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Fire-induced Structural Failure Analysis of an Industrial Warehouse Roof Truss System

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Abstract

Fires are complex and hazardous phenomena governed by the laws of thermodynamics and fluid dynamics. Contrary to common belief, they are not rare occurrences, with approximately 130 buildings in Italy alone experiencing fires daily. The structural behavior under such extreme conditions is difficult to model as material mechanical properties are significantly influenced by temperatures, and thermal deformations alter structural responses.

This paper focuses on investigating the fire scenario and the failure of a reinforced concrete industrial warehouse through dynamic fire simulations and nonlinear thermo-mechanical analyses. The warehouse roof is constructed with truss arches featuring three-hinged joints with a steel chain. The arches are regularly spaced at 5.0 m intervals, with a clear span of 19.40 m and a measured height of 8.40 m at the peak and 5.0 m at the supports. The study is composed of two fundamental phases: simulation of fire conditions using Fire Dynamics Simulator (FDS) and investigation of the structural elements collapse through finite element structural analysis.

This analysis reveals that, due to thermal stresses, the chain in the roof truss can no longer perform its intended role within the arch, resulting in displacement at the ends and subsequent column collapse. Being a structure designed over 50 years ago, it lacks structural redundancy or robustness.

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1. Introduction

The likelihood of encountering fire events in daily scenarios is considerably high. In numerous global standards, fire is classified similarly to other extreme loading actions, including blasts and impacts. It is common for fires to be accompanied by explosions, which subject materials and structures to extreme conditions, as noted in Stochino 2016.

Various established models effectively explain the impact of temperature changes on the mechanical properties of Reinforced Concrete (RC), as detailed in Li 2004. Therefore, accurately reconstructing the temperature-time history along with the development of strain and stress fields is essential to evaluate the degradation of RC under fire exposure. It's also critical to accurately estimate real fire load scenarios, thereby enhancing the performance-based fire design methodology through the analysis of well-documented case studies that quantitatively measure temperature effects. Additionally, modern computational mechanics techniques provide robust tools for modeling highly complex issues. For instance, Computational Fluid Dynamics (CFD) involves the numerical analysis of fluid motion, based on numerically integrating the Navier-Stokes equations. These equations can only be solved analytically for simple cases, while typical scenarios feature turbulent flows and complex geometries that require numerical solutions.

After this brief introduction, this paper applies CFD and Finite Element (FE) analysis to a structure that suffered fire damage. The case study scenario is outlined in Section 2, CFD analysis in Section 3, and the FE thermo-mechanical analysis in Section 4, using the temperature time-history distribution as input data. Results of the analysis are presented in Section 5 while conclusive remarks are drawn in Section 6.

2. Case Study

The case study focuses on the structural behavior of a reinforced concrete industrial warehouse located in the outskirts of Cagliari (Italy) after a fire incident. The main structural support consists of reinforced concrete pillars, each measuring 30x40 cm. The roof is constructed with three-hinged arches, which eliminate thrust via a steel tie chain, supporting a secondary structure composed of simply supported purlins and curved corrugated metal panels.

The fire affected only a portion of the building, see Fig.1, leading to a partial collapse of the roofing. Consequently, the analysis was focused on the behavior of the arch that was most exposed to the fire's action.

Fig. 1. Warehouse after fire.

The roof is constructed with reticular three-hinged arches, where the thrust is counteracted by a steel chain. These arches are spaced at regular intervals of 5.0 meters, have a clear span of 19.40 meters, and at their highest point, they reach a height of 8.40 meters above the ground, which tapers to 5.0 meters at the supports. Each arch comprises the following elements:

- An upper chord measuring 17x15 cm, reinforced with Φ14 bars at the corners;
- A lower chord measuring 17x9 cm, reinforced with 2Φ8 bars on top and 2Φ10 bars on the bottom;
- Diagonal sections measuring 17x5.5 cm, reinforced with 2Φ8 bars in the middle;
- A steel chain with Φ28 diameter.

The structural behavior of the arch, see Fig.2, is dominated by the mechanical performance of the chain, which, when exposed to temperatures higher than the concrete, tends to deform. This deformation increases the arch's span and progressively increases the lateral load on the pillars. Hence, it is possible to approximate the modeling of the arch chords and diagonals. The chain steel's yield strength value of 540 N/mm² was determined with experimental tests developed at University of Cagliari Materials Strength Laboratory. Concrete strength class is assumed C20/25 see NTC18.

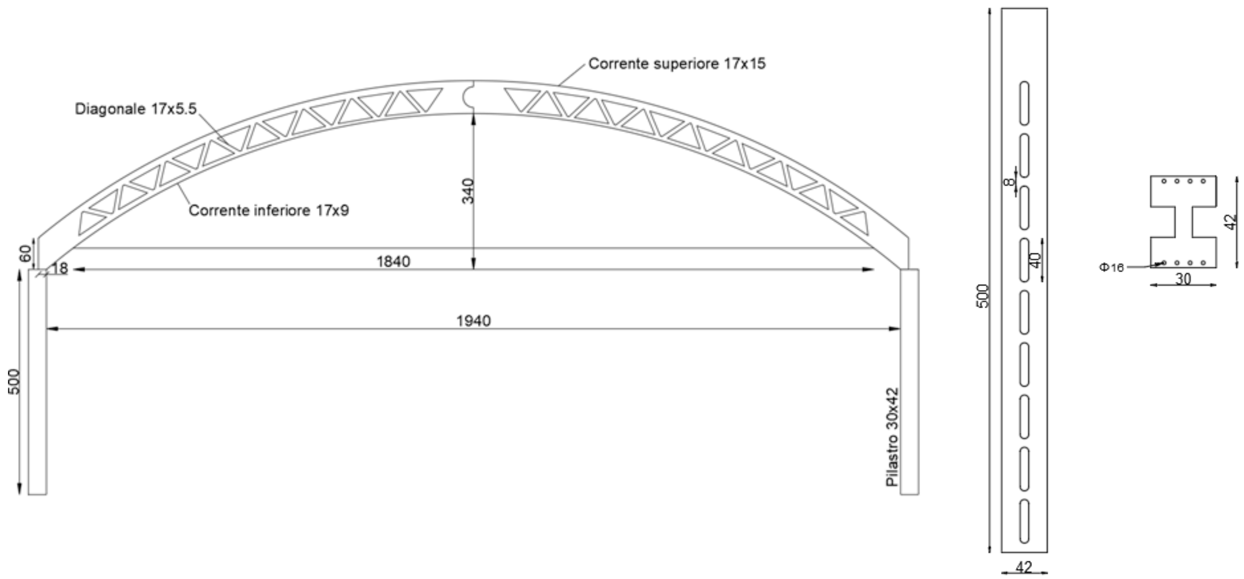


Fig. 2. Three hinged arch dimensions (left) and column (right) in cm.

The geometrical characteristics of columns are presented in Fig. 2, while concrete class is again C20/25 and steel reinforcements yielding strength is 375N/mm^2.

3. Fire Model

The fire-affected area is confined within four walls and the aforementioned roof, covering a surface of 19.4 x 15 m, with wall heights of 5 m and a maximum height at the farthest point of the arch of 8.40 m. For simplicity, a reference system (north, south, west, east) corresponding to the orientation of the walls is used. The North and South walls are made of brick, the West wall, undamaged by fire, is modeled as an adiabatic wall with a thickness of 0.3 m, and the East wall is a fire-resistant (REI) structure. The brick walls have various openings including windows and doors of different sizes. The REI wall consists of a double steel panel with a thickness of 2 mm, enclosing a layer of polyurethane about 30 cm thick. The warehouse contents and material types (mostly fabrics such as bed sheets or draperies) were determined by analyzing archival photographs, and due to the impossibility of exactly reproducing the placement, quantity, and composition of the materials at the time of the fire, objects measuring 1x1x1 m were used to represent the materials present on-site before the fire.

The fire originated from the shutter on the north wall due to a highly flammable substance like gasoline. Assuming the use of a 10-liter canister, it would cover an area of 4 square meters if we consider a thickness of 25 millimeters, see Figure 3.

The fire dynamics simulation was carried out using Fire Dynamics Simulator (FDS), as referenced in McGrattan 2013. This simulator employs a computational fluid dynamics (CFD) framework to address the Navier-Stokes equations, specifically tailored to simulate the movement of smoke and the transfer of heat in low-velocity, fire-induced flows. It employs second-order finite difference approximations to the differential equations governing mass, energy, and momentum conservation. These equations are then numerically resolved at each mesh point and at successive time intervals.

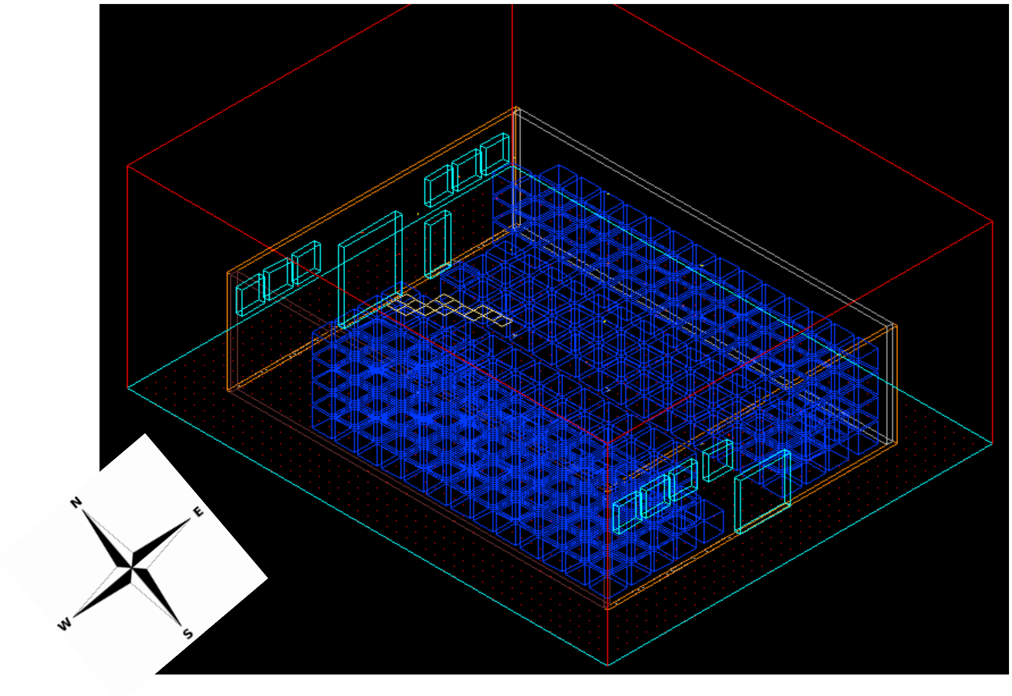


Fig. 3. Fire model geometry.

The study presented here utilizes the Large Eddy Simulation (LES) technique for its solution framework. This approach focuses on capturing the most significant turbulence scales that are influenced by the flow's geometry, while it assumes that the smaller scales are consistent across different flows and thus do not require customization for individual cases.

In the LES method, minute-scale events are not explicitly modeled; instead, the method opts for a streamlined algorithm. This is underpinned by the semi-empirical model conceived by Smagorinsky, as detailed in McGrattan 2013, which integrates the effects of large-scale turbulence directly into the calculation process.

4. Thermomechanical Model

The Finite Element (FE) thermal-mechanical model of the arch, was developed using the Strand7 software. A first thermal analysis has been followed by a mechanical analysis that exploits the temperature distribution obtained by the latter one. The chain has been modelled with 40 beam elements, while the concrete central hinge and the ends are represented by 144 plates elements. The chords and diagonals are modelled with 101 beams elements and have 1510 degrees of freedom. The constraint condition imposed by the column on the arch is modeled using two trolleys equipped with two springs that have translational stiffness along the horizontal axis. The stiffness of the springs is calculated considering the elastic theory of slender beam:

$$k = \frac{3EJ}{L^3} \tag{1}$$

where E is the modulus of elasticity, J is the moment of inertia, and L is the column length. This formula ensures that the mechanical interactions between the arch and its supports are accurately represented, allowing for a detailed analysis of the arch's response under thermal stress.

The thermal load has been applied using the temperature-time curves obtained through the CFD analysis.

The chain was divided into 40 nodes, with different temperature-time tables assigned every 8 nodes. For the lower part of the arch, which is approximately 21 meters long, the three different temperature-time tables were placed at nodes spaced every 7 meters. The calculation step used for this analysis does not affect the results; hence, the same step frequency as that used in the nonlinear structural analysis was employed, which is one step per second.

This approach facilitates a transient nonlinear structural analysis that accounts for temporal variations in temperature and the nonlinear material properties of the model, thereby allowing for an assessment of the arch's mechanical behavior under varying thermal conditions UNI EN 1992-1-2.

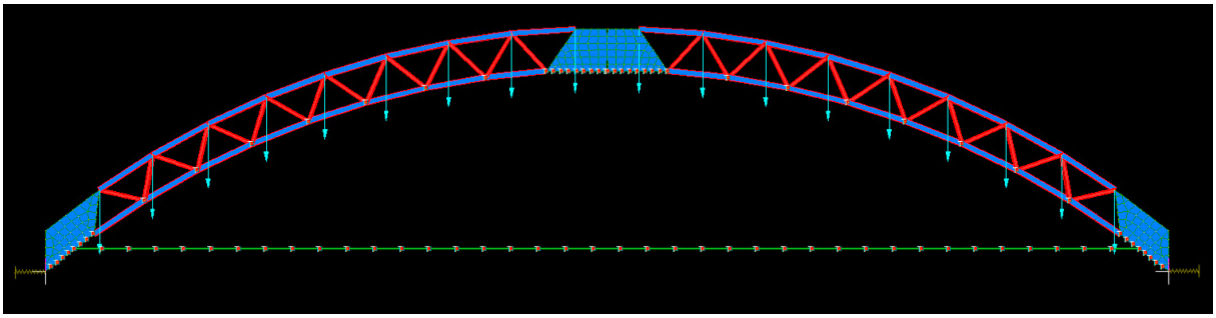


Fig. 4. Arch thermomechanical FE model.

5. Results

The simulation sequence is depicted through a series of images analyzed to understand the fire's behavior during the simulated period.

At 60 seconds (Figure 4), the fire was confined only to the initial ignition zone. Ten minutes into the simulation the fire appeared to remain in the ignition area, but it's important to note that the flames indicate regions with a heat release rate exceeding 40 kW/m³, suggesting that combustion reactions might be occurring in other areas, albeit to a lesser extent. By 1500 sec the flames had spread throughout the warehouse with significant heat release, and temperatures were high enough to trigger the removal of openings; windows and doors changed color from blue to white, indicating activation, except for windows on the north wall's left side, which did not reach the activation temperature and thus remained closed for the duration of the simulation. The absence of coverage in the simulation meant that these openings did not affect air recirculation; three windows stayed closed likely due to the absence of combustible materials within a 3.5 m radius. Further into the simulation, at 3000 seconds, a notable reduction in internal materials is observed as temperatures begin to decline. By 5000 seconds, the fire is nearing self-extinction, indicating the simulation's progression towards an end without external intervention.

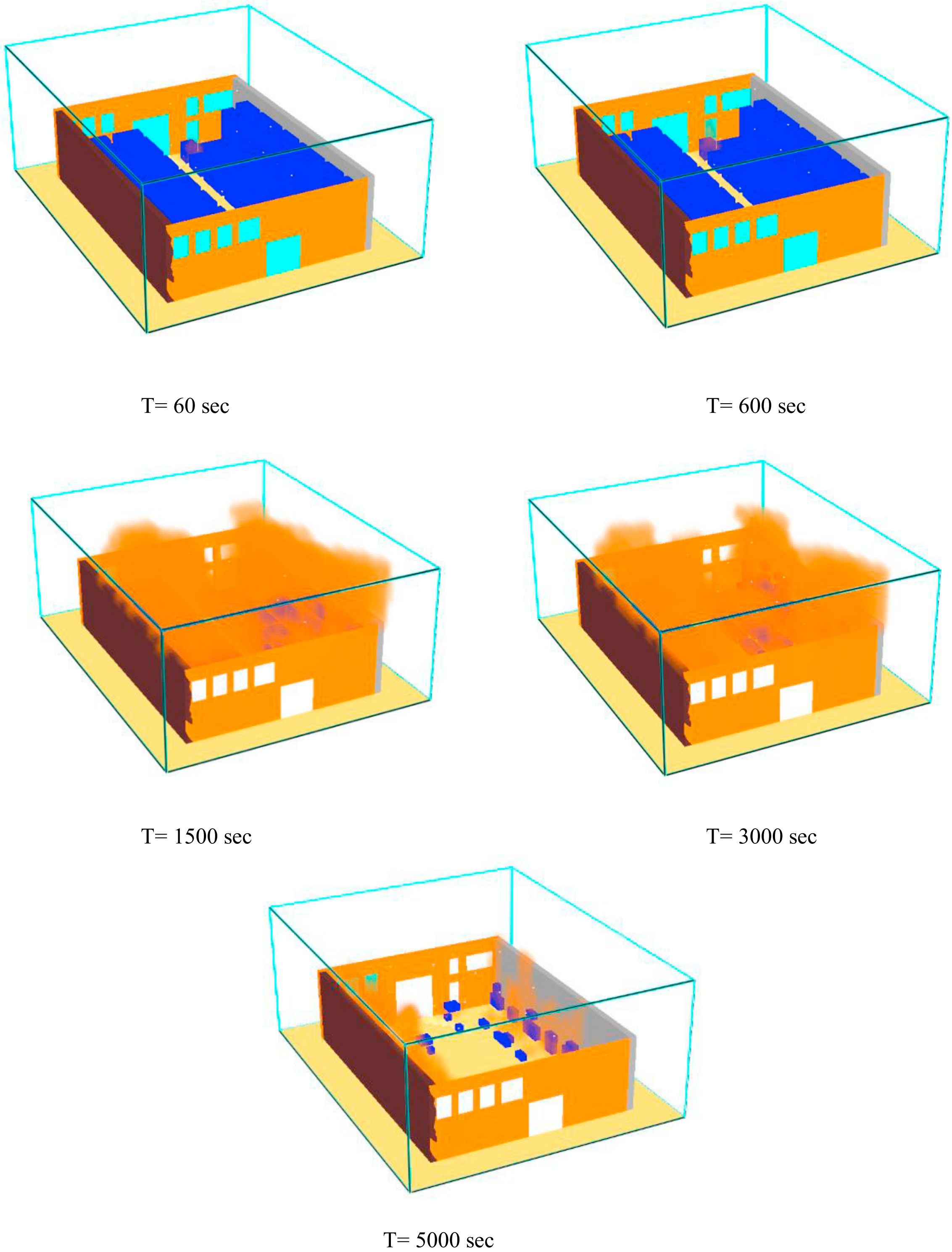
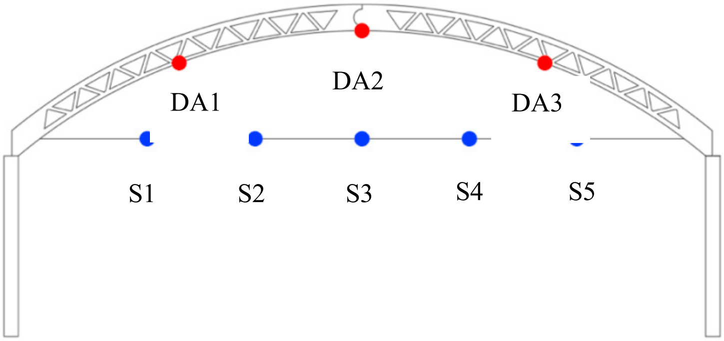
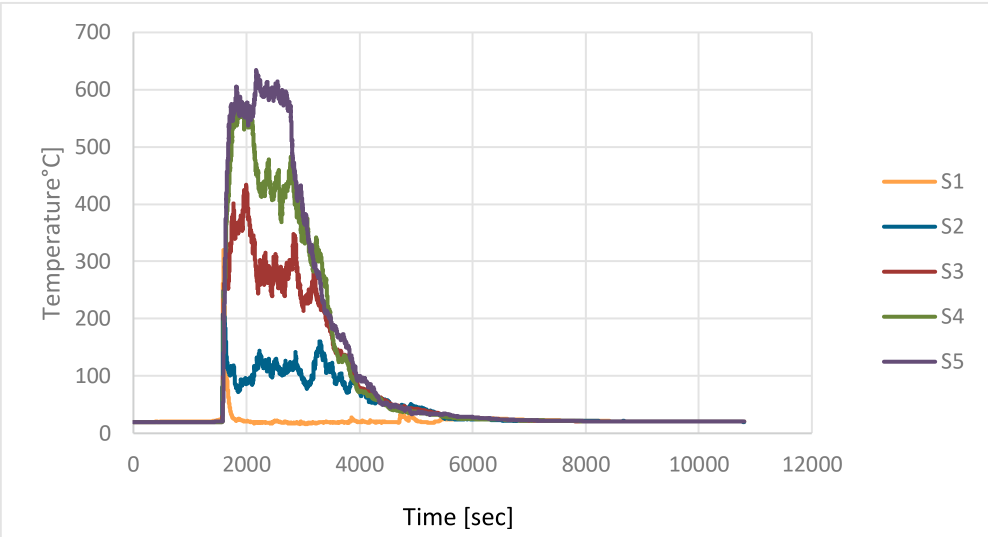


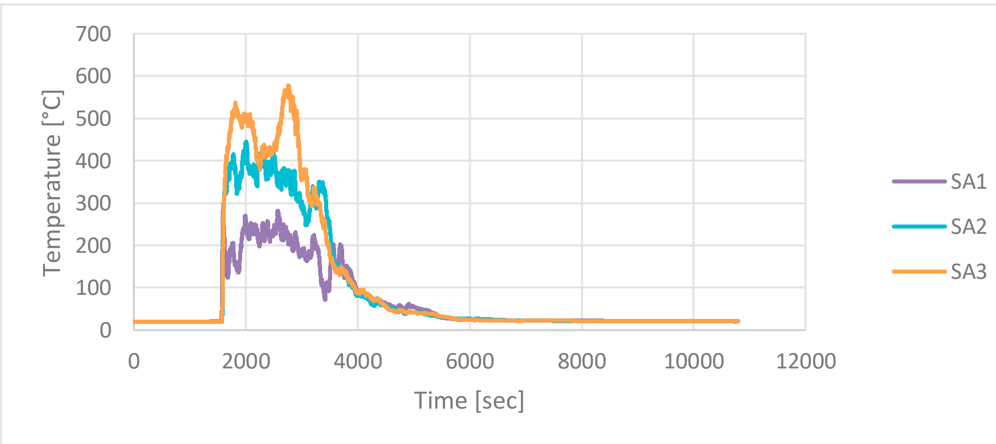
Fig. 5. Fire developments.



(a)



(b)



(c)

Fig. 6. Temperatures time histories on the steel chain (b), on the concrete arch (c) .

Figure 6 presents the time histories of the temperature obtained through the CFD analysis in 3 points belonging the concrete arch and 5 points on the steel chain. It is possible to note that concrete temperature goes beyond 550 °C while steel ones arrives at 600°C yielding to a significant degradation of the mechanical characteristics.

The chain, due to significant thermal stress and extensive expansion, underwent a displacement that induced the collapse of the pillar and the subsequent collapse of the entire roof.

Indeed it is possible to evaluate what is the displacement of the column top and consequently the corresponding bending moment Med using the elastic theory of beams:

$$Med = 2\frac{\nu EJ}{L^2} \text{ [27]} \tag{2}$$

Where ν is the horizontal displacement of column top, E is the modulus of elasticity, J is the moment of inertia , and L is the column length. Using equation (2) it is possible to compare this value with the resistant bending moment representing the capacity of the column as reported in Figure 7. In this way the collapse of the structure should have happened around 26 minutes from the fire ignition.

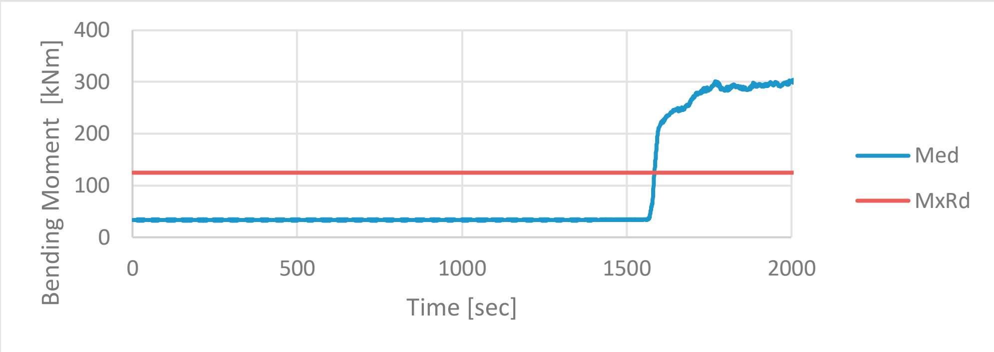


Fig. 7. Comparison between bending moment induced by fire load and resistant bending moment.

6. Conclusions

The aim of this paper was to analyze the effects of a fire on an industrial warehouse, specifically to identify the collapse mechanism considering the stresses caused by a CFD simulation of the real fire scenario.

The simulated time histories need 25 minutes to reach flashover, and while high temperatures are reached, they are maintained only for a limited time. From the analyses, it is evident that due to thermal stresses, the chain no longer performs its role within the arch, leading to end displacements capable of collapsing the columns. Given that the building was designed over 50 years ago, it lacks any structural redundancy (multiple means to perform a function) or robustness. The chain is the key element of the structure; its failure implies no alternative system could counteract the arch's thrust, leading to collapse.

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