically, very intense and explosive nature. This is due to the collisions

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Published online: 19 January 2023

that can occur at high speeds causing rapid ignition of highly flammable gasoline-based fuels, which have low flash points, in an open environment. This fuel burning produces very high temperatures (about 800 ÷ 900 °C) within the first minutes of fire ignition and the temperature peak can exceed 1200 °C.

In many cases, fires in transportation facilities are quickly extinguished by the fire rescue team. However, in some scenarios, very intense fires can induce significant degradation of the load-bearing capacity of structural elements, due to the loss of strength and stiffness of the structure, resulting in possible collapse, as in the case of some recent fires on important bridges in the United States and in European tunnels [4]. After a fire, even in the case of minor events, a proper investigation, inspection, and eventually maintenance of the structure before reopening it to the public are required. The closure of a bridge or tunnel for maintenance requires traffic deviations on alternative routes, causing significant traffic delays in the affected region.

However, as mentioned before, there is a lack of the specific guidelines for the designing of fire risk mitigation of these infrastructures. In some critical fire scenarios, where fire protection of infrastructures is necessary, designers tend to extend the fire protection requirements used for buildings to transportation structures, despite the huge differences between the types of structures. Therefore, these requirements may not be directly applicable to transportation facilities because of significant differences of the fire scenarios (fire load properties, geometry, structural parameters, etc.), producing inappropriate fire safety measures for infrastructures. For example, combustible materials found in buildings are, typically, cellulose-based and, therefore, produce less intense fires than those occurring in bridges or tunnels, which are mainly hydrocarbon-based. The fires from cellulosic materials, represented through the standard ISO 834 fire curve [5], reach a temperature of about 1000 °C in 2 h. While, the hydrocarbon fires, typically associated with bridges, can reach a temperature of 1050 °C in the first 5 ÷ 10 min. Another key difference is the ventilation conditions between buildings and bridges. Indeed, buildings are often designed with compartment features, having a limited availability of oxygen and fuels. Bridges, on the other hand, are in wide and open spaces, providing an unlimited amount of oxygen. When combined with a large amount of combustible materials existing in vehicles, the result is the optimal condition for rapid combustion and fire spread. In addition, for economic considerations, slender structural members are typically chosen in bridges, while class 1 elements are generally chosen for buildings. These slender elements, even if they can provide the correct strength and stiffness, are more vulnerable to fire [6].

In general, the fire protection required for structural members can be achieved on the basis of conventional prescriptive or performance-based approaches. However, most prescriptive approaches are based on fire tests conducted in accordance with the standard fire curve, which is applicable to structural elements of buildings, since fires in buildings are mostly cellulosic in origin [5]. Thus, the use of instructions based on prescriptive approaches and derived from the ISO 834 fire may not be appropriate for bridges structures. For example, 1 h of fire resistance evaluated using ISO 834 curve may be equivalent to less than 1 h of exposure to a hydrocarbon fire. On the other hand, the implementation of performance-based design methods can provide designers with efficient and cost-effective solutions. Indeed, these methods are based on rational and engineering principles to achieve specific solutions for high-risk fire bridges.

The purpose of this paper is to provide the base of a strategy for the design and verification of bridges under fire conditions. In particular, the focus is on the identification of fire performance levels for bridges, giving information also about the selection and modelling of bridges' fire scenarios, according to the performance-based approach principles. The proposed approach can be useful both for designers and industrial category to assess the bridge performances in fire, not only according to prescriptive approach but also considering the performance-based one.

2 Fire risk and assessment of bridge infrastructures

Fire exposure effects are typically neglected in structures and infrastructures design, even though they could determinate their failure. Indeed, high temperatures can reduce mechanical material properties and they can also produce redundant stresses in structural elements; therefore, to evaluate the fire risk of bridges is a crucial aspect.

As a general discussion, the risk is a combination between several factors that are the probability of the event occurrence, the vulnerability, and the exposed value. In terms of probability of occurrence, the technical literature provides statistical analyses of national polls about the occurrence of fire events: a comparison between the fire probability of occurrence in buildings and bridges shows that in the first case, the probability is 29.5% against the 2.3% of bridges [7]. Thus, considering these probabilities, it seems that the fire risk on bridges is not particularly relevant. However, comparing the failure probability of buildings and bridges in case of fire the same conclusion cannot be confirmed. Indeed, these probabilities of failure become more similar to each other, so the bridges' intrinsic fire vulnerability leads to a common structural collapse [8, 9].

Even if the probability of bridge fires is not particularly high, their consequences can be significant, so to design and verify bridge structures in case of fire are necessary. For

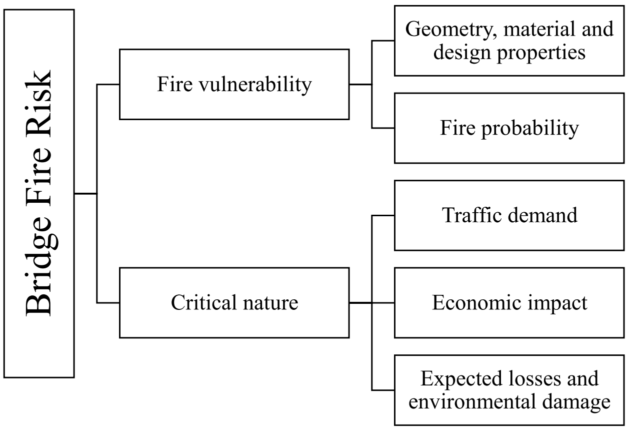


Fig. 1 Key features influencing fire risk in bridges, adapted from [2]

Table 1 Risk grades and associated importance factors, adapted from [2]

Risk grade	Overall class coefficient (λ)	Importance factor (IF)
Critical	≥ 0.95	1.5
High	0.51–0.94	1.2
Medium	0.20–0.50	1.0
Low	< 0.20	0.8

this reason, a general method that allows to identify which bridges should be designed or verified in fire situation can be useful. In this regard, Kodur and Naser proposed the importance factor (IF) of bridges [2] for the classification of their fire vulnerability. Bridges can be classified in four classes according to the importance factor value, corresponding to different risk levels from low to critical. The evaluation of this factor is based on the bridges vulnerability and their critical nature (Fig. 1).

In particular, the bridges' vulnerability is described by considering its structural features, such as the structural system, the materials, the length spans, the lanes number, etc. The critical nature measures the value exposed to the risk and, in general, this value includes all the economic losses consequent to the bridge failure (such as the costs to repair or rebuild the infrastructure), the social damage caused by the stopped viability, the ADT (vehicles/day), the economic impact, the historical importance, etc. The combination of all these factors leads to the importance factor evaluation that measures the fire risk grade of each bridge. The importance factor can be classified according to fire risk, that can vary from low to critical, as shown in Table 1.

The method proposed by Kodur also provides the verification criteria, as shown in Table 2. For low fire risk, no verification of the bridges has to be performed. While, the method proposes a fire verification in the time domain by monitoring the maximum displacement, which has to be lower than $L/30$ (where L is the length of the bridge span) for 1 h in case of high-risk level or 2 h in case of critical one. This verification must be led using the hydrocarbon fire curve, to take into account the most probable fire nature in bridges.

3 Fire design and safety check of bridges

In the context of the modern technical codes, as the new Italian Technical Code on Fire Prevention [10], the fire resistance is defined as a passive fire protection measure to guarantee load-bearing and compartmentation capabilities of structures according to performance levels, selected by the designer in order to achieve the defined fire safety objectives. The Italian code, in accordance with European ones, defines five performance levels (PL), described in Table 3, depending on the importance of the building.

Table 2 Descriptions and recommendations for the fire risk categories, adapted from [2]

Fire risk category	IF	Impact of fire on bridge	Recommended fire proofing to structural members
Low	0.8	Negligible impact on integrity of bridge or operation of facility, with no human losses	No need of fire proofing
Medium	1.0	Minor impact on structural member of bridge and operation with no human losses. No investments are necessary to restore bridge following fire incident	No need of fire proofing
High	1.2	Significant impact on structural members of bridge with partial/complete collapse of main structural elements, partial shutdown of operation with possible human injuries/losses	At least 1 h fire proofing should be provided to main structural elements
Critical	1.5	Immediate/severe impact on bridge (loss of carrying load capacity and total collapse) and complete loss of operation. Expected human casualties and permanent closure of highway/bridge	One-to-two hour (s) fire proofing should be provided to main structural elements

Table 3 Performance levels for buildings

Performance level (PL)	Description
I	No external consequences for structural collapse
II	Maintaining the fire resistance requirements for a period sufficient for the evacuation of occupants
III	Maintaining the fire resistance requirements for the whole fire duration
IV	Limited damage of the structure after fire duration
V	Complete serviceability of the structure after fire exposure

To satisfy the fixed performance level, different design solutions according to prescriptive or performance-based approaches can be chosen.

The main difference between the prescriptive (PA) and the performance-based (PBA) approaches is that the first one is based on standard fire resistance tests or empirical calculation methods, using nominal fire curves. In particular, the code provides three types of conventional fire curves (standard ISO834, hydrocarbon, and external nominal curve), selected according to the nature of the combustible materials in the compartment. On the other hand, the PBA considers the complexity of structures and the inter-relationship between the various fire safety measures and systems, using specific natural fire curves, generally obtained by advanced fire models. The first step of the PBA design consists of the thermal input assessment through the selection of design fire scenarios, which represent qualitative description of the fire development, based on key aspects that characterize the real fire (e.g., compartment dimension, ventilation, fire loads, etc.).

About the verification criteria, the PA approach provides a verification in terms of minimum fire resistance in the time domain, classifying the structures in a discrete number of classes (R30, R60, etc.). All these aspects about the fire resistance of buildings cannot be directly applied to infrastructures like bridges, as many differences have to be underlined. As also described before, in the case of buildings, the fire occurs in a compartment and the natural fire curve is influenced by the oxygen available as a function of the openings. In case of bridges, it is not possible to define a confined compartment, so the standard fire curves do not represent the real fires adequately. A better way to define the fire curve in the case of bridges is the computational fluid dynamic (CFD) analysis that allows to model the fire propagation near the bridge structure. These analyses also allow to model different fire scenarios to take into account the most severe fire event location for the structural bridge verification. Even if the performance-based approach seems to be the best way to design and verify bridges in case of fire, no defined criteria are provided in technical references.

Starting from the performance levels for the buildings, the ones related to infrastructures can be defined, taking into account the importance factor proposed by Kodur as a measure of the fire risk of any bridges. In this work, four fire performance levels are defined (Table 4). The first two can be related to low and medium fire risk grades and correspond to the satisfaction of resistance criteria. The other two can be related to high and critical risk grades and, therefore, require an improved performance that can be achieved by limiting displacements. In this way, the importance factor also sets the performance level that must be achieved in bridges.

If the prescriptive-based approach is used, this PLs have to be linked to a certain fire resistant class. In particular, PLI considers a required fire resistance time (t_{I}) equal to the minimum between 15 min and $2t_{\mathrm{evac,}}$ where t_{evac} is the time to evacuate the bridge. For PLII, the required time can be fixed equal to 60 min (t_{II}), also considering the Italian regulation suggestions [10], in which this time is obtained as a function of the specific design fire load $q_{\mathrm{f,d}}$. For the bridges, the specific design fire load was considered equal to 900 $\mathrm{MJ/m^2}$ [3]. For the satisfaction of PLIII, the structure has to preserve its bearing capacity for the time required by level II (i.e., 60 min) and the damage recorded at the same time Δt_{II} has to be limited to L/100. While, for the PLIV, no damage must be recorded, meaning that after 60 min, a maximum deflection of $L/250$ is accepted. Those displacement limits are already reported in Italian regulation for the

Table 4 Proposed performance levels for bridges

Performance level (PL)	Description	IF	Fire risk grade
I	The bridge must hold for the time required for evacuation	0.8	Low
II	The bridge must withstand the duration of the fire	1.0	Medium
III	Displacements should be limited to L/100 for the duration of the fire	1.2	High
IV	Displacements should be limited to L/250 for the duration of the fire	1.5	Critical

buildings and extended in the proposed methodology also to the case of bridges.

4 Advanced fire safety check: parametric analysis

To investigate the response of a typical steel–concrete fully composite bridge exposed to fire, parametric thermo-mechanical analyses were performed using the FEM software SAFIR [11]. These analyses were carried out following both the prescriptive and the performance-based approach, to highlight the main differences between the two approaches and to identify how to optimize the fire design of bridges. For the first one, the hydrocarbon fire curve was chosen and the analyses were carried out on four different structural systems, variable for constraint conditions and exposure to fire. According to the performance-based approach, natural fire curves have been obtained using the software CFAST [12], considering five fire scenarios. All the details are described below.

4.1 Prescriptive-based approach

Thermo-mechanical analyses were performed using the FEM software SAFIR [11], simulating a fire close to a typical steel–concrete composite bridge. Their results allowed investigating several aspects of fire vulnerability of road bridges.

In thermal analyses, different emissivity values were considered to take into account the shadow effect offered by the lower flange to the rest of the profile. According to Kodur and Aziz suggestions [13], an emissivity value of 0.7 was chosen for the lateral and lower parts of the bottom flange, and a value of 0.5 was used for the remaining part of the bottom flange and for the web, while 0.3 was chosen for the upper flange.

Furthermore, according to the Eurocode 1—Part 1–2 [14], convection coefficients $\alpha_c = 50\frac{\mathrm{W}}{\mathrm{m}^2}{}^{\circ}\mathrm{C}$ and $\alpha_c = 35\frac{\mathrm{W}}{\mathrm{m}^2}{}^{\circ}\mathrm{C}$ were used for the thermal analyses carried out with the hydrocarbon curve and with the natural fire curve, respectively. The thermal properties of steel and concrete (conductivity, specific heat, and thermal expansion) vary with temperature according to Eurocodes [15, 16]. The temperatures reached in the elements of the composite beam (slab, web, and flanges) were obtained as average of temperatures recorded in several nodes of each element.

To study the response of a bridge under fire, only dead loads were considered applied to the structure, neglecting live loads, according to the Eurocode 1. Furthermore, Paya-Zaforteza and Garlock [17] carried out mechanical analyses considering four different load combinations and they observed that the amount of live load does not have a strong influence on both time and type of failure. Thus, live loads can be neglected.

To validate the thermo-mechanical model performed with SAFIR, the experimental results of a composite beam exposed to fire were simulated. The experimental test was carried out by the British Steel Technical and Sweden Laboratories [18]. The steel profile, simply supported with a 4.5 m span, is not insulated and was exposed to ISO 834 fire curve. The tested steel beam has a height of 357 mm and a width of 171 mm, while the concrete slab has a thickness of 126 mm. The 3D thermal analysis model is shown in Fig. 2a, while a comparison between the temperatures predicted by the FEM model and the ones measured in the fire test is shown in Fig. 2b. The upper flange of the beam has lower temperatures compared with the bottom one, due to the effect of the concrete slab which dissipates heating in the top flange.

The predicted theoretical temperatures are in a very good agreement with the experimental data and the slight difference can be due to the variation in the heat-transfer parameters, such as emissivity and convection coefficients, used in the analysis compared with the real values inside the furnace.

After the SAFIR thermal analysis validation, a typological fully composite bridge was analysed. In particular, several parametric analyses were performed, varying the constraint conditions, fire scenarios, and fire protection. The cross-section of the analysed bridge is shown in Fig. 3 and the structural materials are C25/30 concrete and S355 steel. To understand the fire effect on this type of bridges, both the prescriptive and performance-based approaches were used; all the details are described below.

Considering this typical steel–concrete bridge located in an urban area, according to the Kodur classification [2], it has an importance factor of 1.2, so its structural members have to guarantee a fire resistance of 60 min under the hydrocarbon fire curve. For this reason, it is necessary to carry out thermo-mechanical analyses for evaluating the behaviour of the bridge in fire conditions and to determine whether the bridge can guarantee 1 h of fire resistance.

The first step was to perform thermal analyses of the composite steel–concrete section; the bridge cross-section was subjected to different boundary condition in its surfaces (see Fig. 3), in particular:

- the steel profile and the bottom faces of the concrete slabs were exposed to the gas temperature time history (red surfaces);
- the top faces of the concrete slab were exposed to the function F20 in SAFIR, which maintains the gas temperature at 20 °C during the thermal transient, allowing the heat exchange with the outside ambient;

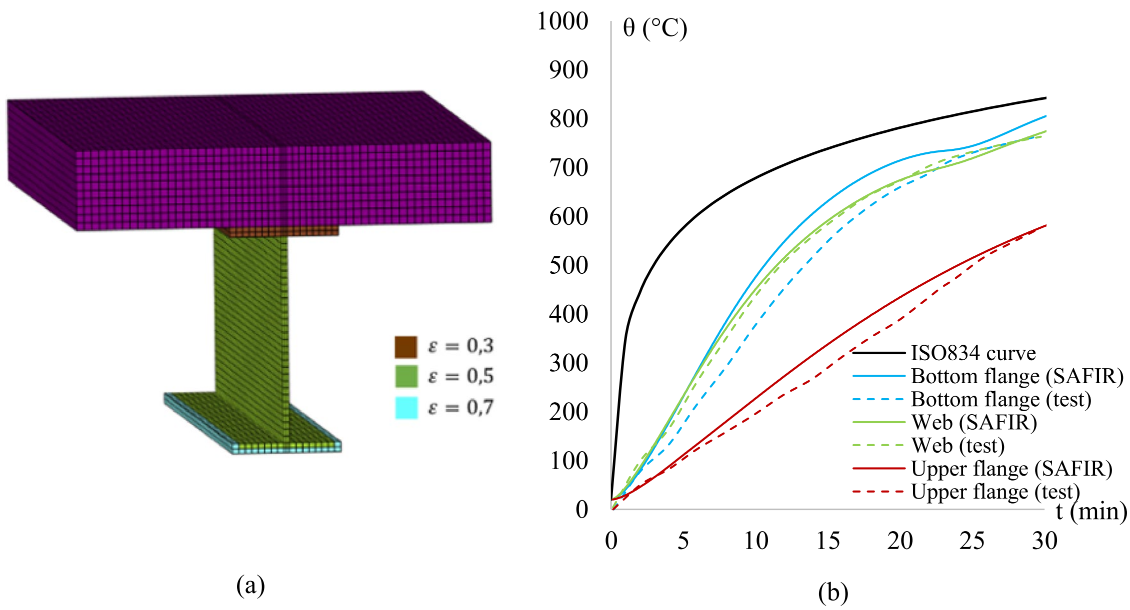
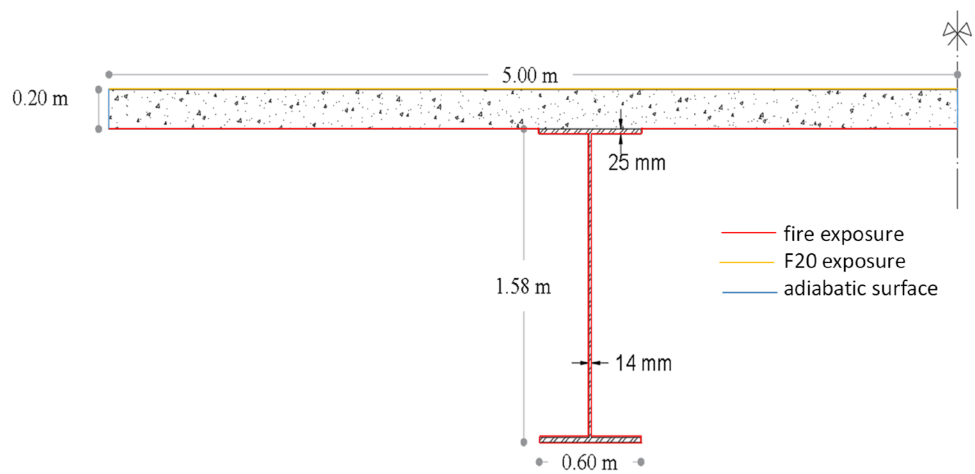


Fig. 2 **a** 3D model used for thermal analysis; **b** comparison between the temperatures from SAFIR e from the experimental test

Fig. 3 Cross section of a typological steel–concrete fully composite bridge



- lateral surfaces of the concrete slab were considered adiabatic for symmetry reasons.

The resulting temperatures in the steel profile are shown in Fig. 4.

After the thermal analyses, the mechanical ones were carried out considering different structural systems, to evaluate the failure time of the bridge as the constraint and exposure conditions vary. In particular, four systems were considered: (1) simply supported beam constrained with a hinge and a spin, (2) simply supported beam constrained with two hinges, (3a) continuous beam with two spans exposed only on the left span, and system (3b) where both the spans are exposed to fire (Fig. 5).

A 2D mechanical model was built in SAFIR software using beam elements. Each beam is related to a cross-section whose properties, in terms of geometry and temperature reached during the fire event, are the output of the thermal analysis. Thus, the concrete slab is part of the cross-section properties of the beam frame used in mechanical analysis.

Each span is 27.5 m long and the applied load is equal to 62 kN/m, corresponding to the structural loads (concrete slab and steel profile) and the not structural ones (road surface) of half section, for symmetry. These conditions in system 1 lead to utilization factors of 0.35 (flexural) and 0.19 (shear). The failure time ($t_{R,SAFIR}$) obtained with SAFIR mechanical analysis for system 1 is 414 s, because the bridge structural section reaches the resistant moment M_{rd} in the

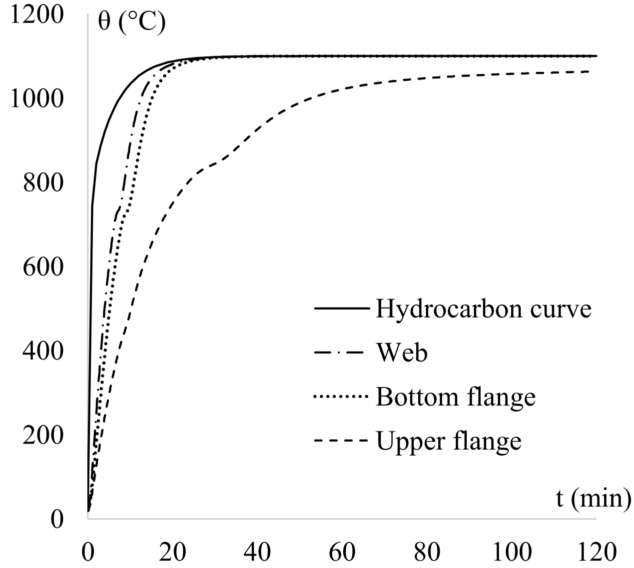


Fig. 4 Temperatures in the steel profile under hydrocarbon fire

middle of the span (Fig. 6a). The decrease of the beam stiffness, due to high temperatures, leads to a consequent increase in displacements (Fig. 7).

In system 2, due to the structural redundancy and the constrained thermal expansions, the axial force increases during the first part of fire exposure, leading to an increase of bending moment M_{ed} (II-order effects) and displacements, whereas in the second part of fire exposure, a tension axial force develops allowing the so-called "chain effect": the chain effect in this case is beneficial, as it avoids the flexural failure of the beam, which after almost 16 min reaches the maximum resistance tensile force inside the steel profile (Fig. 6a).

The maximum negative M_{ed}^{-} and positive M_{ed}^{+} bending moments recorded in systems 3a and 3b are shown in Fig. 6b: they vary during fire exposure due to flexural redundancy and the constrained thermal deformations. The failure behaviours of the two structural systems are similar to each other: in both cases, after about 5 min, a plastic hinge is generated on the central support, where the negative resistant moment is reached. Once the ductility is exhausted, the formation of an additional plastic hinge is not expected and positive moment is always lower than the resistant one.

The trends of displacements (Δ_{max}) over time in the four structural systems are shown in Fig. 7, where it can be seen that redundant systems guarantee much lower deformation levels.

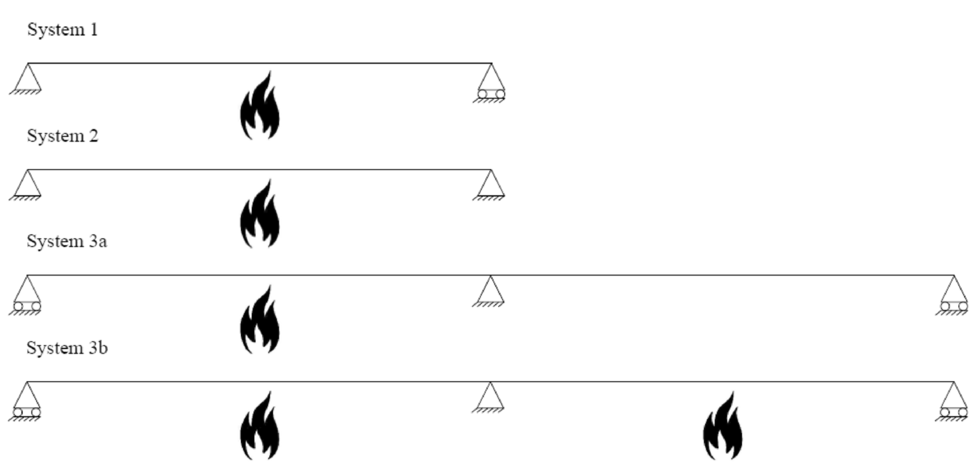
The failure times $t_{R,SAFIR}$ and the time at which the maximum displacement L/30 is reached $t_{L/30}$ in the four systems are represented in Table 5.

According to structural checks proposed by Kodur [2] for a high fire risk, the maximum displacement must be less than $L/30$ for at least 1 h. In this case, the limit displacement $L/30 = 0.92$ m is reached at 5.8 min in system 1 and at 5.4 min in system 2, respectively. In the two continuous beam system, SAFIR does not record the $L/30$ displacement.

The $L/30$ limitation can be seen as a different way to interpret the collapse. This value of deflection is not particularly restrictive and it does not allow to preserve the functionality of the structure: indeed, after this displacement value, the bridge is out of service and it has to be repaired. Therefore, considering the proposed performance level for bridges (see Sect. 3), this criterion corresponds to a performance level II; if levels III or IV are required, it is necessary to add a more severe limitation on deflections and on operation after fire. Then, in general, the failure time can be seen as the minimum value between the time at which $L/30$ is reached and the failure time recorded by SAFIR.

In Table 6, the outcome of safety checks in fire conditions is shown for each structural system, depending on the IF value. If the risk grade is low or medium, there is no need of fire proofing (NFP). If, on the other hand, the risk grade is high or critical, a fire resistance of at least 1 h or between 1 and 2 h is required, respectively.

Fig. 5 Structural systems considered for mechanical analyses



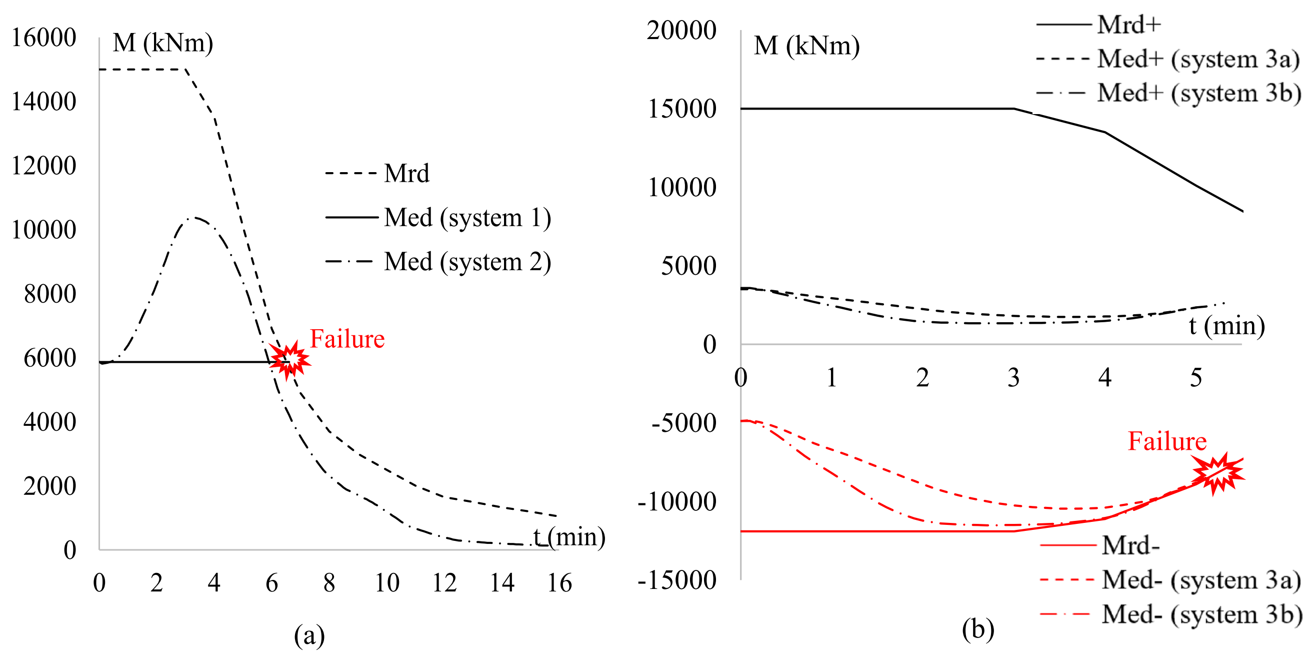


Fig. 6 **a** Bending moments under hydrocarbon fire in systems 1 and 2 and **b** in systems 3a and 3b

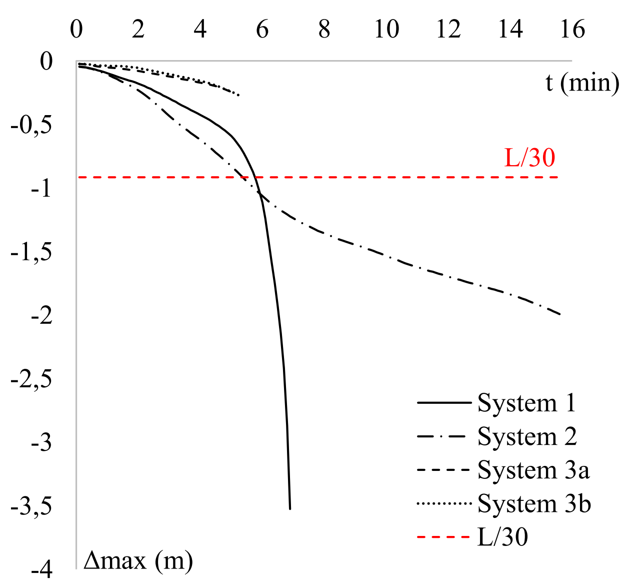


Fig. 7 Displacements in the four systems under hydrocarbon fire

Table 5 Collapse times and times at which L/30 is reached

#System	$t_{R, SAFIR}$(min)	$t_{L/30}$(min)
System 1	6.9	5.8
System 2	15.6	5.4
System 3a	5.0	–
System 3b	5.1	–

Table 6 Results of safety checks in fire conditions

#System	Low (IF = 0.8)	Medium (IF = 1.0)	High (IF = 1.2)	Critical (IF = 1.5)
System 1	NFP	NFP	NV	NV
System 2	NFP	NFP	NV	NV
System 3a	NFP	NFP	NV	NV
System 3b	NFP	NFP	NV	NV

In conclusion, performing prescriptive-based analyses, all the considered systems were not verified (NV) in fire conditions if the risk grade is high or critical. Following these criteria, regardless of the constraint and exposure conditions, the application of fire mitigation strategies is required to reduce the IF.

4.2 Performance-based approach

One of the novelties of this paper is the application of the Fire Safety Engineering (FSE) criteria to the bridges, demonstrating the satisfaction of the different fire performance levels of bridges, according to the fire risk classification proposed by Kodur [2]. In particular, to simulate fire scenarios more realistic for road bridges, natural fire curves have been obtained through zone models in CFAST [12] and the fire performance was assessed according to FSE, considering the performance levels for bridges, proposed in Sect. 3.

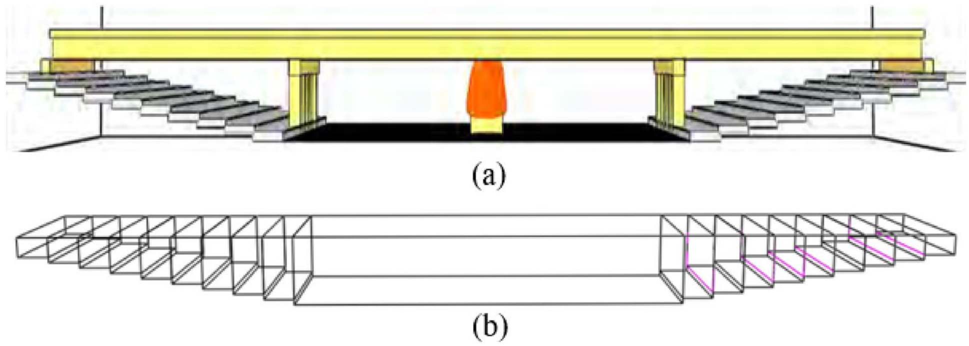


Fig. 8 **a** FDS model. **b** CFAST model

4.2.1 Fire scenarios and modelling

The volume below the bridge was modelled in CFAST with two closed faces, the ceiling and the floor, while the other faces were completely opened, to simulate the real ventilation condition. The fire was modelled by choosing the location of the ignition point and inserting the heat release rate (HRR) curve of the vehicle subject to fire. The gas temperatures were recorded by thermocouples located at different positions, to investigate the temperature evolution at which the bridge was subjected.

This model was validated with the results of advanced modelling in FDS [19] performed by Wright et al. [20] in which 14 simulations of bridge fires were presented, varying the type of vehicle (bus, HGV, 1/2 HGV, and tanker) and the position of the fire (in the middle of the central span or longitudinally/transversely translated). The studied steel–concrete composite bridge has 3 spans: the central one 35.8 m long and the two lateral ones of 25 m. To validate the model in CFAST, Case A (which corresponds to the fire of a bus located in the middle of the bridge central span) has been reproduced in CFAST, as shown in Fig. 8. The HRR curve related to the bus fire used in the reference project is shown in Fig. 9 and corresponds to a released thermal energy of 51,250.5 MJ.

In the report [20], for each studied case, Wright et al. provide the average temperature recorded in axis of the fire. Therefore, to obtain the same result, 11 thermocouples have been placed in CFAST, spaced 30 cm from each other above the fire ignition plane. The CFAST results in terms of average temperature recorded by each thermocouple are in a very good agreement with FDS results (Fig. 10), especially in the growing phase, while a higher temperature was simulated by CFAST in the cooling phase.

This result is justified by the fact that a zone model, being more simplified, can often achieve higher temperature than FDS ones [12]. In [20], the same analysis was carried out with other types of vehicles and other fire locations, concluding that the fire in the middle of the span is the most critical for the structure. For this reason, all the fire scenarios

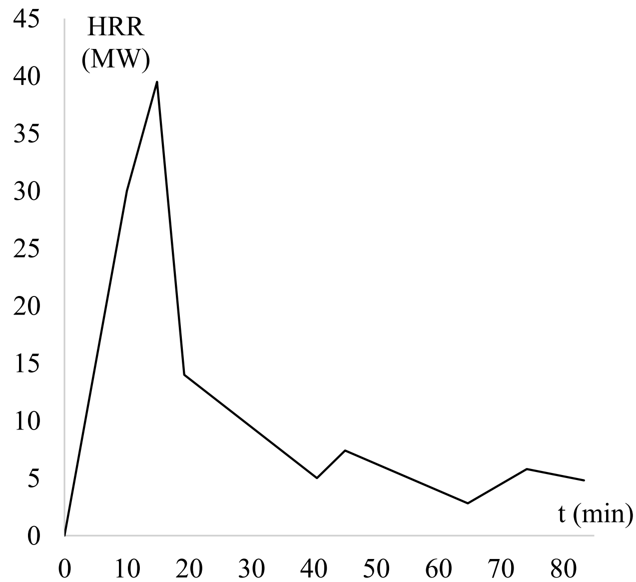


Fig. 9 HRR curve of a bus

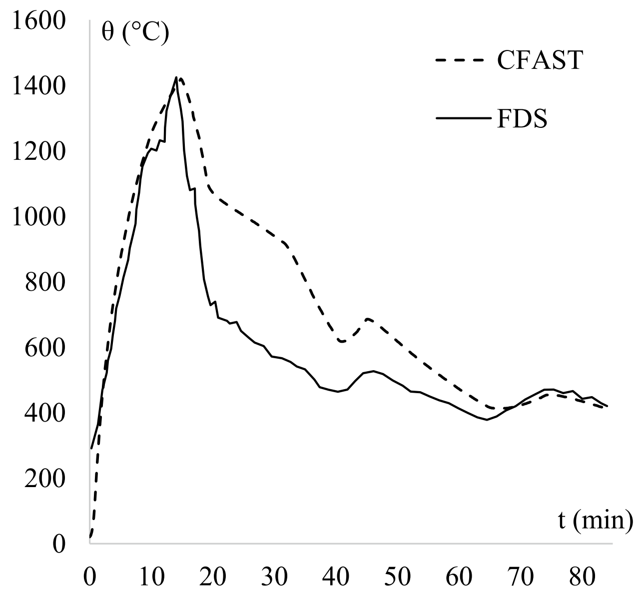


Fig. 10 Comparison between average temperatures in FDS and CFAST

Table 7 Five fire scenarios analysed

#Scenario	Involved vehicle	Total energy (MJ)
Scenario 1	HGV	247.983
Scenario 2	Truck	100.680
Scenario 3	School bus	41.432
Scenario 4	Internal combustion engine car	11.188
Scenario 5	Electric car	9.326

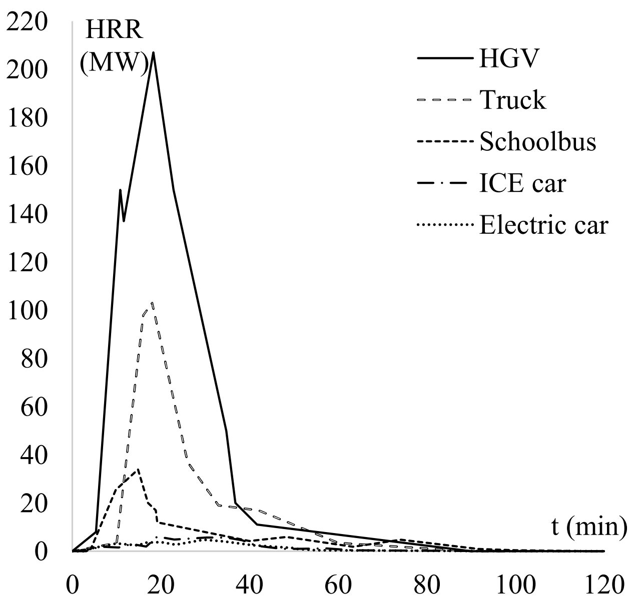


Fig. 11 HRR curve of the five vehicles

analysed below will involve the fire of different vehicles located in the middle of the span.

After the CFAST model validation and since the good agreement between the CFAST and FDS results, the following parametric analyses were carried out with CFAST, which has a lower computational burden than FDS. The volume below the bridge was modelled in CFAST as explained above: it is a volume 55 m long, 10 m wide, and 6.5 m high, corresponding to two bridge spans of equal size, which is the same analysed in Sect. 4.1 (Fig. 5, system 3a). Five zone fire models were carried out corresponding to the fire of five different vehicles: an HGV, a truck, a school bus, a car with an internal combustion engine (ICE), and an electric car (see Table 7). In all these scenarios, the vehicle was located in the most critical position, i.e., in the middle of the left span of the bridge.

The HRR curves corresponding to the fires of the five vehicles are selected from literature ([20–23]) and they are shown in Fig. 11.

The temperatures were recorded by 10 thermocouples arranged along the longitudinal development of the beam at a height of 4.92 m, corresponding to the lower flanges of the steel profiles. The thermocouples layout and the 10 zones in which the volume below the bridge was divided are shown in Fig. 12. The position of each thermocouple and the dimensions of each zone are explained in detail in Table 8.

The volume below the left span of the bridge has been divided into nine zones of equal length (3 m) except zone 5, where the fire is located, which is 3.5 long to consider the maximum temperature in a larger area. The right span has been schematized as a single zone 27.5 m long, considering for safety reasons that the temperature in the whole zone was the one recorded by the thermocouple T10. The temperatures θ recorded in scenario 1 in each zone are shown in Fig. 13, indicating that the maximum temperatures were recorded in the thermocouples T5, T4, and T6, which are the closest to fire ignition; while, in the other thermocouples, the temperature rapidly decreases due to the full ventilated conditions.

4.2.2 Thermo-mechanical analyses

After obtaining the natural fire curves in the fire scenarios explained in the previous section, advanced thermo-mechanical analyses were carried out following the performance-based approach.

Fig. 12 **a** Thermocouples layout and **b** Discretization in 10 zones

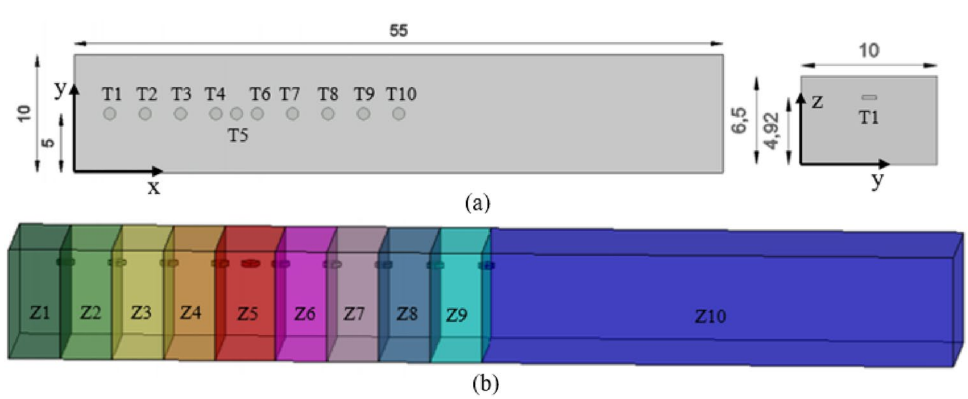


Table 8 Thermocouples and zones geometric features

#Thermocouple	#Zone	Thermocouples coordinates			Zones dimensions		
		x (m)	y (m)	z (m)	Length (m)	Width (m)	Height (m)
1	1	3.00	5.00	4.92	3.00	10	6.50
2	2	6.00			3.00		
3	3	9.00			3.00		
4	4	12.00			3.00		
5	5	13.75			3.50		
6	6	15.50			3.00		
7	7	18.50			3.00		
8	8	21.50			3.00		
9	9	24.50			3.00		
10	10	27.50			27.50		

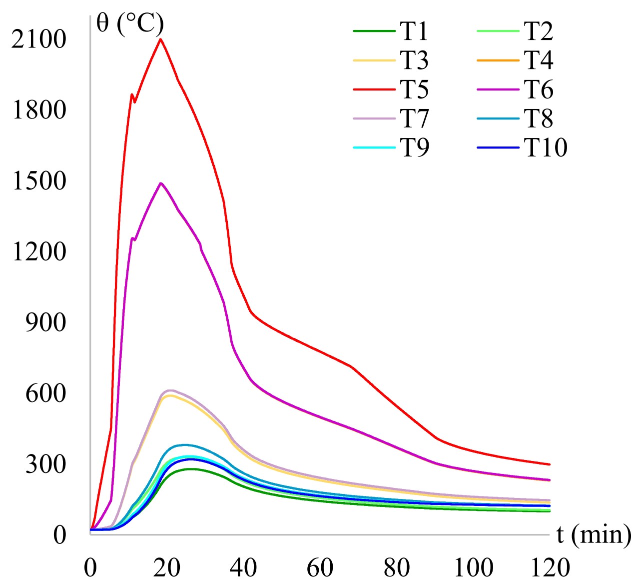


Fig. 13 Temperatures recorded by ten thermocouples in Scenario 1

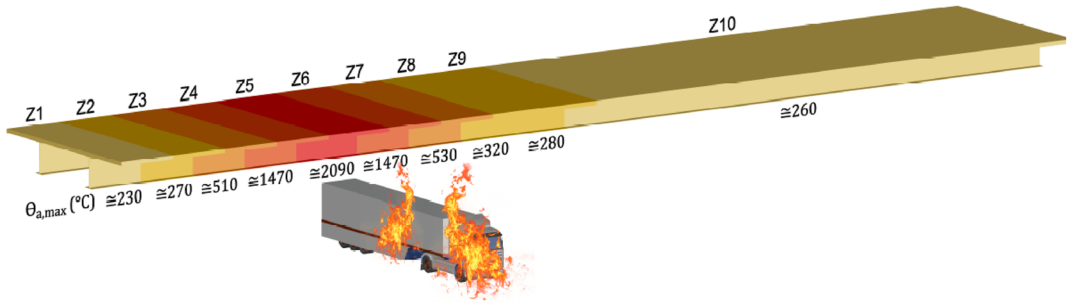
As explained before, the bridge was divided in 10 zones (see Fig. 12), in which different temperatures were recorded during the zone fire models; these temperature curves were used as input in the thermo-mechanical analyses. The first step was to perform thermal analyses of the bridge sections, varying the fire scenarios; Fig. 14 represents the maximum steel temperatures $\theta_{a,max}$ reached in the profile; these temperature evolutions vary according to the ambient temperature; indeed, moving away from the fire, they rapidly decrease due to the elevated ventilation. In particular, the plume temperature measured by the target located at the centerline of the fire plume was conservatively considered in the thermo-mechanical analyses, with reference to zone 5 (see Fig. 12). The plume temperature value was based on the Alpert and Heskestad [24] model and experimentally validated [25].

Focusing on the scenario 1-zone 5, where the maximum temperatures are reached, Fig. 15 shows the temperature trends in the steel profile, founding that in the web and in the lower flange, the temperatures are very similar to each other, both in the heating and in the cooling phases. The heating rate in the upper flange is slower, thanks to the shadow effect offered by the lower flange and to the presence of the concrete slab. This very high sectional temperature is purely theoretical; indeed, at 2100 °C, the steel is already be melted. However, SAFIR never had to handle that such high temperature as the beam will collapse much earlier, this temperature is reached, as also demonstrated in the following thermo-mechanical analyses.

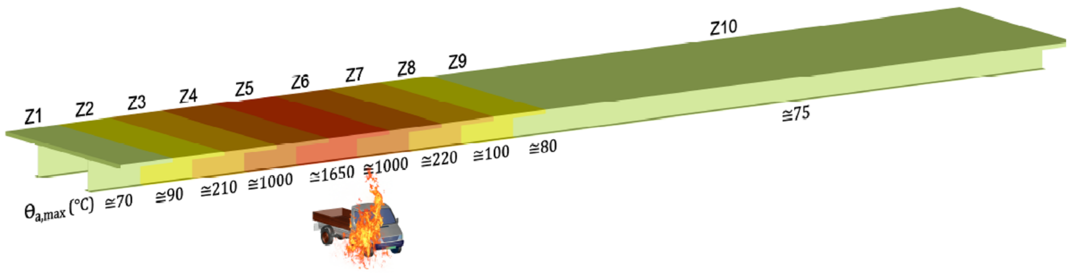
Known the temperatures in the steel profile, mechanical analyses were carried out to evaluate the structural behaviour of the bridge under natural fire conditions. As a result, the bridge in scenario 1 fails in about 9 min (Fig. 16). In every scenario, near the central support, where the negative moment M_{ed}^{-} is maximum, temperatures are less than 400 °C (see Fig. 14) and so no reduction in resistant bending moment M_{rd}^{-} is considered. On the contrary, in the section of maximum positive moment M_{ed}^{+}, the resistant moment M_{rd}^{+} starts to decrease after about 7 min due to the high temperatures reached, since it is located very close to the fire axis.

As can be seen from Fig. 16b, in about 9 min, the resistant bending moments are reached both in the middle of the span and in the central support and, therefore, a collapse mechanism is generated with a consequent failure of the beam.

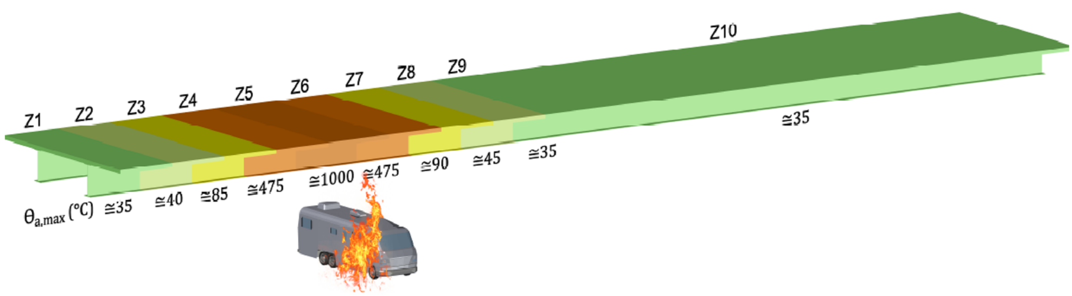
A similar behaviour was recorded in Scenario 2, in which the bridge was subjected to the fire of a truck in the same position and the failure occurred in about 15 min. In Scenarios 3, 4, and 5 (school bus, ICE car, and electric car), significantly lower temperatures are recorded, and therefore, the bridge does not fail for the entire duration of the fires. For example, Fig. 17 shows the maximum deflection and bending moment's trends in Scenario 3. It can be seen that, after the temperature peak, there is a decrease in displacements and stresses thanks to the progressive cooling of the section.



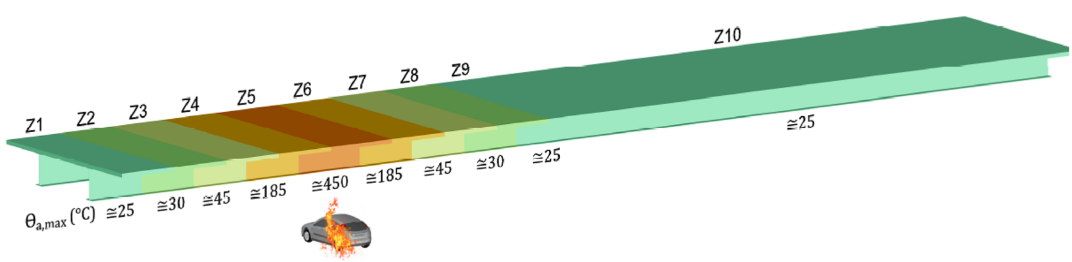
(a) HGV



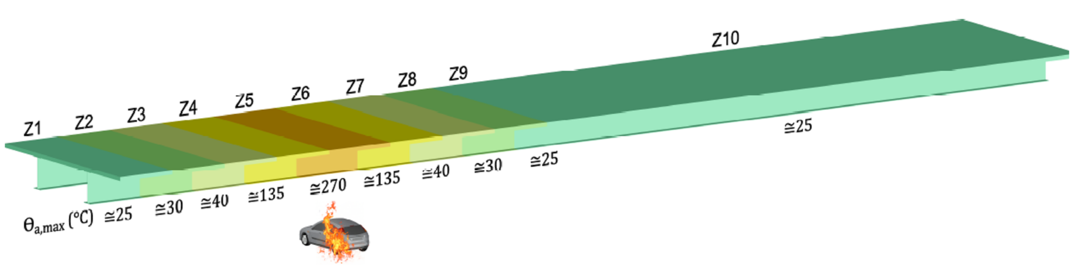
(b) Truck



(c) School bus



(d) ICE car



(e) Electric car

Fig. 14 Maximum temperatures in the steel profile in the 5 scenarios

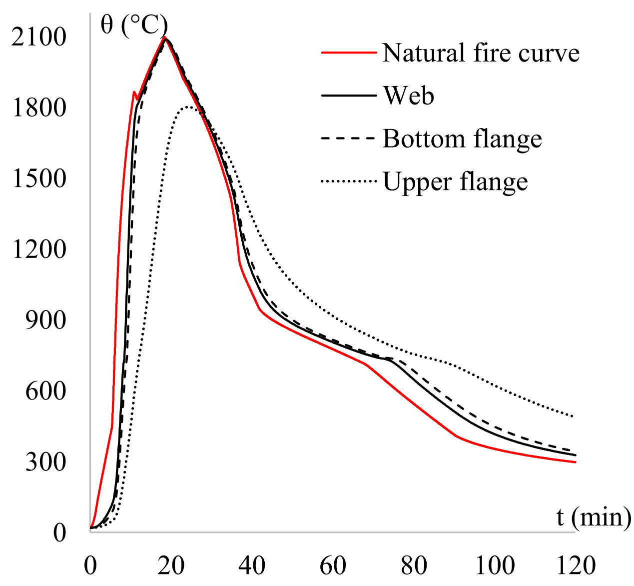


Fig. 15 Temperatures in the steel profile (Scenario 1-zone 5)

In conclusion, analysing the results of the previous analyses, the most critical situation is reached in Scenarios 1 and 2, where the fire of an HGV and a truck was simulated. These scenarios are the most critical both from failure and displacements point of view, so designing a fire mitigation strategy is necessary to avoid the structural failure (performance level II) or to limit the recorded damage (performance level III or IV). In case of light vehicles fires (Scenarios 3, 4, and 5), designing a fire protection is not necessary, since the bridge does not fail during the fire, showing generally limited damages (displacement amounts).

5 Design of fire vulnerability mitigation

Fire mitigation strategies can be implemented to prevent or reduce fire effects in structures and infrastructures. According to the concepts described in Sect. 2, the fire mitigation strategies affect the fire risk of a bridge and so their effect can be quantified by re-evaluating the importance factor. The common fire mitigation features of bridges are grouped in three main parameters: (I) security, (II) laws and regulation, and (III) fire protection and insulation features, as shown in Table 9.

To increase the fire performance of the analysed bridge, a passive protection with a spray applied fire resistive material (SFRM) was designed. The nomogram [26] can be used to design the protection thicknesses needed to guarantee the prescriptive requirements. For this purpose, the nomogram showing temperatures of protected and unprotected steel sections exposed to hydrocarbon fire curve was calculated (Fig. 18).

Regarding the design utilization factor of system 1 (simply supported beam constrained with a hinge and a spin), the needed protection thicknesses depending on the importance factor are shown in Table 10.

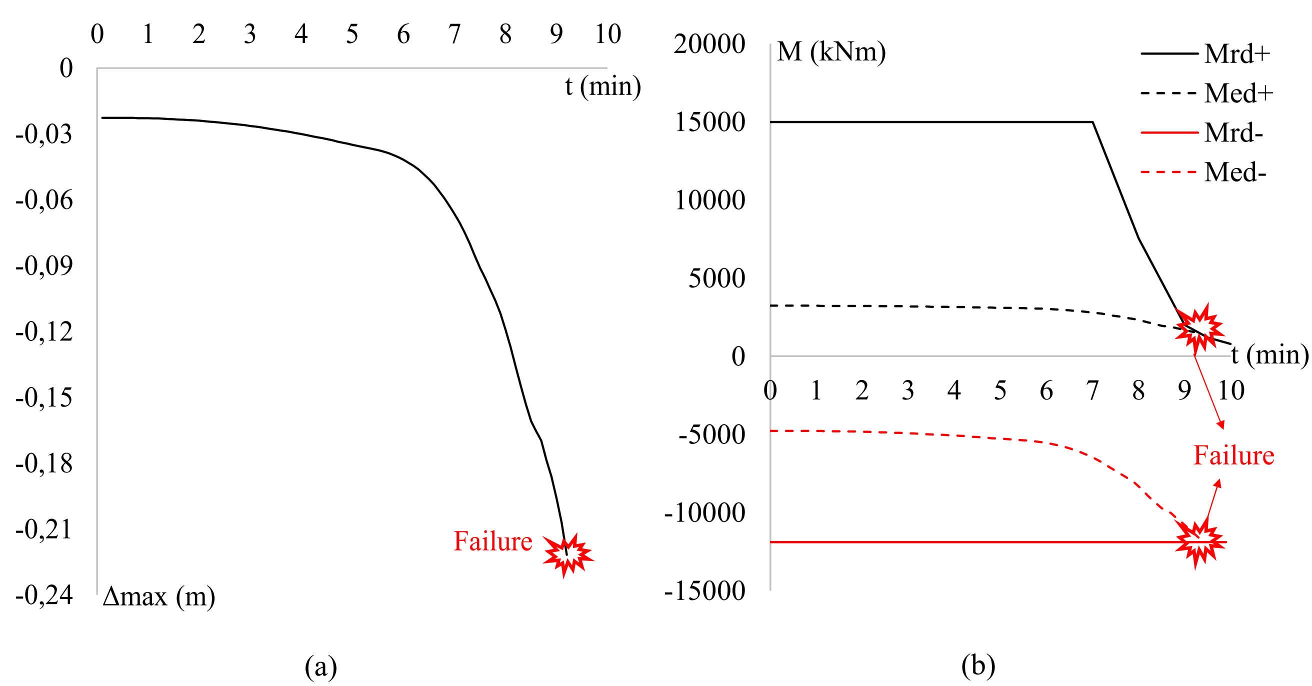


Fig. 16 **a** Maximum deflection and **b** bending moments in Scenario 1

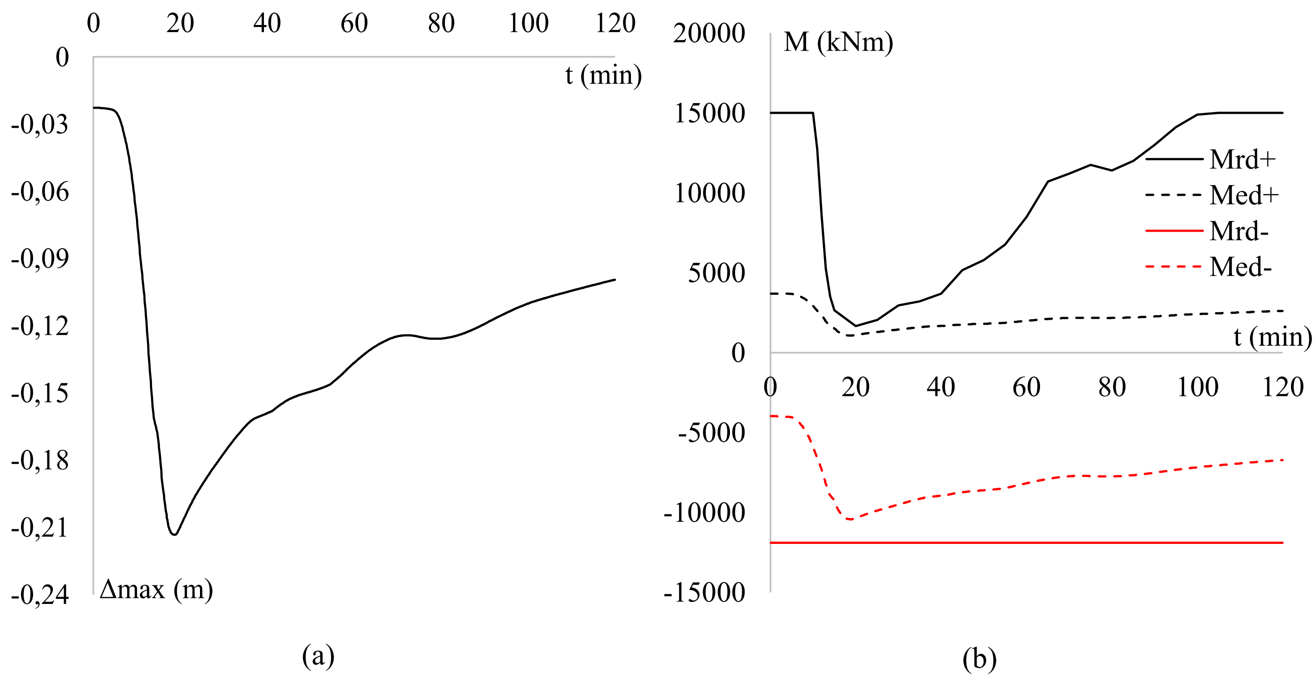


Fig. 17 **a** Maximum deflection and **b** bending moments in Scenario 3

Table 9 Proposed fire mitigation strategies, adapted from [19]

Parameter	Sub-parameter
Security	Monitoring systems
	Guards
	Restricted access zones
	Fire detection systems
Laws and regulations	Provide distinguished exits for large fuel tankers
	Limit operation timings
	Limit vehicle speed
	Limit transport size (20,000 l)
Fire protection and insulation features	On site firefighting equipment
	Use of flooding agents and/or foam deluge systems
	1 h insulation to main structural members
	2 h insulation to main structural members
	Implementing structural fire design for bridge

In case of low- and medium-risk grade, the application of fire protection is not required. Importance factors of 1.2 and 1.5 (high and critical risk grades) correspond to fire resistance requirements $t_{R,req}$ of R60 and R120, respectively. These requirements are guaranteed with the application of 8 mm and 16 mm of SFRM.

Temperatures in the steel profile protected with 8 mm and 16 mm of SFRM are shown in Fig. 19; the effect of the higher thickness protection is evident both for reaching lower steel temperatures and for reducing the heating rate in the profile [27] (Fig. 19b).

The results of the thermo-mechanical analyses of the protected bridges, in terms of failure time $t_{R,SAFIR}$, are shown in Table 11.

The fire protection thickness was first calculated according to the utilization factor of system 1 and it is used for all the analyses listed in Table 11. From the same table, it can be observed that the System 1 always satisfies the required fire performance time, varying the thicknesses protection. Also, the System 2, thanks to the chain effect, is able to satisfy the fire resistance requirements. Systems 3a and 3b, on the other hand, with the same protection thicknesses do not guarantee the design resistance requirements, since the values and the distribution of the internal forces in systems 3a and 3b are very different from the ones of system 1, also for the presence of the redundant actions and their variation during the fire exposure due to the constrained thermal deformations.

Therefore, for these systems, greater fire protection thicknesses have to be provided. In particular, thicknesses of 16 and 22 mm of SFRM have to be chosen for satisfying the R60 and R120 requirements (Table 11).

As for unprotected structures, the collapse was interpreted also checking the deflection of the bridges and comparing it with the limit of *L*/30.

Table 12 shows that *L*/30 is reached in the systems 1 and 2, varying the protective thickness, while in systems

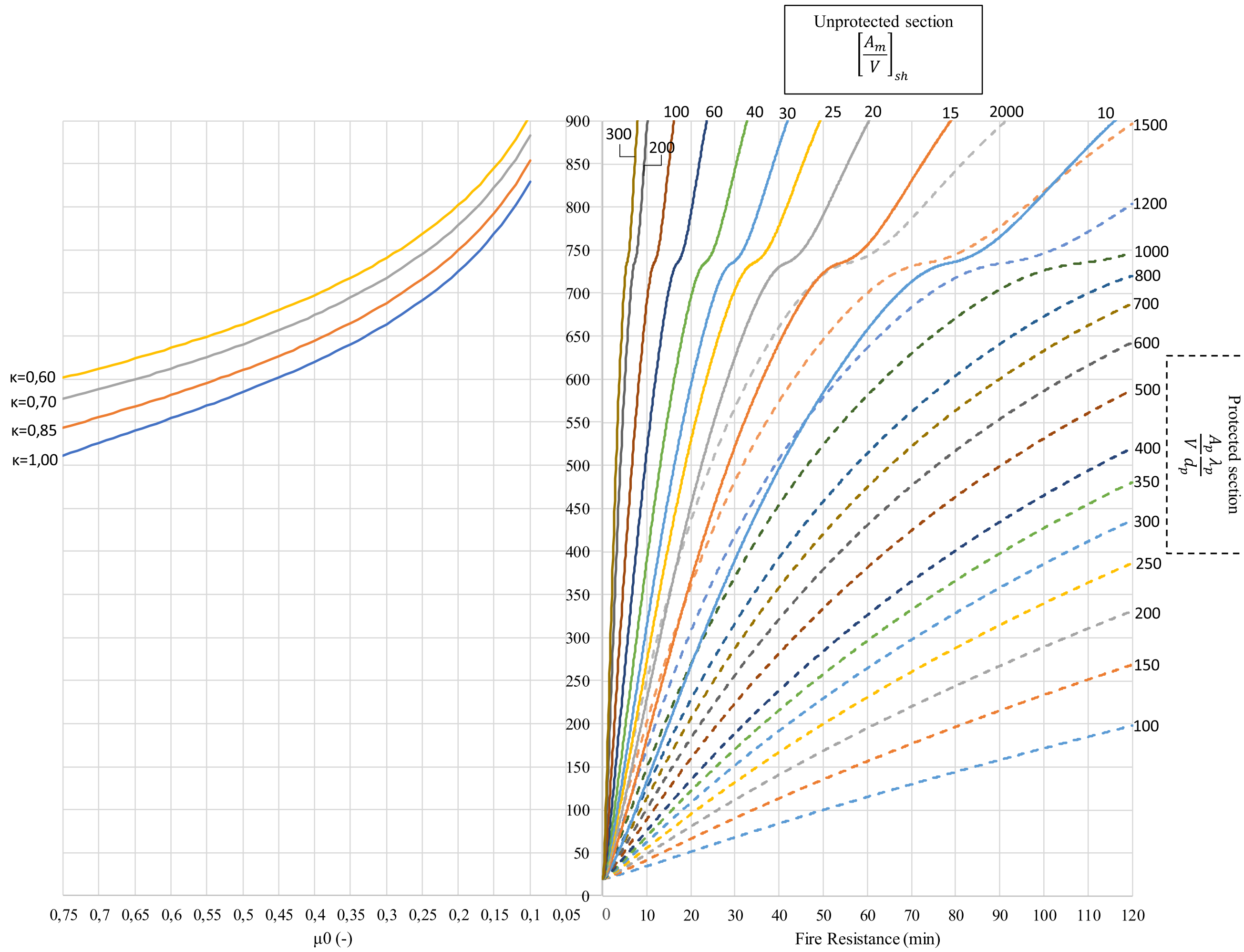


Fig. 18 Nomogram for hydrocarbon fire curve

Table 10 Protection thicknesses designed

	IF = 0.8	IF = 1.0	IF = 1.2 ($t_{R,req}$ = 60 min)	IF = 1.5 ($t_{R,req}$ = 120 min)
Protection thickness (mm)	–	–	8	16

3a and 3b, even with protections, the structural failure occurs before reaching the displacement of $L/30$.

Remembering that, according to the criterion introduced in Sect. 2, the displacement has to be less than $L/30$ for 60 and 120 min if the risk grade is high or critical, the systems 1 and 2 protected with thicknesses equal to 8 mm and 16 mm do not satisfy these requirements. A protection thickness equal to 22 mm is sufficient for system 1 to satisfy the R120 requirement, while a greater thickness would be required for system 2, because it collapse at 115 min. Therefore, for the structural schemes 1 and 2, this design criterion is more restrictive than the nomogram one.

Figure 20 represents the deflections trends in the four structural schemes varying protected systems (i.e., the protection thickness), showing that as the protection thickness increases, the structural deflection decreases; observing these figures, it is evident that for systems 3a and 3b, the structural failure occurs before reaching a deflection of $L/30$.

Finally, the risk grades and the importance factors for the four structural schemes varying the protection system were re-evaluated. Table 13 shows that, for 8 mm of fire protection, low beneficial effects are provided and there is no reduction of the risk grade, and for 16 mm, beneficial effects are evident only for the systems 3a and 3b, while with 22 mm for all the systems, the risk is reduced except for system 2 which is again the most critical one (see Sect. 4.1).

Considering the analyses according to the performance-based approach, the most critical fire scenarios 1 and 2 require the application of a fire protection, and so, an SFRM

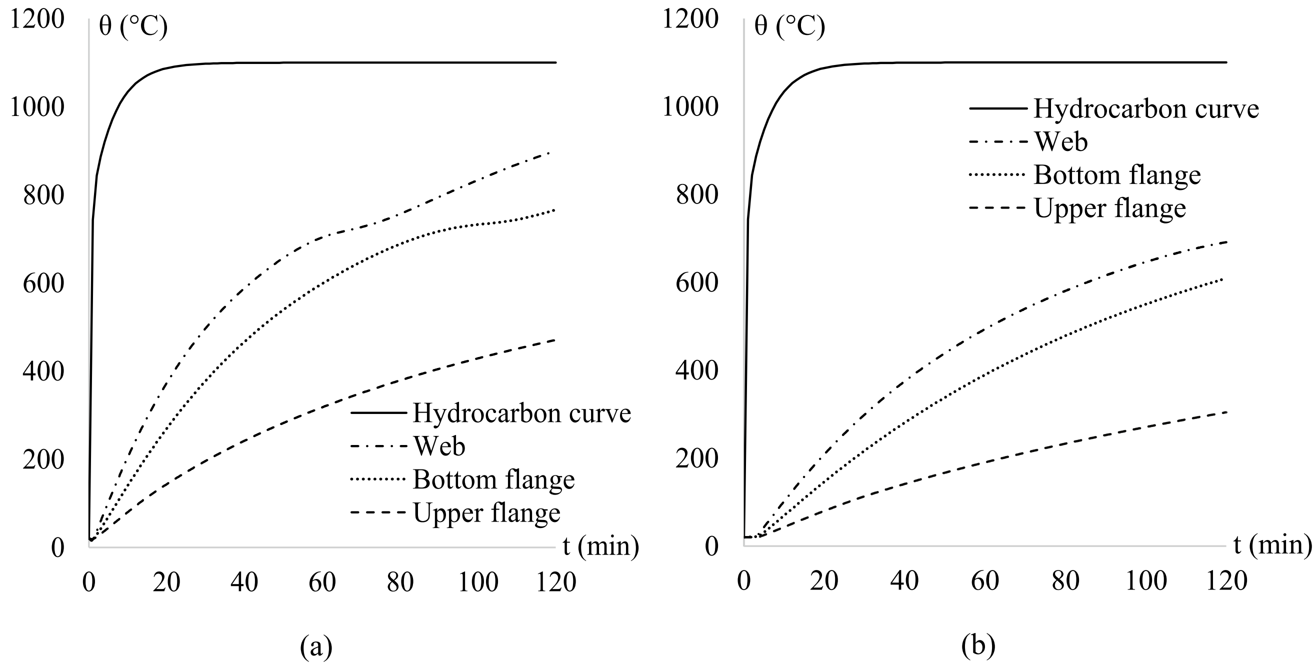


Fig. 19 **a** Temperatures in the steel profile with 8 mm and **b** with 16 mm of SFRM under hydrocarbon fire

Table 11 Failure times recorded in SAFIR in the four structural systems

$t_{R,SAFIR}$ (min)				
#System	Unprotected	8 mm (R60)	16 mm (R120)	22 mm
System 1	6.9	71.3	> 120	> 120
System 2	15.6	> 120	> 120	> 120
System 3a	5.0	46.6	91.0	> 120
System 3b	5.1	45.7	89.0	> 120

Table 12 Times at which *L*/30 is reached

$t_{L/30}$ (min)			
#System	8 mm (R60)	16 mm (R120)	22 mm
System 1	54.3	106.7	> 120
System 2	44.5	84.6	115
System 3a	–	–	–
System 3b	–	–	–

with a thickness of 16 mm was chosen, with a consequent reduction in steel temperatures (see Fig. 21a).

Decreasing temperatures, the bridge does not fail for the entire duration of the most critical fire scenario, with a maximum deflection of 11.3 cm recorded after 91 min (Fig. 21b).

Therefore, considering a fire protection of 16 mm allows to reduce the steel temperatures and also the stresses and displacements, giving to the bridge the possibility to satisfy PLIII or PLIV.

6 Results' comparison and discussion

To compare all the performed analyses, a benchmark between all the results is discussed in the following.

Table 14 summarizes all the results of the thermo-mechanical parametric analyses with the prescriptive approach.

The considered failure time $t_{failure}$ is the minimum between the time at which failure is achieved in SAFIR and the one at which the limit deflection *L*/30 is recorded. In particular, under the hydrocarbon fire curve, the four systems failed in about 5 min if the beam was unprotected, not satisfying any performance level. By considering a passive protection with an applied spray fire resistive material (SFRM) thickness of 16 mm, it is possible to verify the achievement of performance level II, III, or IV, varying the structural system. To verify performance level III or IV, it is necessary to evaluate the displacement recorded at $t_{II} = 60$ min, checking that it does not exceed *L*/100 or *L*/250 for PLIII or PLIV.

The systems 1 and 2 protected with 16 mm of SFRM fails at about 107 min and 85 min, respectively, so only the requirement of PLII is satisfied. The systems 3a and 3b are able to satisfy also PLIII, being the maximum displacement less than *L*/110 for 60 min.

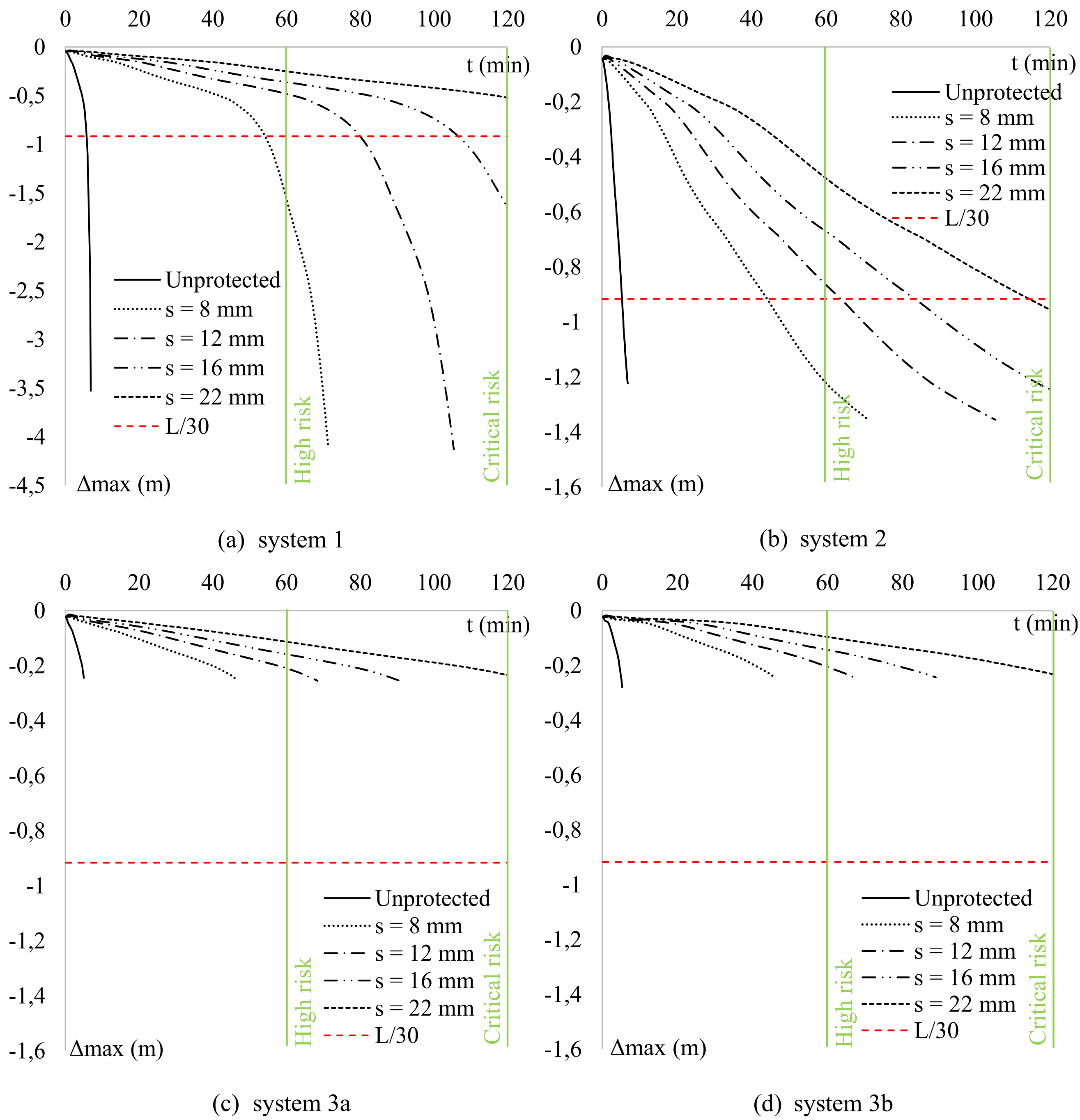


Fig. 20 Maximum deflections in the considered structural schemes under hydrocarbon fire

Table 13 Fire risk grade re-evaluation

Fire risk grade				
#System	No protection	8 mm (R60)	16 mm (R120)	22 mm
System 1	High	High	High	Medium
System 2			High	High
System 3a			Medium	Medium
System 3b			Medium	Medium

Table 15 summarizes the results of the five fire scenarios analysed with the performance-based approach.

In this case, the PL verification is direct; indeed, it is not necessary to define a time to evaluate the performance, but the entire duration of the fire is considered. The system 3a was subjected to several fire scenarios, finding that the analysed bridge, without passive fire protection, fails only in the case of HGV and truck fires, satisfying only the PLI in the truck case.

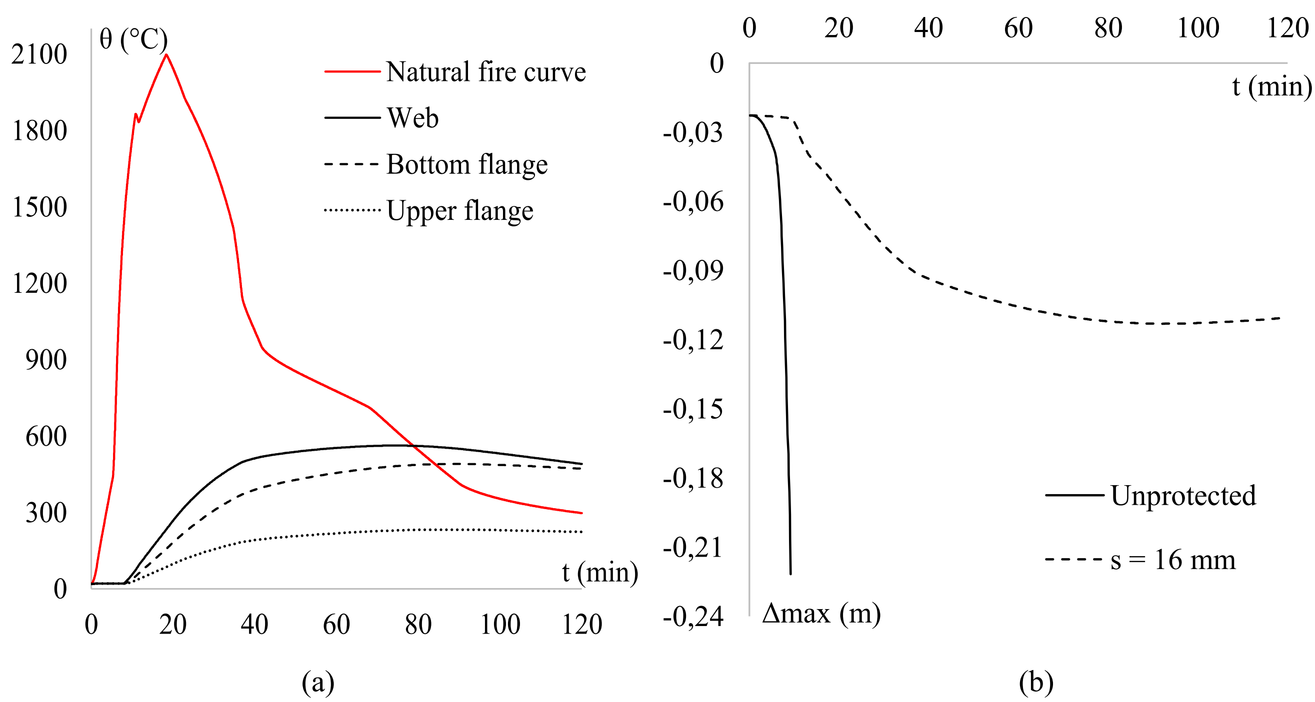


Fig. 21 **a** Temperatures in the steel profile protected with 16 mm of SFRM (Scenario 1-zone 5), and **b** deflections of protected and unprotected beam (Scenario 1)

Table 14 Results obtained in prescriptive approach analyses

#System	Fire curve	Protection thickness (mm)	t_{failure} (min)	$\Delta_{t_{\text{II}}}$ (m)	$\Delta_{t_{\text{II}}}/L$ (–)	PL
System 1	Hydrocarbon	–	$5.8 < t_{\text{I}}$	∞	∞	–
		16	$106.7 > t_{\text{II}}$	0.36	$\frac{1}{86} \geq \left(\frac{\Delta}{L}\right)_{\text{III}}$	II
System 2	Hydrocarbon	–	$5.4 < t_{\text{I}}$	∞	∞	–
		16	$84.6 > t_{\text{II}}$	0.67	$\frac{1}{41} \geq \left(\frac{\Delta}{L}\right)_{\text{III}}$	II
System 3a	Hydrocarbon	–	$5.0 < t_{\text{I}}$	∞	∞	–
		16	$91.0 > t_{\text{II}}$	0.16	$\left(\frac{\Delta}{L}\right)_{\text{IV}} \leq \frac{1}{172} \leq \left(\frac{\Delta}{L}\right)_{\text{III}}$	III
System 3b	Hydrocarbon	–	$5.1 < t_{\text{I}}$	∞	∞	–
		16	$89.0 > t_{\text{II}}$	0.14	$\left(\frac{\Delta}{L}\right)_{\text{IV}} \leq \frac{1}{196} \leq \left(\frac{\Delta}{L}\right)_{\text{III}}$	III

In both Scenario 1 and 2, a fire protection with SFRM (thickness 16 mm) is applied, avoiding failure and giving the possibility of reaching PLIII and PLIV, because the displacements are less than $L/100$ or $L/250$, respectively.

Considering the fire scenarios 3, 4, and 5, the failure does not occur for the unprotected structure, satisfying PLII. If PLIII is required, any fire protection is still necessary, while, in the case of PLIV, only the school bus fire needs a fire protection for limiting the displacement at $L/250$.

From the comparison between the results obtained with the two approaches, it is evident that carrying out an advanced analysis following a performance-based approach allows to consider less sever and more realistic fire conditions, thanks to the use of natural fire curves, which lead to an optimization in protections design. In performance-based analyses, a protective layer of 16 mm is enough to ensure that the bridge does not fail for the entire duration of the fire, recording limited deflections even in case of very serious fires such as the HGV or truck ones. Furthermore, in case of the most common fires, i.e., those of light vehicles, it is not necessary to provide a fire protection to the bridge, being able to satisfy performance levels III or IV.

Table 15 Results obtained in performance-based approach analyses (continuous beam bridge—scheme 3a)

#Scenario	Total HR (MJ)	Protection thickness (mm)	Δ_{max} (m)	Δ_{max}/L (–)	Failure	PL
Scenario 1 (HGV)	247.983	–	∞	∞	YES (9.2 min)	–
		16	0.113	$\left(\frac{\Delta}{L}\right)_{IV} \le \frac{1}{243} \le \left(\frac{\Delta}{L}\right)_{III}$	NO	III
Scenario 2 (truck)	100.680	–	∞	∞	YES (15.2 min)	I
		16	0.095	$\frac{1}{290} \le \left(\frac{\Delta}{L}\right)_{IV}$	NO	IV
Scenario 3 (school bus)	41.432	–	0.211	$\left(\frac{\Delta}{L}\right)_{IV} \le \frac{1}{130} \le \left(\frac{\Delta}{L}\right)_{III}$	NO	III
Scenario 4 (ICE car)	11.188	–	0.088	$\frac{1}{313} \le \left(\frac{\Delta}{L}\right)_{IV}$	NO	IV
Scenario 5 (electric car)	9.326	–	0.064	$\frac{1}{430} \le \left(\frac{\Delta}{L}\right)_{IV}$	NO	IV

7 Conclusions

This paper proposes the base of a strategy for the design and verification of bridges under fire conditions. In particular, the focus is on the identification of fire performance levels for bridges, giving information also about the selection and modelling of bridge fire scenarios within the framework of the performance-based approach principles. The proposed approach can be useful both for designers and industrial category to assess the bridge performances in fire, not only according to prescriptive approach but also considering the performance-based one.

Starting from a deep literature review, some preliminary conclusions can be drawn:

- fire can represent a severe hazard for bridges and it can lead to significant damages or failure of structural members. The effects of fire on bridges can be mitigated by designing appropriate fire resistance to structural members;
- the probability of bridge fire is lower than the building one. However, the impact of a fire on bridge structure can be more critical due to lack of adequate fire protection and firefighting measures;
- to date, there is no specific regulatory framework for the design and assessment of bridges in fire conditions;
- the methodology proposed by Kodur et al. could be a valid guideline in case of prescriptive approach application, taking into account the level of vulnerability and the critical nature of the bridge to evaluate its importance factor;
- four performance levels can be defined for the assessment of fire resistance of bridges, starting from the ones proposed for structures by the Eurocodes and these performance levels can be linked to the fire risk classification proposed by Kodur et al.

To understand all the parameters that can influence the fire behaviour of bridges, and to apply the methodology proposed in the first part of the paper, parametric analyses of a typological steel–concrete fully composite bridge were carried out, using both the prescriptive- and performance-based approaches. The main conclusions are the following:

- according to the prescriptive approach and considering the hydrocarbon fire curve, the bridge failure was always achieved in about five minutes. To avoid the structural collapse, a fire protection has to be designed for the structural element, satisfying a performance level II and also a limited damage according to the performance level III;
- for satisfying the performance level IV, for which no damage has to be provided, a proper thickness fire protection has to be designed;
- thanks to the fire protection, the risk of bridges can be mitigated, changing its classification according to the Kodur methodology.

From the application of the performance-based approach, it emerges that:

- considering the fires of the most common light vehicles, the unprotected bridge does not fail for the entire duration of the fire with limited or no damage. In case of fires involving heavy vehicles, the application of fire protection is required, ensuring limited damage;
- the application of performance-based approach allows to consider more realistic fire conditions, thanks to the use of natural fire curves, leading to an optimization of the protection system design;
- the proposed performance level for bridges allows to quantify the structural fire response of the bridges, according to its intrinsic fire risk, providing also technical criteria for its verification.

Further developments of this research certainly include advanced analyses to investigate other bridges typology, structural schemes, and fire scenarios, to further validate the proposed performance level approach and the verification criteria related to them.

Acknowledgements This activity is carried out within the Research Agreement on "Guidelines on risk evaluation and safety check of existing bridges" between Consiglio Superiore dei Lavori Pubblici (Ministry of Infrastructure of Italian Government) and Reluis (Consorzio della Rete dei Laboratori Universitari di Ingegneria Sismica e Strutturale)—D.M. 578/2020 and following.

Funding Open access funding provided by Università degli Studi di Napoli Federico II within the CRUI-CARE Agreement.

Data availability The data is available upon written request from corresponding author.

Declarations

Conflict of interest The authors have no relevant financial or non-financial interests to disclose. This manuscript has no associated data.

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