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Fire hazard assessment, performance evaluation, and fire resistance enhancement of bridges

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Abstract

Although the performance of bridge structures under prescriptive fire scenarios has been the subject of numerous studies, performance-based approaches are yet to be developed to achieve an efficient and economical design. This paper presents a performance-based framework that identifies bridges at high fire risk, produces realistic fire scenarios, provides an open source tool to apply the realistic fire load to the thermomechanical model and evaluate the structural performance of the bridge. It also provides guidance to improve the fire resistance of the bridge. The proposed framework is implemented by simulating the I-65 overpass fire accident in 2002, Birmingham, Alabama, USA. Firstly, fire risk of the bridge is estimated by considering various criteria such as the social and economic impact of fire, structural vulnerability, and the likelihood of fire. Secondly, a realistic fire scenario is developed using the real fire accident data by conducting computational fluid dynamics (CFD) simulations. Thirdly, the newly developed open source FSDM framework is utilised to apply the realistic fire load to the thermomechanical model and finally, the fire resistance of the bridge structure is estimated. The unprotected bridge failed after 12 minutes of fire exposure which is found in compliance with the actual failure time of the bridge during the accident. Further thermomechanical analyses are performed applying different thicknesses of fire protection to estimate the suitable amount of fire protection to achieve improved fire resistance. It is observed that the fire resistance of the bridge can be enhanced up to 60 minutes by providing a fire protection of 12mm thickness. This framework presents an important methodology for the highway department and bridge engineers to identify bridges at high fire risk and accurately determine the amount of fire protection required to reduce the fire risk and enhance the fire resistance of these bridges.

Keywords: Fire risk assessment, performance-based design, fire load, fire resistance, fire protection, CFD

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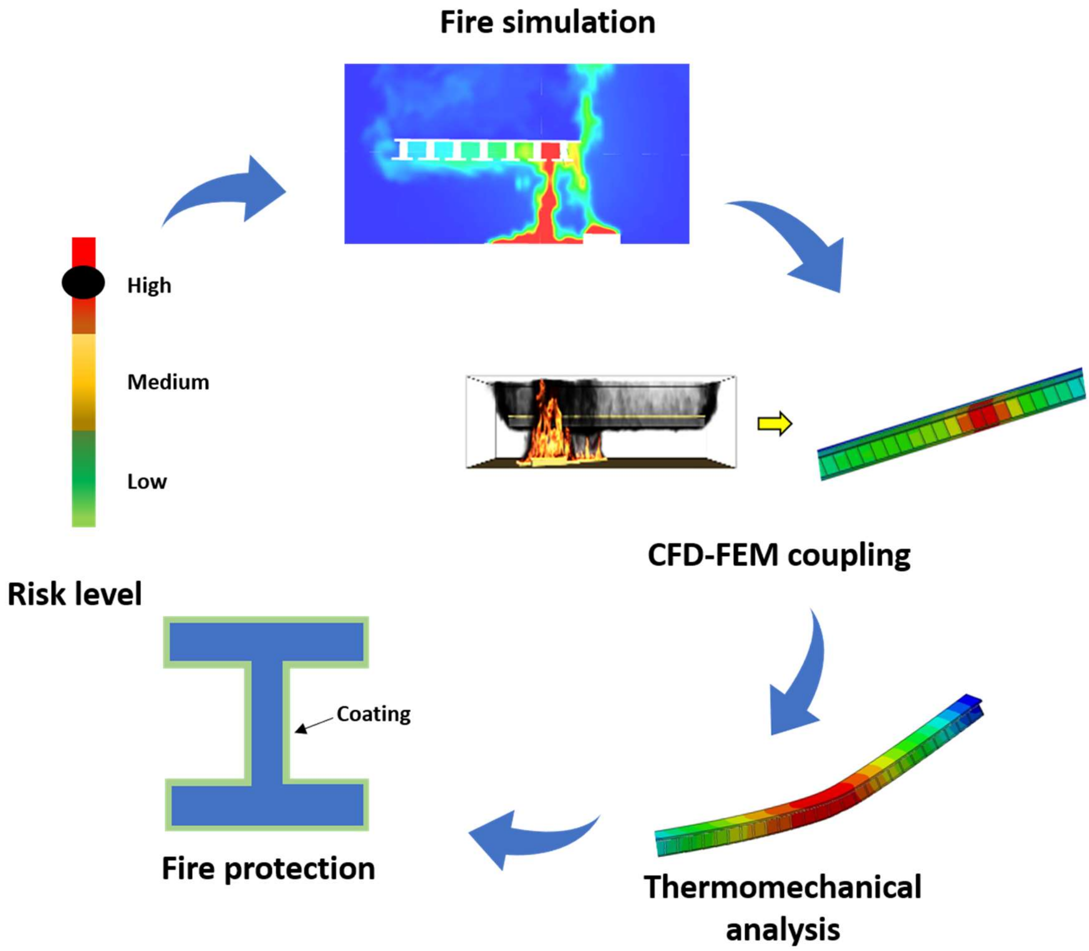
Graphical Abstract



1. Introduction

Fire is a severe hazard to built infrastructure and can significantly damage the structure. Bridge fires are responsible for the failure of bridges because they are characterized by a high heat release rate (HRR) which can result in the temperature rising as high as 1100 °C within few minutes of ignition [1]. A statistical study conducted by Lee et al. [2] showed that the number of bridges damaged due to fire is significantly more than the number of bridges damaged due to earthquakes. A total of 1062 bridge failures were studied by Lee et al. [2], fire was the cause of failure for 3.2 % of bridges compared to 1.8% and 2.1 % of bridges failed due to wind and earthquake, respectively. It was concluded that extensive research has been conducted to understand and predict the bridge behaviour under the effect of extreme loading conditions such as impact, earthquakes, and winds, while the bridge behaviour under fire is not well understood [2,3]. Due to the lack of attention in design codes for bridge fires design, it is required to develop a PBD method for bridge fire design that can enable fire engineers to design bridge structures for realistic fire loads. Though, designing all bridges for fire is also not appropriate and may increase the project cost substantially. Bridges that are more vulnerable to fire or possess high fire risk are only required to be designed for fire. Therefore, bridges at high fire risk are to be

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identified first. For fire risk assessment of bridge structures, a framework is required that considers all critical factors affecting the bridge fire risk such as socioeconomic importance of the bridge, structural vulnerability, and likelihood of fire accidents. Only a few research has been conducted to estimate the bridge fire risk [4–6]. Khan et al. [6] have developed a fire risk assessment framework for bridge structures where weighting to different factors and classes have been calculated depending upon their relative importance. Once a bridge with high fire risk is identified, it provides an indication to structural fire engineers if the bridge is required to be designed for fire effects or not and if the existing bridges at high fire risk are to be provided with fire protection. The level of fire protection required for different bridges can vary significantly. Therefore, it is required to estimate the correct amount of fire protection for each bridge that is at high fire risk. The accurate level of fire protection can only be estimated by analysing the thermomechanical behaviour of a bridge when it is exposed to a realistic fire.

There have been less than a handful of previous studies analysing the performance of bridge structures under fire situations. Paya et al. [7] performed a numerical study to understand the response of bridges subject to fire. A 3D numerical model of a typical steel girder bridge was developed, and a parametric study was performed considering different grades of structural steel for the girders. This study assumed non-realistic fire exposures such as Hydrocarbon (HC) fire and Stoddard's fire. These fire scenarios are highly conservative in terms of peak temperatures (peak temperature is around 1100 and 1500 °C for HC fire and Stoddard’s fire) and the extent of application because the entire bridge span is assumed to be exposed to a uniform temperature. Another experimental and numerical study was conducted by Aziz et al. [8] to understand the fire performance of typical steel girders used in bridges. Three steel-concrete composite girders were tested under simultaneous loading and ASTM E119 [9] fire exposure. The fire scenario considered in their study was not suitable for analysing the fire response of bridges as bridge structures are present in open spaces, which bear no resemblance to the approximately $3m^3$ compartment on which the “standard fire” is based. Song et al. [10] studied the behaviour of prestressed concrete box bridge girders under fire exposure. A 3D nonlinear finite element model of the prestressed concrete box bridge girder was developed using ANSYS software and the effects of fire exposure duration and simultaneous structural loading was studied. It was observed that a reduced fire exposed area or preventing fire exposure near the midspan (high bending moment zone) results in enhanced fire resistance and failure of prestressed concrete box bridge girders exposed to HC fire was mainly governed by the deflection rate criterion in many cases. There are only a handful of studies that considers realistic fire load estimation for bridge structures using CFD modelling. Various fire models were established using Fire Dynamics Simulator (FDS) which is an open-source CFD package for fire modelling - developed by NIST [11]. Wu et al. [12] studied the posttensioned concrete bridge behaviour under fire exposure and used spatially varied heat fluxes or AST as fire load. It was observed that the fire exposure resulted in residual deflection and reduced load-carrying capacity of bridge structure. In many cases, the failure of the posttensioned bridge can take place during the cooling phase because of the fact that heat continues to penetrate into the structure which increases the strand temperature. A numerical analysis of the fire of the I-65 overpass in Birmingham, USA was carried out by Moya et al. [13] . The realistic fire scenario was developed using CFD simulations, and the sequential thermomechanical analysis was performed using FE software (Abaqus) to understand the fire response of the bridge. The numerical model was used to study the influence of a realistic fire scenario (CFD versus standard fires). It was observed that applying the standard and HC fires along the entire length

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of the bridge do not accurately represent a real bridge fire. In most practical engineering designs, fire load is defined using standard fire curves such as the HC fire curve [14]. This prescriptive approach to represent the likely fire hazard intensity has proved to be inadequate and provoked new thinking to develop realistic fire scenarios that would account more faithfully for features of real fires. The most common method to estimate the demand imposed by complex fire scenarios on the bridge structures is to develop a CFD model. The gas temperatures, heat fluxes or adiabatic surface temperatures (AST) obtained as output from CFD analysis can be used to represent a realistic fire scenario. The fire scenarios generated using CFD simulations enable bridge fire safety engineers to analyse and design bridge structures more efficiently and accurately.

In this paper, a PBD methodology for analysing and designing bridge structures under fire exposure is proposed. The proposed approach enables the structural fire engineers to estimate the fire risk to any bridge structure. For accurate estimation of the fire resistance of the bridges at high fire risk, a realistic fire load would be developed by performing CFD simulations. A CFD-FE coupling methodology is developed to transfer the fire data obtained from CFD simulations to an FE model (heat transfer). The actual fire resistance of the bridge structure would be determined by performing a sequential heat transfer and thermomechanical analysis. The thermomechanical analysis would be performed with different amounts of fire protection to achieve an improved performance of the bridge. Using this approach, structural fire engineers can identify bridges at high fire risk and provide recommendations to improve their fire resistance.

2. Methodology

To estimate fire risk and design bridge structures against fire, this paper proposes a framework comprising five steps, i.e., 1) Fire risk assessment; 2) Realistic fire load estimation; 3) CFD-FE coupling; 4) Estimating fire resistance; 5) Enhancing fire resistance. In the first step, the fire risk of the selected bridge is assessed analytically using the framework developed by the authors [6]. Once the fire risk of the bridge under consideration is estimated, and it is established that the bridge is at high fire risk, a realistic fire scenario is developed in the second step using fire simulation. In the third and fourth steps, the AST [15] obtained from FDS simulations would be used as thermal boundary conditions to evaluate the fire performance of the bridge by conducting sequential heat transfer and thermomechanical analysis. In the final step, to enhance the performance of the bridge under the same fire load, the thermomechanical response is studied with different thicknesses of fire protection, and the amount of fire protection needed to achieve an improved performance would be estimated. This five-step methodology for assessing fire risk, producing a realistic fire scenario, evaluating structural performance, and enhancing its capacity is demonstrated by conducting a case study on the I-65 overpass fire in 2002, Birmingham, Alabama, USA.

2.1 Fire risk assessment

In this section, a framework, developed by Khan et al. [6] that considers various factors that can influence the fire hazard to a bridge is utilised to assess the fire risk to a bridge. In this study, three main criteria are considered which contribute to the fire risk of a bridge; (1) the social and economic importance of the bridge, (2) structural vulnerability, and (3) likelihood of a fire.

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Each criterion is decomposed into various sub-criteria, as shown in Fig. 1. For instance, time and cost to repair the damaged bridge are some of the important sub-criterion for assessing the social and economic importance of the bridge. These sub-criteria are further decomposed into alternatives, as shown in Fig. 1. Similar criteria, sub-criteria and alternatives were also used in many studies [5,16]. The individual weighting factors associated with each criterion, sub-criteria, and alternative were calculated Khan et al. [6] using the AHP process.

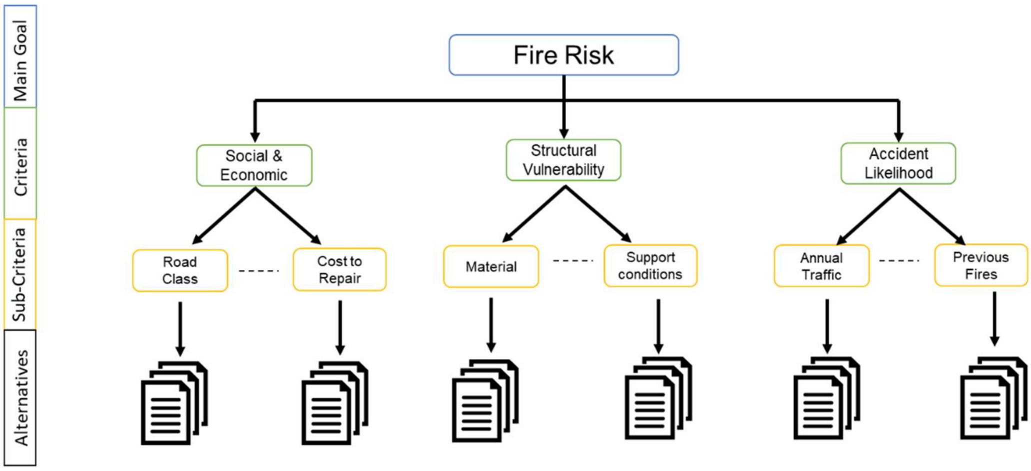

Figure 1 Hierarchical diagram for bridge fire risk assessment

On January 5th, 2002 a tanker truck travelling on the I-65 Birmingham bridge carrying 37.5 m^3 of gasoline swerved and crashed into the piers supporting the North East end of the central span. The piers supporting the girders survived the tanker impact as they were protected by a half meter wall around them. Whereas, the resulting fire scenario severely damaged the bridge girders within few minutes of exposure as they were not designed to resist any fire load. When the firefighters quelled the fire, it was found that one of the girders was heavily damaged and deflected up to 2.5 m at the location around 15 m. from its North end. The bridge deck was structurally irreparable and was replaced by constructing a new precast prestressed concrete deck.

The bridge was comprised of three spans with a total length of 88.53 m. The central span was 37.32 m. long and two lateral spans were 25.91 and 25.30 m long. Each span was a simply supported composite section with a steel girder connected to a reinforced concrete slab with shear studs. Other details such as annual traffic, age of the bridge and number of previous fire accidents that are required to calculate the fire risk of the I-65 Birmingham bridge have been taken from the literature [6]. The fire risk value for the I-65 Birmingham bridge is estimated using the factors defined in the literature [9] as 0.39, indicating that the bridge is at high fire risk i.e., during the actual accident, the steel girders of the main span sagged off about 3 m. The damaged part was demolished and rebuilt in 38 days [6]. Therefore, it is required to estimate the structural capacity of the bridge in terms of fire resistance, but before calculating the fire resistance of the bridge, a realistic fire scenario must be developed. In the next section, a realistic fire scenario is developed for the bridge by considering the details such as the size of the vehicle, HRR, etc., from the previous fire accident that occurred on the bridge and caused the collapse of the bridge.

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2.2 Realistic fire load estimation

If the fire risk estimated for the bridge is high, then it is required to evaluate the structural performance of the bridge under realistic fire exposure so that recommendations can be provided to improve its fire resistance. On the other hand, bridges with low fire risk do not require further investigation. Before conducting a thermomechanical analysis of the bridge to understand its structural fire response, it is critical to estimate the correct amount of fire load applied to the bridge. Fuel tanker fires are observed as the most common type of fire in a great number of severe bridge fire incidents [16–18]. It allows researchers to use HC fire to characterise fire hazards for bridges, resulting in a conservative estimation of fire load. While these prescriptive fires are not a true representation of a realistic bridge fire scenario where high-intensity fires occur only in a localised zone, and the assumption of applying prescriptive time-temperature curves uniformly along the entire bridge span is also not consistent with the principles of performance-based engineering [3,19,20]. However, due to the presence of bridges in the open environment, localised fires can be used to predict the most accurate fire scenario. In a real bridge fire incident, the intensity, and duration of fire may greatly be influenced by the locations and sizes of the vehicles involved in an incident.

Using prescriptive code-based time-temperature curves such as the HC fire curve [14], the external fire curve [19], or the standard fire curve [21] are the most prevalent ways of applying fire loading to bridge structures. The structural response of bridge structures exposed to these uniform or prescriptive fires is potentially unrealistic and can be over-conservative. A non-uniform fire scenario that can represent a fire with high intensity in the vicinity of the vehicle (near-field) and a decaying fire intensity at locations away from the vehicle (far-field) is therefore needed to develop a PBD approach. This can be achieved using fire inputs such as heat fluxes, gas temperatures, and AST from CFD models which considers a spatial decay of fire. Therefore, CFD fire simulations are sometimes used to define realistic fire scenarios. Some studies simulated two and three-dimensional fire spread using FDS, which is the most commonly used CFD package for fire simulation. To predict the CFD and heat transfer results within a reasonable range, the observation from real accidents (mainly by the fire department) provides a general idea of outputs for modellers given that very limited validation resources are available. In this paper, realistic fire scenarios have been developed and quantified using FDS 6.5.3 [22]. AST obtained from CFD analysis are coupled with an FE heat transfer model by establishing a coupling procedure. Using these fire scenarios, a realistic decay of hazard intensity along the span away from the vehicle can be simulated, resulting in more realistic thermomechanical responses.

2.2.1 Fire simulations

FDS is used to carry out the fire simulation. A simplified geometrical model of the I-65 Birmingham bridge is generated in FDS. The input parameters, such as material properties, soot yield, are collected from an article by Moya et al. [13], where the authors obtained the necessary data to conduct the numerical simulations from the Alabama Department of Transport. Fig. 2(a) shows the computational domain ($86\times40\times14m^3$) that includes the girders and slabs of the bridge and fire source. Except for the bottom surface, an open boundary condition is applied, which allows a continuous supply of air in the computational domain, and fire could be considered

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fuel-controlled where fuel burned locally [23,24]. It seems the classical case of localised fire and HC curve cannot be used to represent such fires. Furthermore, due to the large size of the fire, the localised fire models presented in Eurocode 1 [14] are not suitable to mimic such fire scenarios [20,25,26]. As FDS is based on LES, no grid independency test is carried out, but sensitivity analysis to the mesh size must be performed [27]. Therefore, several numerical simulations are performed until no variation in the value of ASTs is observed. Numerical simulations for the cell size of 0.3, 0.25, 0.2, 0.15, and 0.125 (all in metre) were conducted. To keep lower computational cost with reasonable accuracy, a cell volume of $0.2m^3$ is used in the current fire simulations.

In the fire simulation, the tanker is considered as a ‘burner’ of size $12\times2.5\times1m^3$, where the heat release rate per unit area (HRRPUA) is assigned as $2500kW/m^2$ [13]. For validating the results with the numerical study [13] and deflection observed after the accident, the HRRPUA for the spilt region is taken as $1000kW/m^2$. To capture the temperature profiles along the bridge span, AST devices are installed at a distance of 1 m. A total of 74 AST sensors are installed in the fire simulation; 37 sensors to record the temperatures on the steel girder, and 37 devices on the slab. Although ASTs are used as a thermal boundary condition in the current study [28], it is recommended to readers and engineers to be aware of the assumptions and limitations of this approach [20]. In this paper, the fire load location (exposure region) has been assumed as per the data obtained from real fire accident to validate the model in terms of failure time. Although, to simplify the uncertainties related to bridge fire location, authors suggest to apply the fire load (exposure region) at three critical locations i.e., near the support, at quarter span and at midspan. Fire location at the mid-span and near support are the most critical location in terms of potential damage level (maximum bending moment and maximum shear force location). The quarter point under span can be investigated as an intermediate case. Designer can establish the worst-case scenario for a particular bridge by carrying out the CFD simulation for only the mentioned critical locations of fire load rather than developing several fire scenarios that can be computationally expensive.

The fire simulation is run until the temperatures reach steady-state. Fig. 2(b) the temperature contour plot at x=30m (around 20m from the North corner of the bridge), where the first girder can be seen to be engulfed in flames from the tanker and spilled fuel. The temperature profile along the span of the girder can be seen in Fig. 2(c). Fig. 3 shows the AST temperature profiles in the near field (location of girder which is engulfed in flames) and far-field (15m away from the fire location, where the girder is heated mainly by smoke). A significant decay of the temperature is represented by these temperature profiles when moving away from the location of the burning vehicle.

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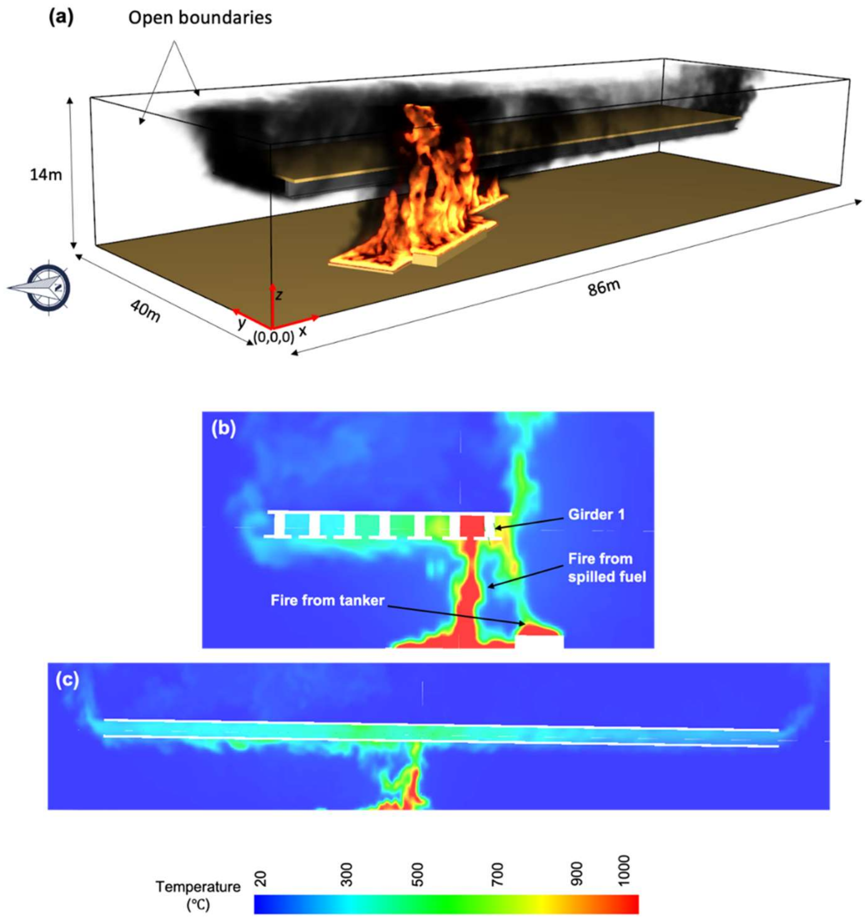

Figure 2: (a) Computational domain for CFD simulation (b) temperature contour plot at x=30m (c) temperature contour plot at y=15

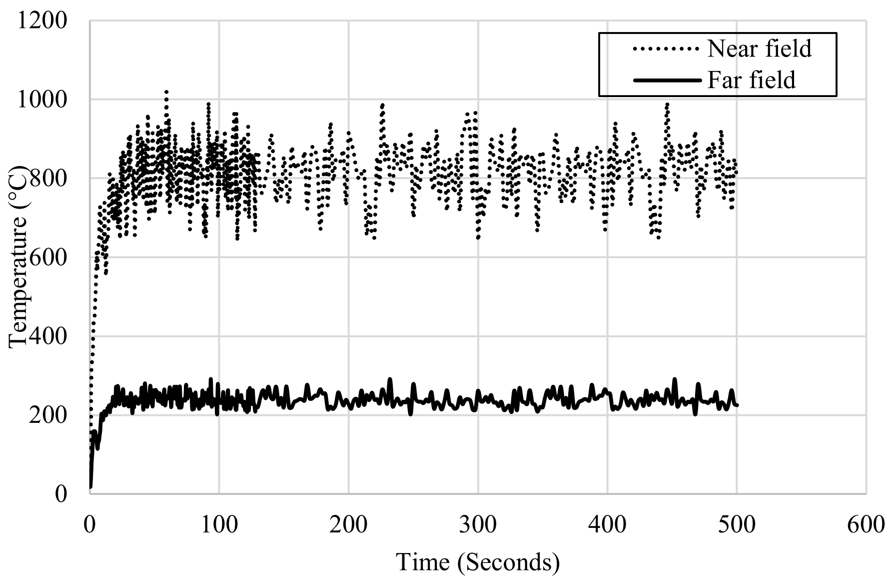

Figure 3: AST temperatures obtained from FDS at the near field and far-field

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2.3 CFD-FE coupling

To evaluate the structural fire resistance for PBD solutions or carrying out structural analysis of any structure, three models need to be defined (a) fire model (fluid domain), which can be obtained from a CFD simulation or experimental data; (b) heat transfer model (solid domain) to obtain the thermal history of relevant structural components; and finally (c) thermomechanical model (solid domain) to determine the structural response in terms of deformations and failures. However, due to enormous differences in the relevant length and time scales associated with fire and structural models, it is a challenging task to couple a CFD model with FEM for simultaneous or coupled simulations. For example, the characteristic time for a fluid domain is much lower than a solid model, and the mesh size in a CFD model could be even larger than the cross-section of structural elements. Performing CFD simulations with such a fine mesh would be computationally very expensive and practically not feasible. In the past, many researchers used various approaches to couple CFD packages (including FDS) with different FE software [29,30]. Furthermore, the format of the output files from CFD would be different from most of the FEM tools. In this paper, to map the data from FDS to Abaqus, a middleware is developed, which is capable of identifying the location of the structural boundary (specific elements) where specific temperature history needs to be applied. Therefore, there is no need to coincide the grids of CFD and FEM. The middleware can also transform the FDS output files in the format required by the Abaqus to carry out the heat transfer analysis. Once the heat transfer analysis is finished, the thermal gradients inside the solid can be used for performing sequential thermo-mechanical analysis. The workflow of the whole process of CFD-FEM coupling is shown in Fig. 4 and explained in detail in the following sub-section.

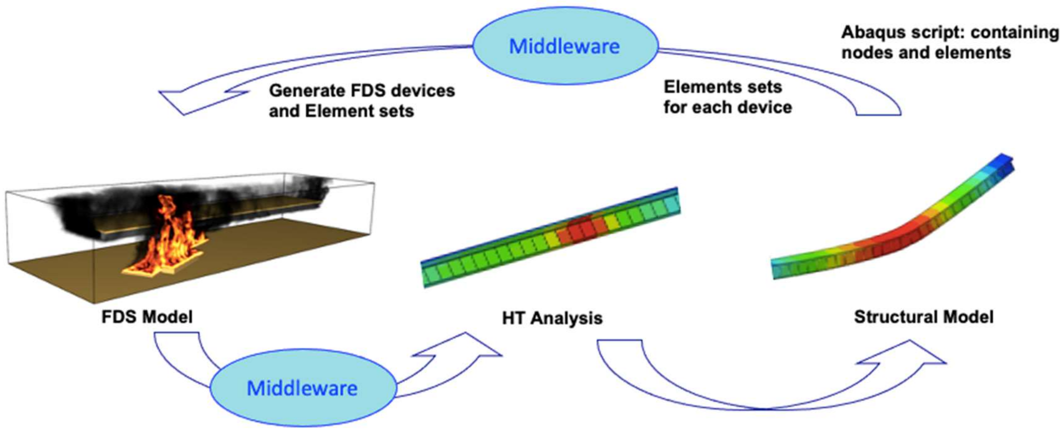

Figure 4: Sequential coupling of CFD and FEM

2.3.1 Middleware

To map the FDS data to Abaqus heat transfer model, a middleware named; Fire Structure Data Mapping (FSDM) is developed (executables of FSDM can be downloaded from [31]). The middleware can generate the time-varying temperature history that can be used in the FE model for conducting heat transfer analysis by copying all the device data to the Abaqus input file. Each device data is mapped to the FE model which is applied as thermal boundary conditions

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to a set of elements in the FE model which are generated by FSDM middleware corresponding to the device location in the FDS model.

Generally, it is not required to define structural components in FDS, however, measuring devices are needed to be placed at the required location where data needs to be recorded from FDS to use as boundary conditions for the thermal and structural analysis in Abaqus or any other FE software. The link between the fire model and structural model is the device (sensor) location. A module of the middleware can write a part of the FDS script for the device locations (AST, HTC, HF) based on the structural geometry. GUI of the module is shown In Fig. 5. The middleware is written in Python programming language, and all source codes are freely available on the authors' GitHub page [32].

The output data from the FDS is to be used as an input for the heat transfer (HT) and thermomechanical analysis. This output data is applied as the thermal loads to the structural elements, therefore, it is required to identify the correct elements for the application of specific time-temperature history recorded by sensors. While providing the sensor's location, the middleware searches the nodes and elements within the defined range where single temperature data needs to be applied. It is worth noting as the computational domain size is different for both FDS and FEM models, however, it is necessary to keep the global coordinates the same for both models, as the middleware searches the element sets based on the device locations. The middleware uses nodes and elements data of the structural model to find the elements, as shown in Figure 5. The module generates a part of the script file for heat transfer analysis in the Abaqus by providing the corresponding elements sets associated with the specific device in FDS.

FDS is written in Fortran [11] so the input script file must be written in such format. Generally, in all FEM tools, the script files are written in an exclusive format. To perform the heat transfer analysis using the data obtained from CFD simulations, it is required to convert the data in a specific format such that the FE software can read (*.inp in case of Abaqus). A pre-processing work needs to be done to covert the output which can be suitable to perform HT analysis in FE tool. Another module of the middleware (GUI is shown in Fig. 6) combines the temperature data of all devices in the format required for heat transfer analysis in Abaqus. Now, the output files (containing the time-temperature history for each element sets) would be applied to the structural elements present at the same location as the device in the FDS model. In this way, accurate mapping of the data from FDS to Abaqus for conducting heat transfer analysis can be performed with minimal chances of error.

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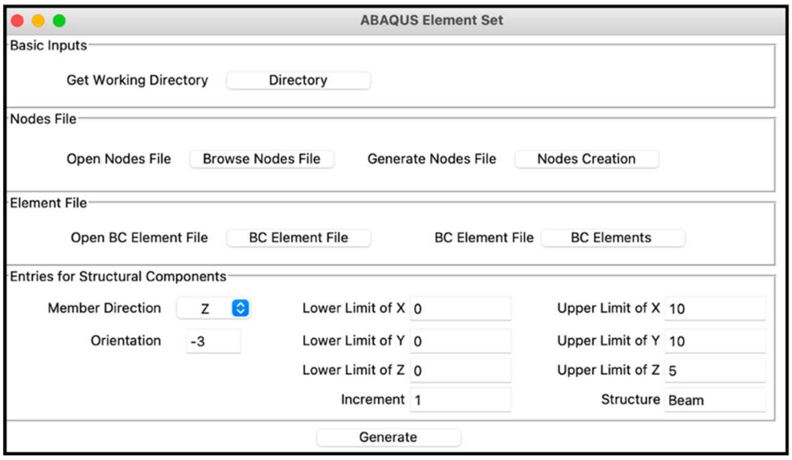

Figure 5: GUI of the module used to generate the FDS devices and element sets

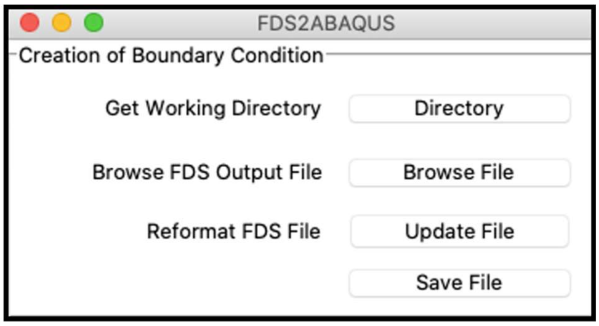

Figure 6: GUI of the module used to transfer data from FDS to Abaqus

The steps involved in the FDS-FEM coupling for producing a realistic fire scenario and conducting a sequential thermo-mechanical analysis is represented using the flowchart as shown in Fig. 7.

Following are the various steps that are performed in this open-source framework to conduct an FDS-FE coupled analysis.

1. In the first step, this module generates devices (AST) and writes in the FDS script file. FSDM simultaneously searches the elements in the heat transfer model corresponding to each device location in the FDS model. Element sets are created in the Abaqus input file for the application of each amplitude as the thermal boundary condition.

2. Fire simulation is carried out.

3. The output of FDS is post-processed in the form of amplitudes as required by Abaqus to create thermal boundary conditions.

4. Convective and radiative boundary conditions are created by applying all the amplitudes to the corresponding element sets, and heat transfer analysis is conducted (see Fig. 6).

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5. Once the heat transfer analysis is complete, the output file (*.*odb*) created by Abaqus is used for conducting sequential thermomechanical analysis.

6. Finally, the thermomechanical analysis is conducted to understand the response of the bridge structure exposed to a realistic fire scenario as shown in Fig 7.

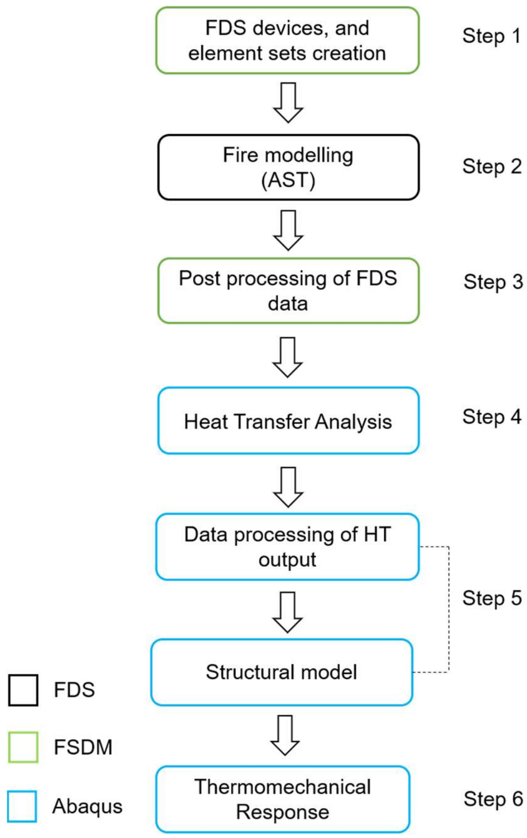

Figure 7: Step to carry out integrated simulation of 'structures in fire' in Abaqus

2.4 Fire resistance estimation

After establishing a realistic fire scenario using FDS modelling, a FE model of the bridge is developed for conducting heat transfer analysis. The central span of 37m long, which was mainly exposed to the fire comprised of steel girder and reinforced concrete slab, is modelled using a FE software; Abaqus. The steel girder (built-up section) was made of steel with a yield strength of 350 N/mm^2. A concrete slab of 170 mm thick was connected with the steel girder using shear studs and had a compressive strength of 40 N/mm^2 [2,11]. The flange was made of a 457 mm wide and 28 mm thick plate, and the web had dimensions of 1344 mm depth and 12 mm thickness. The central girder was stiffened with a total of 34 stiffeners on each side of the web, and four of them were present at the girder supports. The thickness of the support stiffeners was 25.4 mm and the rest of the stiffeners were 11 mm thick [2,11]. DC3D8 heat transfer elements available in the Abaqus element library are used to model the steel girder and concrete slab. The reinforcement in the slab is modelled with using the stringer feature. The tie constraint

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available in Abaqus is used to model the connection between the concrete slab and steel girder. Thermal properties of steel and concrete, such as conductivity and specific heat have been assigned as per Eurocodes [33]. The AST obtained from FDS simulations are applied at different locations on a steel girder and bottom of the slab as thermal boundary conditions. Heat transfer from the gas phase to the structural elements was modelled by applying appropriate convection and radiation boundary conditions. A convection coefficient of 25 W/m^2 °C for exposed surfaces and 9 W/m^2 °C for other ambient exposed surfaces and emissivity of 0.7 were considered [25]. The variation of temperature profiles across sections above the location of the vehicle due to realistic fire exposure obtained after conducting heat transfer analysis is shown in Fig 8. It can be seen that web temperatures are the highest being the thinnest part of the steel girder, and the temperatures of the top flange are significantly lower compared to the bottom flange because of its direct contact with the slab. While the temperatures at the bottom surface of the slab are similar to the top flange as both are in direct contact with each other. However, the temperatures at the mid-depth of the slab and the top surface are significantly lower, resulting in a steep thermal gradient which is attributed to low conductivity and high specific heat of the concrete material.

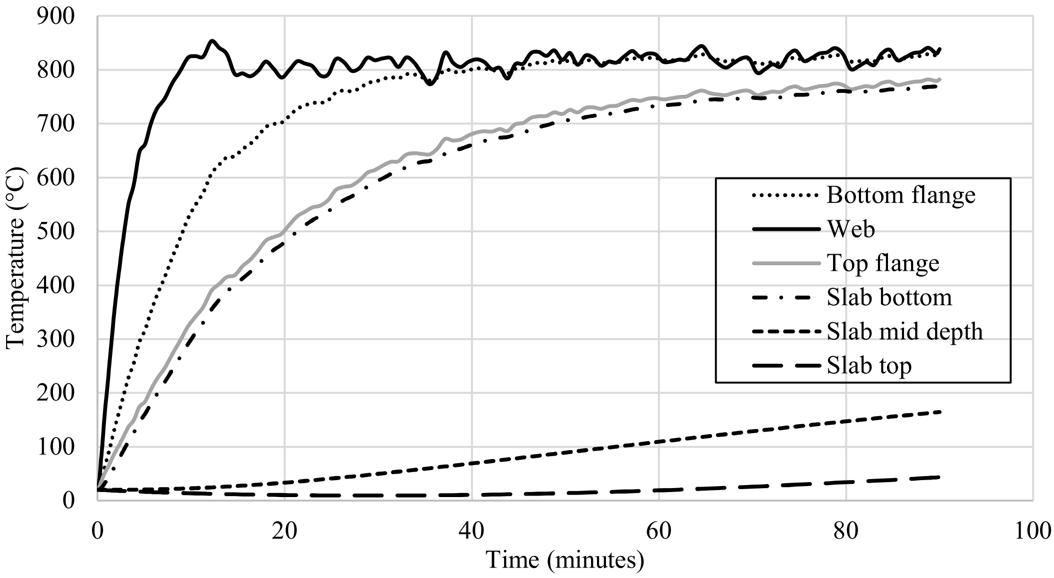

Figure 8 Temperatures at the section above vehicle location with fire intensity of 1000kW

Fig. 9 shows the web temperatures along the entire span length of the girder for a fire intensity of 1000 kW and 1500 kW. It can be seen that in both fire intensities, the temperatures are the highest at the location above the vehicle, and decay in temperatures can be observed at away locations. It is a true representation of a localised fire scenario resulting from a vehicle fire incident. While considering a uniform fire exposure such as HC fire would not be suitable and highly conservative as the entire span is assumed at a uniform temperature.

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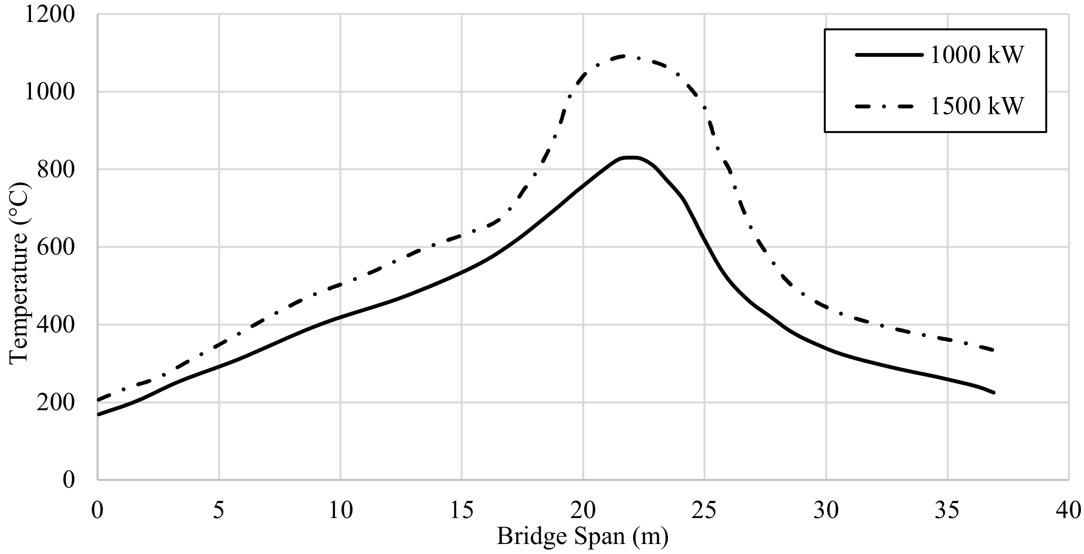

Figure 9 Temperatures profiles along the span with different fire intensities

Following the heat transfer analysis, a sequential thermomechanical analysis is also conducted to estimate the fire resistance of the composite bridge. In thermomechanical analysis, the temperature history obtained from heat transfer analysis is applied at different locations of the bridge as thermal loading. The thermomechanical model of the bridge is also generated using Abaqus to utilise the seamless coupling of heat transfer and thermomechanical models. The mechanical properties for both concrete and steel at elevated temperatures are used as per Eurocode recommendations [34,35]. Gravity loads due to the self-weight of the steel girder and the concrete slab are applied as automatically calculated by the software. The total dead load of 204.8kN due to the wearing surface of the deck and safety barriers has also been considered [13]. During the event of fire, not all the studies considered live loads on bridge structures, which is a reasonable assumption because the massive black smoke provides a stop signal to the oncoming vehicles. It is observed that the amount of live load does not have a strong influence in the collapse event [7]. Therefore, in this study no live load has been considered.

Fig. 10 shows the maximum deflection of the span for both fire intensities. The maximum vertical deflection is observed above the location of the vehicle when the bridge is exposed to the realistic localised fire scenario developed by FDS modelling. It can be seen from Fig. 10 that the rate of increase of deflection during the initial phase of the fire is very high despite having simply supported boundary conditions, this is due to the high thermal gradient within the composite bridge section. In the case of 1000 kW fire, after 12 minutes of fire exposure, maximum web temperature reaches 800 °C while the slab's temperatures are significantly lower with 400 °C at the bottom surface and 30 °C at the mid-depth, as shown in Fig. 8. As the overall slab temperature is quite low compared to the steel girder, the strength reduction of the steel girder is significantly higher compared to the concrete deck. The sudden increase of deflection after 12 minutes of exposure followed by the failure of the bridge is mainly contributed to the loss of strength of the steel girder as temperatures of the steel girder are comparatively higher than the concrete slab. Therefore, if the steel girder is provided with some level of fire protection, it can significantly improve the fire performance of the bridge. The bridge's failure

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occurred after 12 minutes of fire exposure, as shown in Fig. 10, which was also confirmed by the literature [13], and the deflected shape of the bridge obtained with 1000 kW fire exposure is also compared to the observed after the bridge accident as shown in Figure 11. Moya et al. [13] also conducted a numerical study by developing realistic fire scenarios using CFD simulations to evaluate the structural performance of the I-65 Birmingham bridge, and they also achieved a failure time of 12 minutes. Therefore, developing a realistic fire scenario using CFD simulations can lead to a reasonable estimation of the fire resistance of bridge structures.

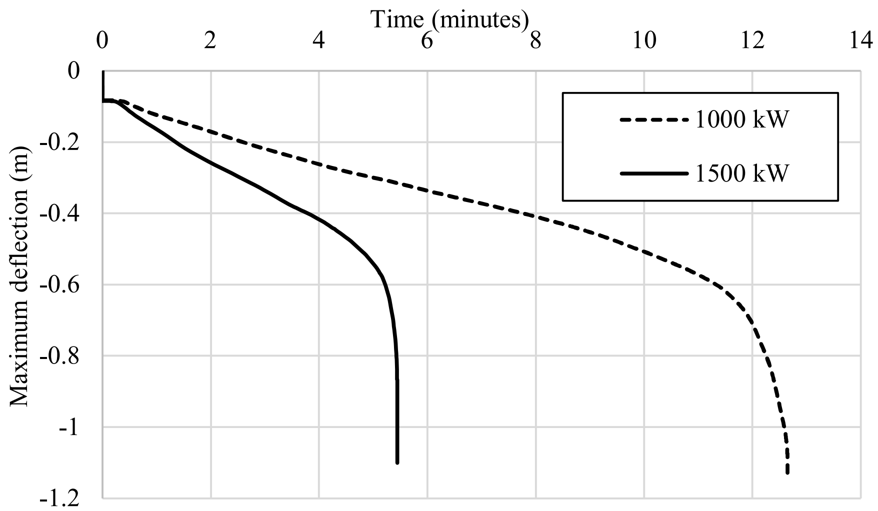

Figure 10 Maximum span deflection-time behaviour with different fire intensities

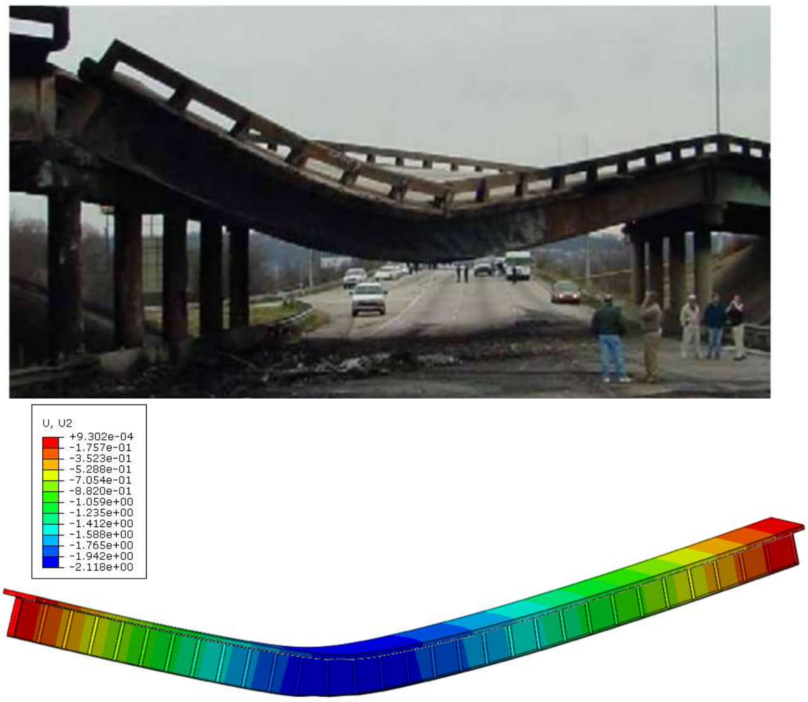

Figure 11 Deformed shape of the bridge in the real fire accident and FE simulation (blue colour representing maximum deflection)

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2.5 Enhancing fire resistance

In order to achieve an improved structural performance of the bridge under fire conditions, the right amount of fire protection must be provided. In this section, the heat transfer analysis of the bridge is conducted with various thicknesses of fire protection, i.e., 5mm, 8mm, 10mm, 12 mm, 15mm, 18 mm, and 20mm. The corresponding temperature histories are obtained to further analyse the structural performance of the bridge. CAFCO 300 is used as the fire protection material as used by various researchers [36], and temperature-dependent thermal properties of CAFCO 300 such as conductivity and specific heat are assumed to follow the recommendations of Bentz and Prasad [37]. In the heat transfer model, only the steel girder is provided with fire protection, and realistic fire scenarios developed by conducting CFD simulations for HRR of 1000 kW and 1500 kW are considered. Fig. 12 shows the temperature history of the composite bridge at a section above the vehicle’s location when it is exposed to a fire of intensity 1000 kW and fire protection of 15 mm is considered.

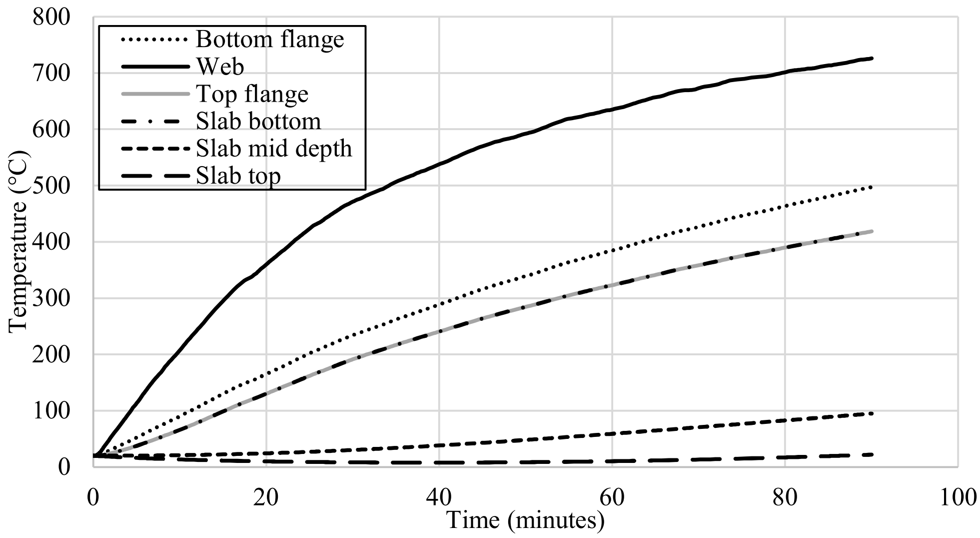

Figure 12 Temperatures of the protected section above vehicle location with fire intensity of 1000kW

It is observed that by applying fire protection of 15mm to the steel girder, the structural temperature of the girder reduces significantly. In the protected section, the web temperature reaches 700 °C after an exposure of 80 minutes compared to 6 minutes in the unprotected section. The temperatures at other locations, such as the top flange, are also greatly reduced from 768 °C in the unprotected section to 418 °C with 15mm fire protection after a fire exposure of 90 minutes (see Fig. 9 and 12). These reduced temperatures at all locations in the composite bridge can result in improved structural performance. In the next step, the thermomechanical analysis of the composite bridge is performed by considering the different amounts of fire protection to steel girder. It can be seen from Fig. 13 that with an increase in the thickness of fire protection, the rate of increase of span deflection reduces during the initial stage of fire and the failure time of the bridge is also delayed with an increased amount of fire protection. When an insulation of 15 mm is applied, the bridge managed to survive the fire for 90 minutes and did not collapse, as shown in Fig. 13. Whereas a failure of bridge is observed with a lower level of fire protection. Similarly, the structural performance of the composite bridge is analysed with different levels of fire protection when it is exposed to a fire scenario developed using an HRR

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of 1500 kW. In this case, insulation of 15 mm increases the failure time from 5 minutes to 67 minutes. Whereas 15 mm of insulation is not sufficient to avoid a collapse of the bridge when exposed to a fire of 90 minutes duration, therefore, a higher level of fire protection is provided and the structural response is studied. It is observed that when the bridge is exposed to a higher intensity fire of 1500 mW and 90 minutes duration, fire protection of 20 mm is required to avoid the bridge from collapsing, as shown in Fig. 14. In this paper, CAFCO 300 is used as the fire protection material due to its well-researched thermal properties and recommendation in previous studies for bridge structures [4–6], which has resulted in different level of thicknesses (5, 8, 10 and 12mm) for different survival time. While this methodology is not limited to any particular type of fire protection, other fire protection materials such as intumescent coating which can also enhance the fire performance of bridge but with lesser thickness can also be used without affecting the applicability of the methodology. This study helps structural fire engineers to easily estimate the right amount of fire protection required to achieve the desired performance level for bridges exposed to realistic fire scenarios with different fire intensities.

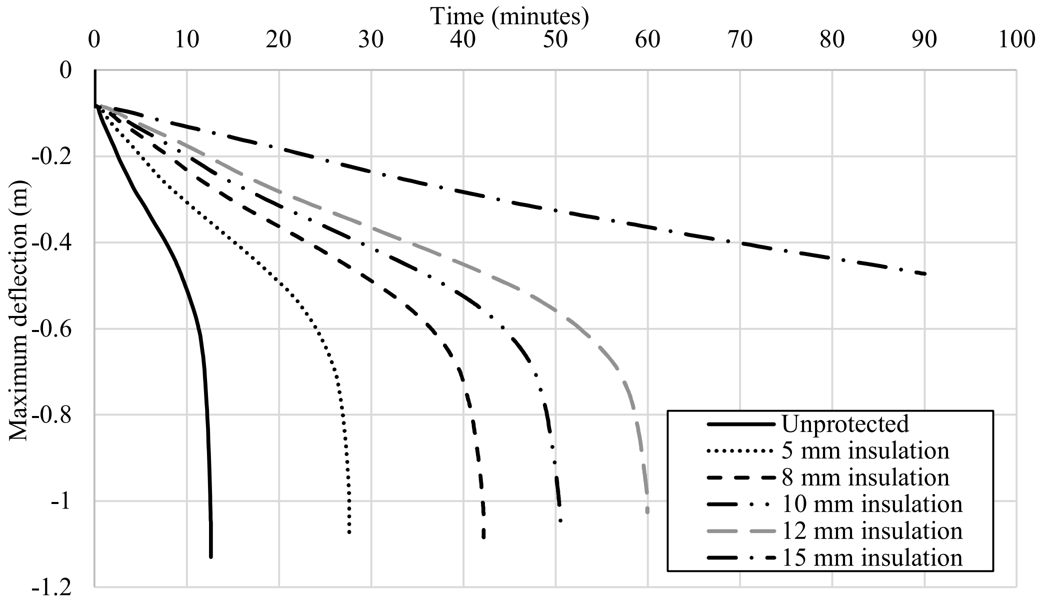

Figure 13 Maximum span deflection-time behaviour with different fire insulation and 1000 kW fire intensity

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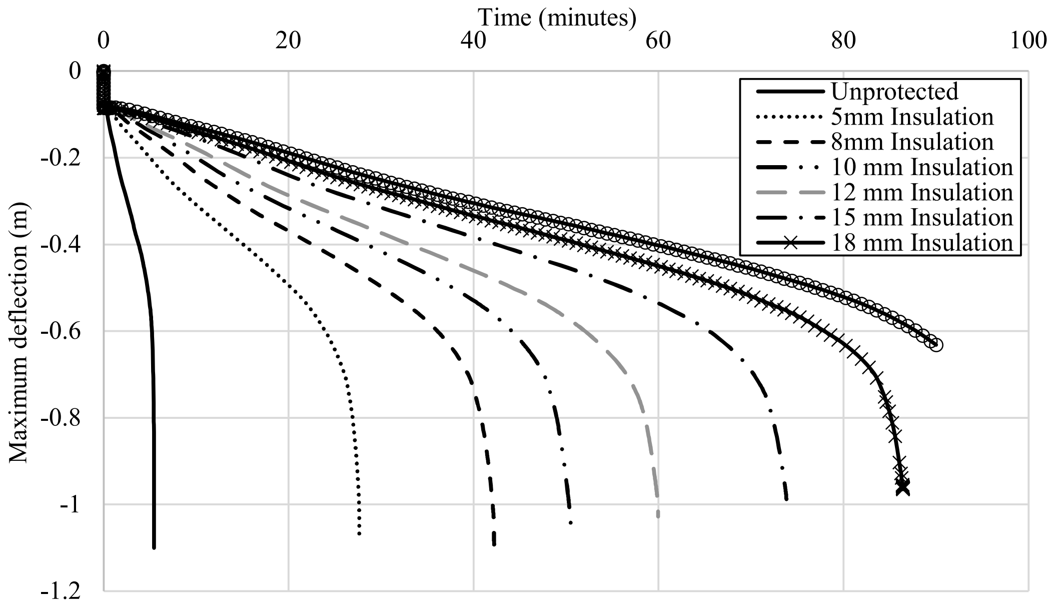

Figure 14 Maximum span deflection-time behaviour with different fire insulation and 1500 kW fire intensity

3 Conclusions

The following conclusions can be derived based on the results presented in this study:

1. Designing all bridges for fire hazard may be highly uneconomical, this study recommends fire risk assessment to identify bridges at high fire risk and design only them for a fire hazard.

2. The use of prescriptive fire scenarios such as HC fire to design bridges may result in a highly conservative design, this study presents a methodology to develop realistic fire scenarios for accurate estimation of fire loads.

3. The CFD-FE coupling approach developed in this study allows researchers to understand the thermomechanical behaviour of bridges and estimate their fire resistance under realistic fire conditions.

4. Bridge infrastructure exposed to vehicle fires can reach a very high temperature within the first few minutes of fire and can experience an early failure (10-15 minutes). This early failure can be delayed and completely avoided by providing a suitable amount of fire protection which can be calculated using the methodology presented in this study.

5. This study provides a useful methodology for the highway departments to minimise fire hazards for bridge infrastructure economically and effectively.

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