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# Impact of partially damaged passive protection on the fire response of bolted steel connections using finite element analysis

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A R T I C L E I N F O

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A B S T R A C T

This article aims to quantify the impact of a potential failure of passive fire protection on the ultimate response of a top and seat steel connection with double web angles. A numerical, finite element analysis scheme is proposed considering the real, semi-rigid behaviour of the connection, using unilateral contact-friction laws between the interfaces of the beam, the column, and the steel angles. The model has been validated by previous experimental research at ambient temperatures. Scenarios of unprotected connections, undamaged and partially damaged fire protections are numerically tested. A change in the failure mode and a reduction of the strength equal to 28% for standard fire and 35% for hydrocarbon fire arise for the model with the damaged protection. In this case, maximum temperatures locally at the beam reach the ones of the unprotected connection (900 °C), which is more than 800 °C higher than the connection with undamaged protection. Significant temperature increases of more than 288 °C and 406 °C for standard and hydrocarbon fires also arise on the top angle, compared to the model with undamaged fire protection.

## 1. Introduction

When exposed to fire, steel structures become increasingly vulnerable over time, leading to loss of strength and stiffness [1–4].

Passive fire protection is provided by gypsum and cement-based boards, sprays, encasements, and intumescent coatings. Fire insulation is achieved through their designed thickness and their intrinsic properties (e.g., low thermal conductivity, heat-absorbing reactions).

Several parameters may lead to failure of passive fire protection, decreasing the capacity of the structural system to sustain elevated temperatures. Improper application of passive fire protection, incorrect thickness, thermal displacements, or damage due to loads preceding fire may create failure in the fire protection. Under mechanical actions in ambient conditions, the adhesion of fire insulation with steel members can fail, reducing the capacity of the protection to support fire actions [5]. In addition, post-earthquake fires could be very destructive for steel structures due to earthquake-induced damage, leading to fracture or loss of passive fire protection [6]. The performance of passive fire protection may also deteriorate due to weathering or corrosion of the protected structure [7]. Proper preparation of the substrate (cleaning, priming, etc.), a controlled application within environmental specifications, adequate treatment of edge features, and appropriate inspection and maintenance are some measures that would provide fire protection and additional

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weathering resistance.

Steel connections crucially influence the mechanical response of steel structures, and therefore, their capacity to resist fire loading is significant. In [8], the initial rotation stiffness of a bolted-welded hybrid composite connection in a steel frame rapidly decreases with increasing temperature. Several steel connection types were numerically and experimentally tested in fire conditions, including fin plate and web cleat connection, flexible endplate, flush endplate, and extended endplate [9]. Results depicted a significant improvement in the survivability of steel frames in elevated temperatures when an increase in the thickness of all components in a connection, including beam flange and web, column flange and web, and connection plates, was considered. In [10] is shown that the beam flange of a reduced beam section steel connection, which is directly exposed to fire loading, is susceptible to local bucking, critically influencing the response of the structure in elevated temperatures. More research efforts can be found in [11], highlighting the response of shear angle connections in restrained steel frames under fire conditions, and in [12], investigating the performance of bolted steel splice connections in fire.

From the literature review, a relatively small number of studies focus on the impact of damaged passive fire protection on the response of steel structures under fire. In [13–16], finite element models were developed to investigate the impact of damaged passive fire protection (due to delamination) on multi-storey steel frames and connections. Damage of fireproof coatings after cyclic loading before fire exposure and its impact on the fire response of steel beams was experimentally evaluated in [17]. In [18], it was shown that fire-protected multi-storey frames show a reduction of fire resistance, which is proportional to the damage of the insulation.

To the authors' best knowledge, no research articles can be found in the literature investigating the impact of partially damaged fire protection on beam-to-column connections with top, seat, and web steel angles, considering the actual, semi-rigid response of the

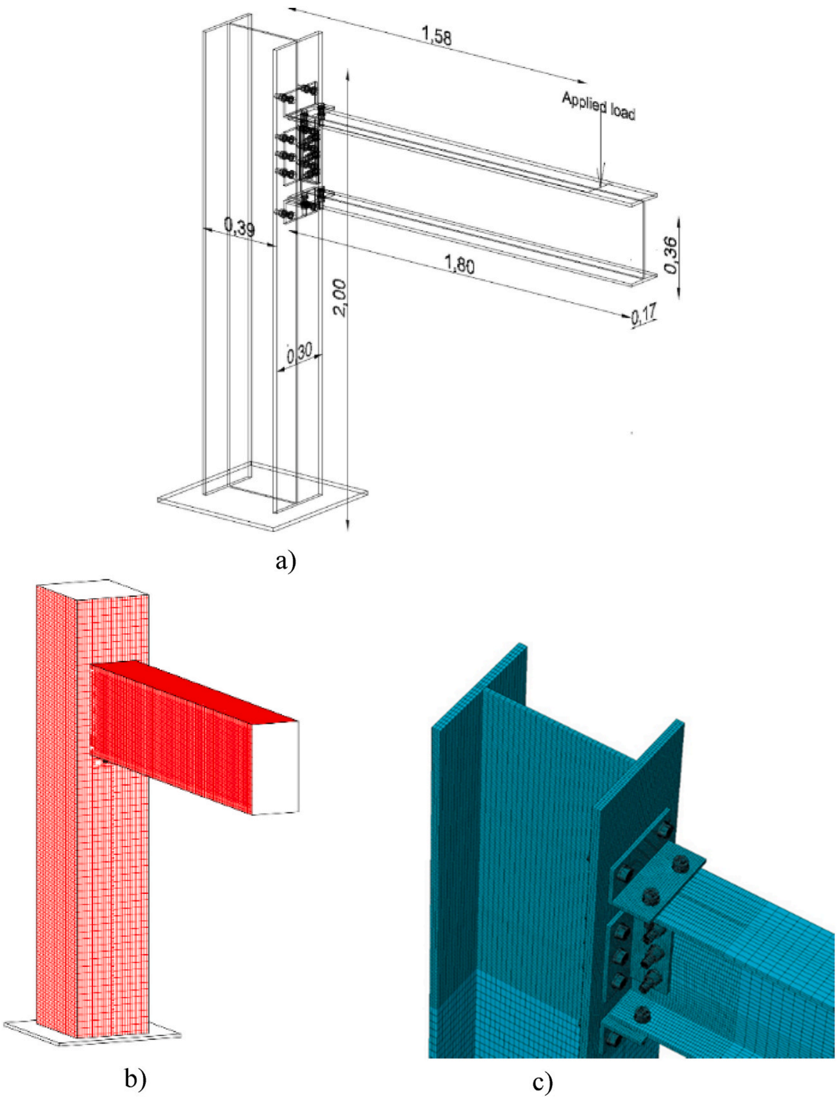



**Fig. 1.** a) Geometry (m) b) Passive fire protection c) Mesh of the finite element model.

connection. Thus, the present work aims to cover the gap and provide further insight into these investigations. Sequential transient-thermal and structural non-linear finite element analysis models of the fire-protected connection are developed in commercial software (Abaqus) to evaluate arising failure modes, ultimate moments, and durations to reach maximum temperatures.

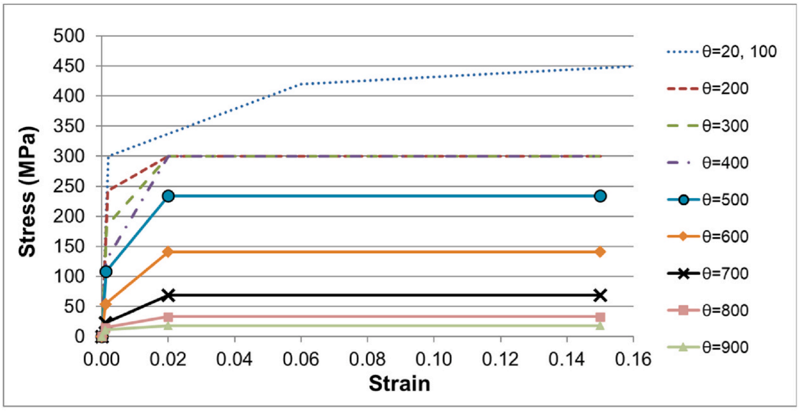
## 2. Aim and objectives of this research

A non-linear finite element model is developed in this article, aiming to evaluate the ultimate structural response of a top and seat angle connection with double web angles in fire conditions. Passive fire protection boards are attached to the perimeter of the connection. Thermal loading, in the form of temperature-time curves, is applied on these boards within transient-thermal finite element analysis, providing the temperature distribution on the steel connection. A static-structural analysis is then conducted in the steel connection, using as input the temperature distribution, which is derived from the transient-thermal analysis.

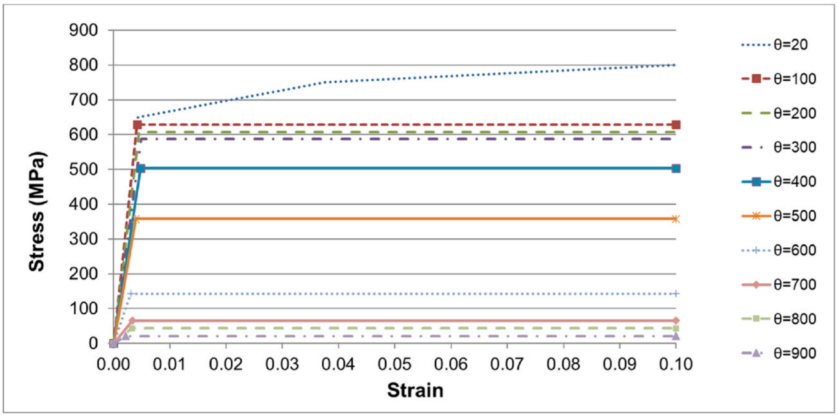
This work aims to quantify the effect of the potential damage of the fire protection, in the proximity of the column-to-beam joint on the beam, on the structure's capacity to support mechanical loads in fire conditions. Unilateral contact-friction interfaces are introduced between the angles and the steel sections to simulate the real, semi-rigid response of the connection. The strength of the connection is derived using moment-rotation diagrams, and the time is recorded, that maximum temperatures are developed in the connection.

## 3. The proposed fire protection numerical scheme

The investigated steel connection (Fig. 1) consists of a HEA 400 × 350 column and an IPE 360 × 170 beam section, connected with 4, L100 × 100 × 10 steel angles and 17, M20 high-strength bolts of grade-8.8 [19]. The thickness of the column flange is 19 mm, the column web is 11 mm, the beam flange is 12.7 mm, and the beam web is 8 mm. The thickness of each angle is equal to 10 mm, and the thickness of the base plate (bottom of the column) is 15 mm. Passive fire protection of a thickness of 30 mm in the form of gypsum




a)




b)

**Fig. 2.** Stress-strain curves at elevated temperatures [23] adopted for a) the steel parts, b) the bolts.

boards is introduced, as shown in Fig. 1b [20].

A transient-thermal model is initially developed, to derive the distribution of temperatures using a standard ISO834 or a hydrocarbon temperature-time fire curve loading [23], applied to all exterior surfaces of the protection (all-sided fire exposure, Fig. 1b).

In the interface between the protection and the steel a thermal-tie constraint law is considered, indicating that thermal energy passes from the protection, through this interface, to steel without restrictions.

The structural analysis follows, using the temperature distribution derived from the thermal analysis as input. The actual, semi-rigid behaviour of the connection is considered, using unilateral contact-friction laws in the interfaces between the connected steel parts, depicting potential opening/sliding. The response of the connection is then accurately determined since any opening/sliding crucially influences the strength and stiffness of the connection in fire conditions.

To confirm that the behaviour of the protected connection is not influenced by the relatively low mechanical contribution of the fire protection boards, these boards are not included in the structural model.

Loading of the structural model is a vertical point force applied to the beam (Fig. 1a). Both the mechanical and the thermal loads are simultaneously considered in the same load step of the structural model.

## 4. Thermo-mechanical material properties and thermal loading

Yield and ultimate stresses are 300 MPa and 450 MPa for steel, and 650 MPa and 800 MPa for bolts as determined from a previous experimental investigation at ambient temperatures [19,21,22]. Young's modulus and Poisson's ratio at ambient temperatures are taken equal to 200 GPa and 0.3, respectively, for both the steel parts and the bolts.

Concerning the thermal properties of steel, values taken from Eurocode 3 have been adopted [23]. To consider the degradation of the material properties of steel and bolts at elevated temperatures, a proper reduction of the stress-strain diagrams and the elasticity moduli, using reduction factors taken from Eurocode 3, are considered with temperature increase [23]. The stress-strain diagrams, which are obtained in this framework and used in the simulations for the steel parts and the bolts, are shown in Fig. 2.

For the gypsum boards, thermal properties derived from published experimental research have been used [24–27].

To simulate fire conditions, a heat transfer (thermal-transient) finite element analysis was considered for each model. Within this analysis, fire loading in the form of temperature-time curves [23] was applied in the external faces of the protection boards shown in Fig. 1b. Due to this thermal loading, elevated temperatures extend from the external faces to the thickness of the protection boards and gradually pass to structural steel. In the second step, a static-structural finite element simulation takes place, considering the

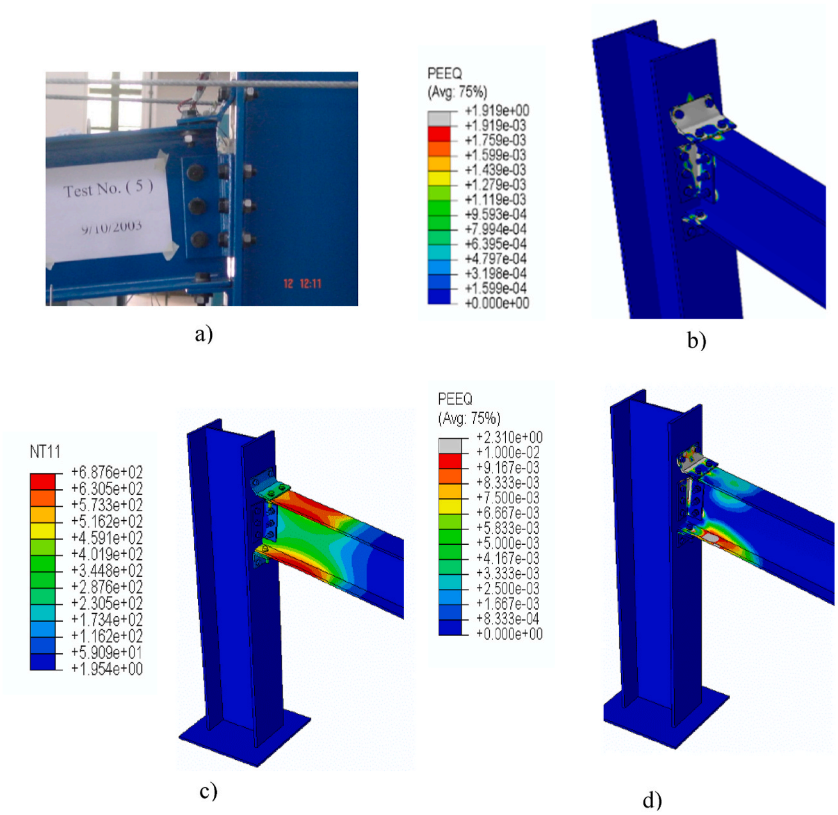



**Fig. 3.** a) Experimental specimen in ambient temperature [21,22] b) Plastic strain distribution on the model with undamaged fire protection c) Maximum temperature distribution and d) Plastic strain distribution on the model with partially damaged protection.

temperature distribution derived from the heat transfer simulation as input.

In the mentioned models, heat transfer from gypsum to steel occurs through conduction, radiation, and convection. The convection coefficient is 25 $W/m^2K$, and the Stefan Boltzmann radiation constant is $5.67 \times 10^{-8}$ $W/m^2K^4$. For the hydrocarbon fire, the convection coefficient is taken equal to 50 $W/m^2K$.

## 5. The finite element models

Three-dimensional, non-linear finite element models have been developed (Fig. 1c). A number of 167,785, 8-node brick elements are used to simulate the steel connection. To provide an accurate representation of the structural response in the critical area of the beam-column joint, a denser mesh is adopted in this part of the model, consisting of at least two elements along the thickness of each section (Fig. 1c). To avoid shear/volumetric locking, reduced integration elements were used.

A von Mises plasticity law is adopted to represent yielding on steel, and large displacement analysis is activated. For each bolt, washers attached to the bolt's head and nut were considered in the finite element model. Between the bolt shank and the surrounding hole of each steel part, unilateral contact interfaces without friction were introduced. The problem is strongly non-linear due to the material non-linearity for steel, geometric non-linearity, and the unilateral contact-friction interfaces connecting the steel parts. The Newton-Raphson procedure has been used in the analysis.

It is noted that for subsequent results derived by the finite element analysis, NT11 and PEEQ terms shown in relevant figures provide the temperature distribution and the plastic strain distribution for the considered models, respectively.

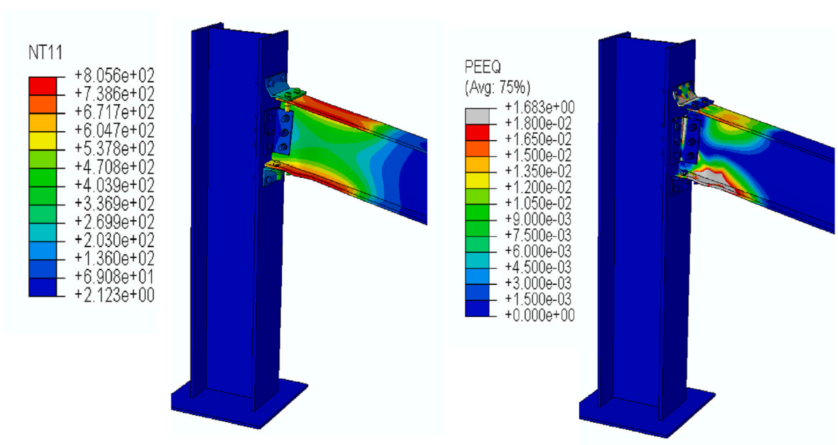
## 6. Results and discussions

### 6.1. Verification of the numerical model

The structural finite element model, which is used in this study to evaluate the structural response of the connection with fire protection at elevated temperatures, has been verified by [19] in ambient temperatures, using previous experimental research on the same connection (Fig. 3a). In particular, six specimens fabricated at the Jordan University of Science and Technology, were experimentally tested in ambient temperatures [21,22]. According to the main design criterion adopted in these tests, the column was selected to be stiff compared to the beam, and the magnitude of the flexural deformation of the column was expected to be minor, emphasizing in the response of the steel angles. Then, in [19] an identical steel connection was tested numerically in ambient temperatures using the finite element method. The same geometry, cross-section dimensions and material properties for the column, the beam the steel angles and the bolts were used. The semi-rigid response of the connection was simulated using unilateral contact-friction laws between the steel parts and the bolts. A vertical mechanical force was applied to the beam, and the failure response of the connection was investigated in ambient temperatures. The performance and accuracy of this finite element model were verified by comparison of the moment-rotation curves and collapse mechanism obtained numerically and provided by the experimental investigation on the same connection. Under mechanical loads, the connection fails due to the yielding of the top angle, followed by the yielding of the web angles.

### 6.2. Structural response at elevated temperatures

For undamaged fire protection, the maximum temperatures on steel at the ultimate load are below 100 °C, resulting in an identical failure mode to pure mechanical analysis (Fig. 3a and b). Thus, the connection's response is dominated by mechanical and not by




a) b)

**Fig. 4.** a) Temperature b) Plastic strain distribution for the model with hydrocarbon fire and partially damaged fire protection.

thermal actions.

Then, a damage scenario is adopted for a length of 0.5 m of the fire protection on the beam (for instance, due to an earthquake preceding fire [14]), close to the area of the maximum moment, in the proximity of the column-to-beam joint. To consider that the capacity of the damaged part of the protection to protect against fire diminishes, an increased thermal conductivity is assigned to this part of the protection in the thermal analysis.

Results indicate that yielding of the top/web angles and localized failure on the flanges and web of the beam arise (Fig. 3d). This constitutes a change in the failure mode, compared to the case of the undamaged protection, where yielding was restricted to the top and web angles (Fig. 3a and b). This is attributed to the fact that temperatures reach high values (more than 600 °C) at this part of the beam (Fig. 3c) after 38.9 min, indicating that thermal effects now dominate the structure's response. It is noted that, as shown in Fig. 3b and d, no fire protection is considered in the structural model, eliminating any minor influence of the mechanical response of the protection material on the structural behaviour of the connection.

When a more severe (hydrocarbon) fire load is considered [23], yielding on the top flange and the web of the beam is further expanded compared to standard fire load (Fig. 4b and 3d) and maximum temperatures reach 800 °C (Fig. 4a) after 33.7 min (5 min earlier than the case of standard fire loading). Like previous simulations, the fire protection boards have not been included in the static-structural analysis, as shown in Fig. 4b.

If no fire protection is applied, extended yielding on all steel parts and temperatures reaching 670 °C at failure arise for standard fire (Fig. 5) only after 9 min of fire duration (thus, 30 min earlier compared to partially damaged protection).

According to the moment-rotation diagrams shown in Fig. 6, the maximum moment that can be supported in (standard) fire conditions by the partially damaged protection is reduced by 28% compared to the connection with undamaged protection. For a more severe fire event, this reduction reaches 35%. For an unprotected connection, a drastic decrease of the maximum moment equal to 82% and 75% arises, compared to the connection with the undamaged and the partially damaged protection.

To provide further insight into the thermal response of the connection, temperature-time diagrams are given in Fig. 7, at nodes of the finite element model, on the top and web angles, on the beam and on the column of the connection. It is observed that for the beam, the temperatures on the model with fire protection approach the ones of the unprotected model at the end of the thermal simulation (120 min), as shown in Fig. 7c. This is attributed to the tested scenario of the damaged protection on the beam. For the remaining cases shown in Fig. 7, the maximum temperatures on the protected connection at the end of the thermal simulation are less than half, compared to temperatures of the unprotected connection.

## 7. Conclusions

The response of a fire-protected steel connection is evaluated in this article using non-linear finite element analysis. The aim of the work is to provide insight into the structural response of the connection with passive protection in fire conditions. The main fire scenario is that thermal loads are applied in the perimeter of the structure, which would be the case of an internal connection. Emphasis is given on the impact of partially damaged fire protection that may occur due to loads preceding fire (e.g., earthquake), poor application of the fire protection, or deterioration of the protection due to weather conditions on the ultimate structural response under fire. It is highlighted that the failure mode, that is yielding of the top and seat angles, changes and yielding is expanded on the beam for the models with the damaged fire protection. The ultimate moment is significantly reduced in this case, resulting in a 28% (standard fire) and 35% (hydrocarbon fire) reduction of the capacity of the connection to support mechanical and thermal loads compared to the model with undamaged protection. For the partially damaged fire protection, temperatures on the beam at the end of the fire simulation reach similar values to unprotected connection (900 °C and 990 °C for standard and hydrocarbon fire, respectively, as shown in Fig. 7c), indicating an increase of more than 800 °C, as compared to maximum temperatures on the beam for the undamaged protection (where maximum temperatures are below 100 °C). The corresponding temperature increase on the top angle is more than 288 °C and 406 °C for the standard and hydrocarbon fires (Fig. 7a), compared to undamaged protection. The following conclusions are also derived:

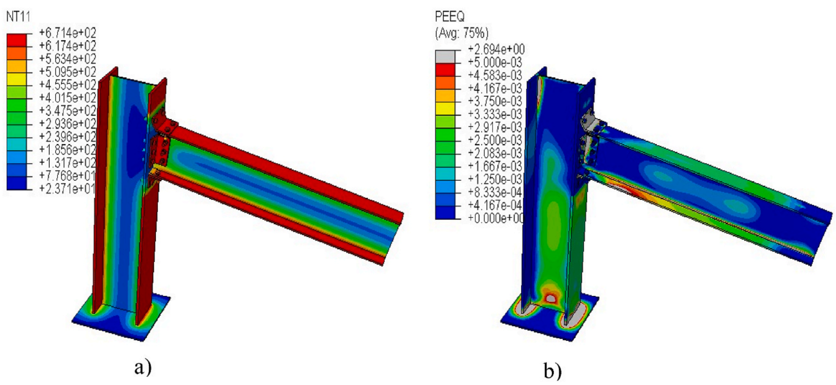



**Fig. 5.** a) Temperature b) Plastic strain distribution for the unprotected connection (standard fire).

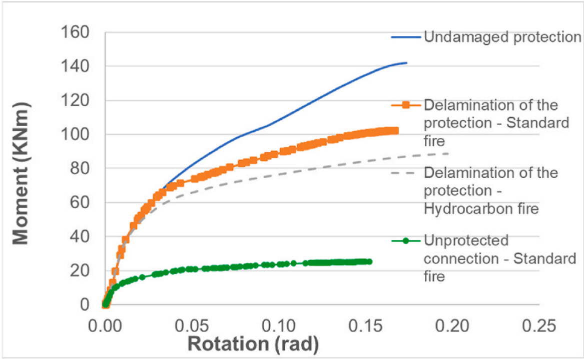



**Fig. 6.** Moment-rotation diagrams.

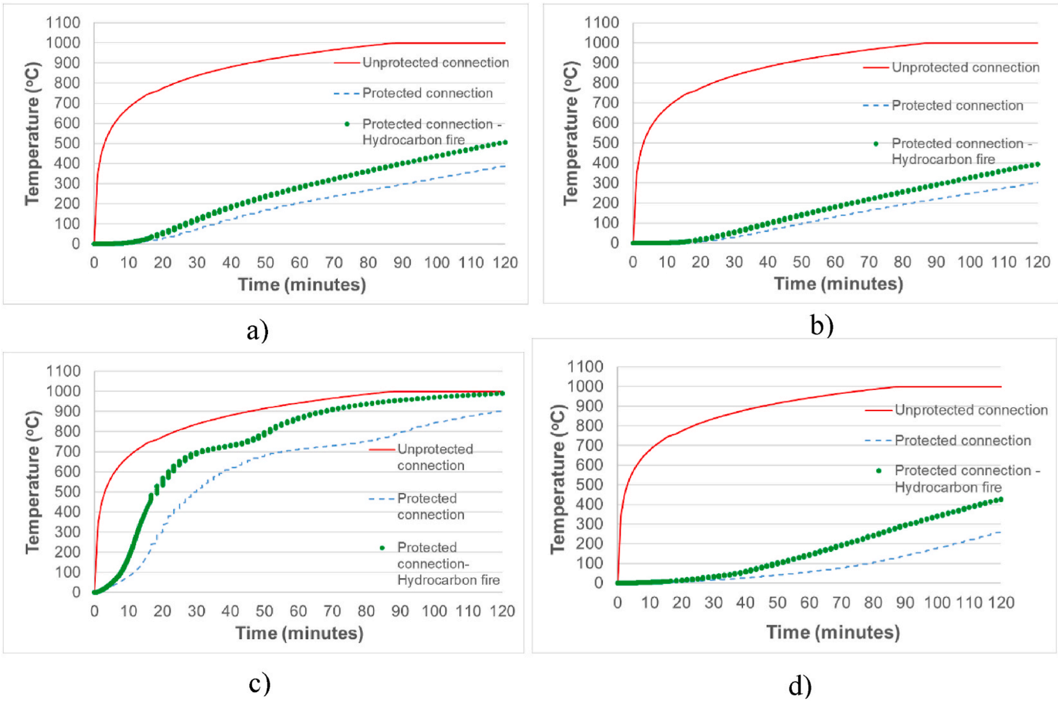



**Fig. 7.** Temperature-time diagrams for the unprotected connection as well as for the connection with fire protection in standard and hydrocarbon fire, in the a) top angle b) web angle c) beam, in the proximity of the beam-to-column joint d) column, in the proximity of the bottom steel angle.

- The protected connection under fire may reach the strength of the connection under pure mechanical load, provided that no damage to the protection occurs before the fire event.
- Potential damage to the fire protection leads to significant temperature increase, failure expansion, and a change of the failure mode, compared to the connection with the undamaged protection.
- The reductions of strength and time to reach maximum temperatures are recorded for the models with damaged protection.

Researchers and designers can apply the concept presented in this article to quantify or estimate potential losses in the capacity of connections to support loads in fire conditions when inspection of these structural systems reveals some damage in the passive fire protection.

It is noted that the concept which is presented in this article can also be applied to other types of connections. In a more general sense, this work can be extended within a holistic framework to more complex steel frames or buildings, evaluating the impact of potential damage of passive fire protection.

## Author statement

**Iksha Singh**: Data curation, Formal analysis, Investigation, Software. **Georgios E. Stavroulakis**: Methodology, Project administration, Resources. **Georgios A. Drosopoulos**: Conceptualization, Formal analysis, Investigation, Methodology, Supervision.

## Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

## Data availability

Data will be made available on request.

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