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Fire-Induced Collapse of Automated Rack-Supported Warehouses

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Abstract

Automated self-supporting rack warehouses are becoming increasingly common in the international logistics sector. Their static and seismic design has been studied in the literature; however, the fire design still needs to be improved. Some Standards, such as the Eurocodes or Italian Fire Code, accept the collapse of rack warehouses during fires but require avoiding a progressive collapse or obtaining an implosive collapse regardless of any other active or passive fire protection systems. A hierarchy that guides the kinematics of collapse needs to be defined appropriately but without any Code going into further details. Moreover, the scientific literature focuses more on optimizing fire protection systems for racks, such as sprinklers, rather than preventing fire-induced progressive collapse. Here, a case study is presented, which, using finite element modelling, provide a first robustness evaluation and highlights some possible aspects to be considered in the structural design to avoid a progressive collapse in the event of a fire. The analyses are performed using the LOCAFI method for localized fires and nonlinear dynamic finite element simulations.

Keywords

Localized Fires; Progressive Collapse; Rack Warehouse; Steel Rack; Fire Design; Structural Robustness

1 Introduction

Logistics is a fast-growing sector requiring even bigger warehouses whose final aim is to stock significant quantities of goods and operate as a distribution center. In the last 20 years, the solution to stock this amount of goods moved from precast concrete warehouses with steel racks installed inside and cargo handling made with man-driven forklift trucks to automated rack-supported warehouses (ARSW, Figure 1). In this case, the rack structure is subject to external loads, such as wind and snow, and must comply with national regulations for ordinary buildings [1], [2] and not only specific codes for rack structures [3], [4]. ARSW are generally made with cold-formed sections: c-shaped or hollow square sections (HSS) for columns (called uprights) and c-shaped for bracings and beams. In the first case, significant nonlinearities are related to columns, which are perforated along the height and subjected to local and distortional buckling phenomena [5], [6], and to beam-to-column joints [7]–[9]; the latter can have a relative influence on the seismic behavior factor [10]. The seismic approach is generally *non-dissipative*, as the rules requested by EN1993 could be hard to implement [10], especially regarding the maximum difference of the over-strength factor between load levels [11].

Figure 1 - An ARSW during the construction phase

The fire design of ARSWs is generally based on active or passive measures, such as sprinklers [12], intumescent coatings, or oxygen reduction systems [13], to prevent or control the fire spreading since the design of these structures using prescriptive approaches based on fire resistance time could be demanding [14]. However, an evaluation of the robustness of ARSW is currently missing in

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the literature. It could be a starting point to define strategies to prevent or control the response of these structures, leading to more effective and economical design methods based on the performance-based approach. The structure of this article is as follows: Section 2 describes the numerical procedure developed and used to carry out finite element analyses, Section 3 describes the case study, Section 4 reports the results for every scenario, Section 5 discusses the results, and Section 6 adds some concluding remarks.

2 Numerical procedure

The numerical procedure followed to evaluate the structure's response can consider large displacements, high strain rate, and collapse and is based on the one proposed by Sun et al. [15]. However, some significant changes were made, developing a 3-step method: static, dynamic implicit, and dynamic explicit (Figure 2).

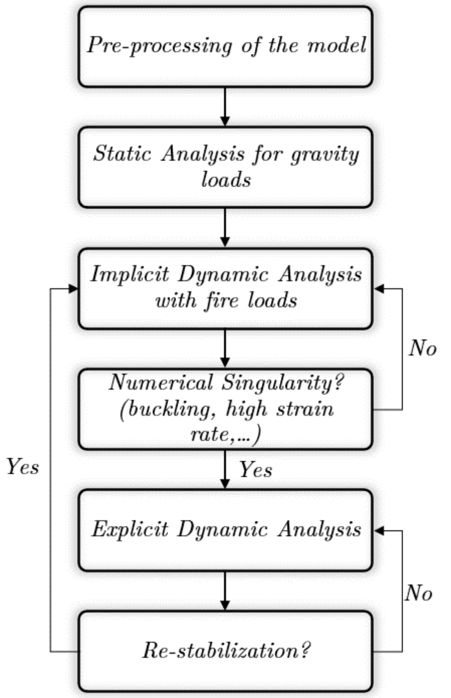

Figure 2 - Flowchart of the proposed analytical procedure

The main advantages of the proposed procedure are reported in [16]; it is worth noting that this procedure allows overcoming a posteriori estimation of collapse based on an implicit scheme as done in [14], [17].

3 Case study

3.1 Geometry and loads

The case study comprises a double-depth, automated storage and retrieval warehouse (AS/RS). The down-aisle section is reported in Figure 3, and the cross-aisle in Figure 4.

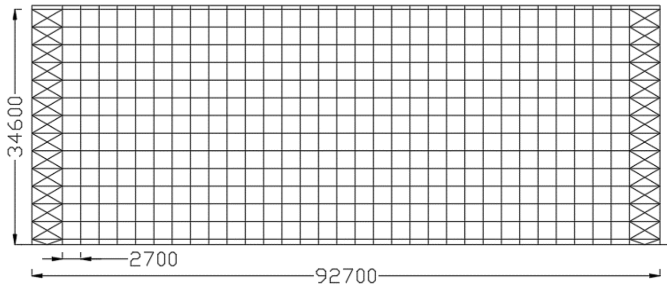

Figure 3 - Case study down-aisle section

Analysing Figures 3 and 4, it is worth noting how the roof truss is the unique element that connects all the columns in the cross-aisle direction; any force that needs to be redistributed after a column loss must pass through the truss.

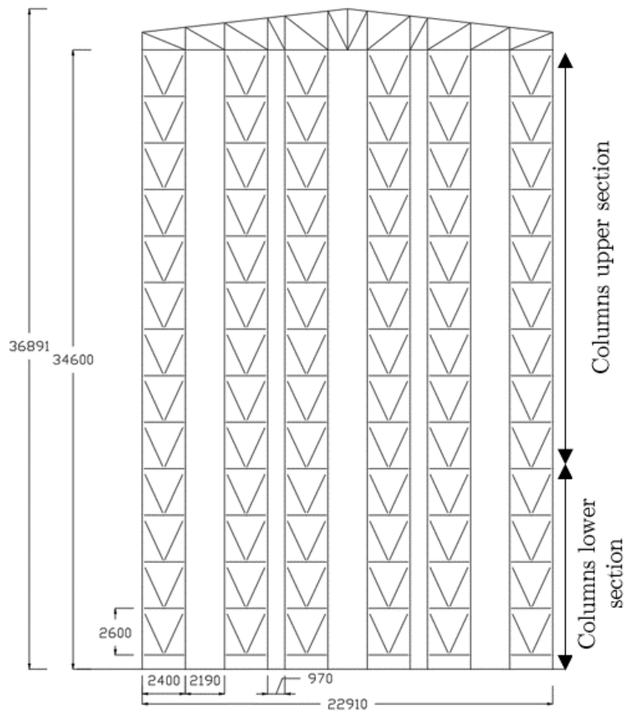

Figure 4 - Case study cross-aisle section

The 2D finite element models have been built in ABAQUS [18] using B21 beam elements. Joints connecting beams and bracings with columns are considered hinges and infinitely robust. Two models have been constructed: cross-aisle and down-aisle. Load units are europallets with a mass of 1000 kg; six pallets are handled in each loading bay. The warehouse is considered to be installed in Italy in a medium-high seismic zone. The design follows a *non-dissipative* philosophy and uses a response spectrum analysis. The parameters to define the spectrum are reported in Table 1, according to EN1998 [2] and EN16681 [3].

Table 1 – Seismic parameters

Peak Ground Acceleration [g]	a_g	0.15
Magnification Factor [-]	F_O	2.4
Reference Period [s]	T_c*	0.3
Soil Type	-	C

Topography	-	T1
Behavior factor (cross-aisle)	q_{ca}	1.33
Behavior factor (down-aisle)	q_{da}	1.50
	E_{D1}	1
Spectrum modification factor	E_{D2}	0.8
	E_{D3}	1
Fill factor (cross-aisle)	$R_{f,\,ca}$	1
Fill factor (down-aisle)	$R_{f,\,da}$	1

The criterion for selecting cross-sections is to pursue at least 85% exploitation for the main elements according to the checks required by the Italian Code [19] and EN1993-1-1 [1]. The results of the design are reported in Table 2.

Table 2 – Cross-sections used for the case study

Structural Element	Cross-section	Notes
Columns (lower section)	HSS 150x8	From 0 to 11.2 m
Columns (upper section)	HSS 120x5	From 11.2 to top
Frame Bracing	2xU 100x50x4	-
Brace Tower Bracings	2xU 60x80x4	Tension only
Beams	U 100x50x6	-
Truss upper/lower chords	HSS 100x100x3	-
Truss web beams	HSS 80x80x3	-

All the elements are in steel S355, and the temperature variation of mechanical properties follows EN1993-1-2 [20]

3.2 Fire load and scenarios

The temperature input was generated using the LOCAFI model [21], considering a localized fire started by the electrical equipment related to the AS/RS [22]. The latter equipment is located at the lower level and provides power to the AS/RS stacker crane to move along the aisle. The LOCAFI heat release rate curve (HRR) upper limit, 50 MW, was taken for safety because it aligns with other studies on fires in large warehouses [23]. The used fire scenarios are reported in Figure 5 and Figure 6. The reason for choosing these fire scenarios should be found in the structural system of an ARSW:

- Scenario #1: the lateral fire in the cross-aisle model could cause, after the frame failure, a domino effect that leads to the rotation of the building and a progressive collapse on the side of the section;
- Scenario #2: a central fire in the cross-aisle model could lead to a progressive structure collapse due to a subsequent failure of near uprights. However, if this happens, it might be an inward collapse. This possibility is also investigated to understand the structure and roof truss responses.
- Scenario #3: a central fire in the down-aisle model is examined to evaluate the possible catenary action of the pallet beams and the consequent high tensile force that could develop in them;
- Scenario #4: a lateral fire in the down-aisle model is explored to assess if the decrease in mechanical properties of the bracing tower could significantly reduce the lateral stiffness of the building and then collapse. However, bracing tower uprights in static conditions are less exploited, as they carry half the weight of the other uprights.

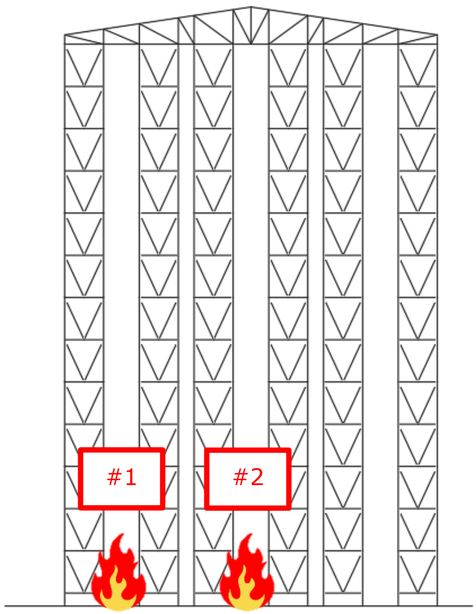

Figure 5 - Fire Scenarios (cross-aisle direction)

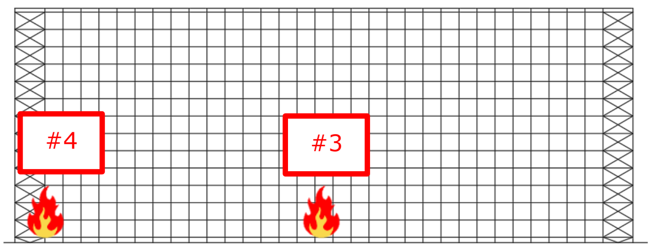

Figure 6 - Fire Scenarios (down-aisle direction)

4 Results

The uprights nomenclature used is reported in Figure 7 and Figure 8.

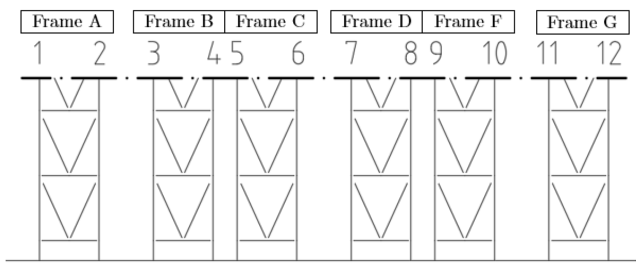

Figure 7 – Upright nomenclature, cross-aisle model

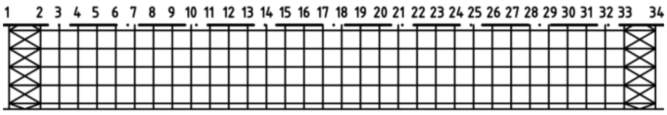

Figure 8 – Upright nomenclature, down-aisle model

The last steps of the deformation sequence are reported in Figure 9, Figure 10, Figure 11, and Figure 12.

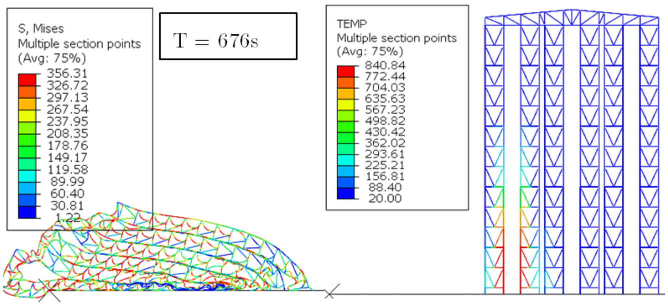

Figure 9 - Last step of scenario #1. Right, Von-Mises tension; left: temperatures

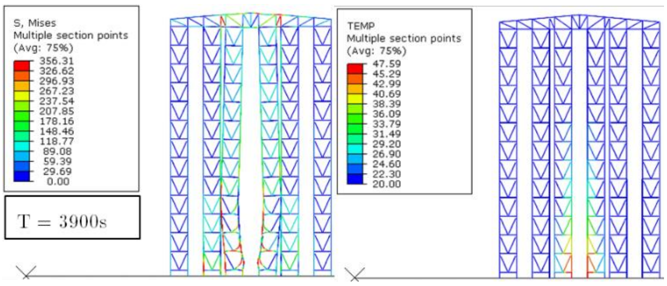

Figure 10 - Last step of scenario #2. Right, Von-Mises tension; left: temperatures

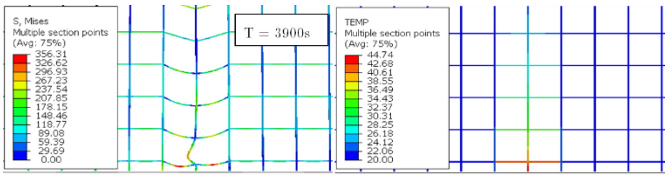

Figure 11 - Last step of scenario #3. Right, Von-Mises tension; left: temperatures

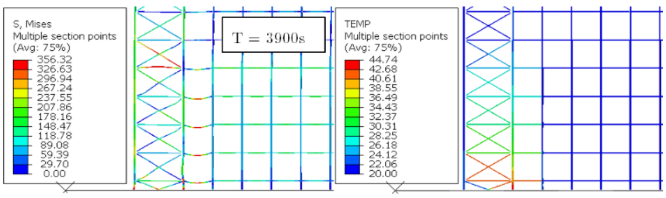

Figure 12 - Last step of scenario #4. Right, Von-Mises tension; left: temperatures

5 Discussion

A progressive collapse is observed in scenario #1: frame A is the first to collapse, then a domino effect occurs. The collapse starts when the roof truss fails (Figure 13); its failure is related to the behavior of frames A and B involved in the fire and to the axial forces in web beams that develop after a remarkable loss in the mechanical properties of heated elements. As the temperatures increase, frames A and B hang from the truss as they lose the capacity to bear loads leading to an elevated compressive force in the web beams and the consequent instability. Since the roof truss failed, a force redistribution cannot be performed. The fire-resistance time, identified in this case as the collapse of the first element of the roof truss, is about 600s (10 minutes). In scenario #2, web elements above the heated frames are in tension, and their capacity is enough to resist and perform in the elastic range. Even if marginally damaged, the roof truss can redistribute the load from the frames to the surrounding structure. The internal forces of the roof truss are analyzed at the moment of the frame hanging, and their schematic displays are reported in Figure 14.

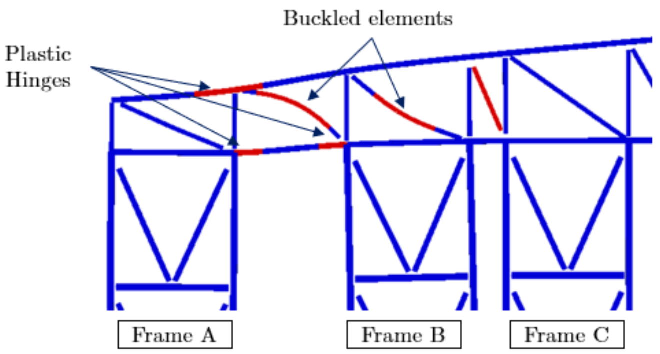

Figure 13 - Scenario #1, T = 179s. Frame A is collapsing, and the roof truss cannot redistribute the force leading to the collapse of the truss itself.

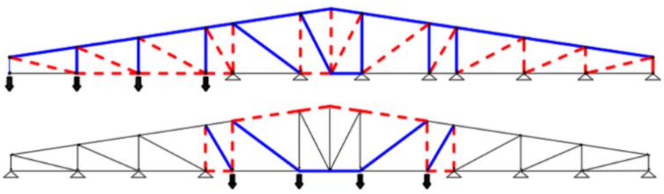

Figure 14 - Internal forces in the roof truss: up, scenario #1; down, scenario #2. Dashed red lines mean compression, blue lines mean tension and black lines are unloaded. Uprights not involved directly in the fire are represented as supports.

A partial collapse occurs in scenario #3 as the column involved in the fire buckles; however, thanks to the presence of the beams and the tower bracings, a collapse turning from local to global is prevented because beams connect every upright to the bracing tower. The latter not only increases the lateral restrain of the frame, reducing the pulling force effects on uprights but can also prevent the collapse from local to global. The latter assertion can be quickly evaluated considering a modified scenario #3 model. Since the fire is in the middle of the building, maintaining only one tower bracing leads the two parts of the warehouse to behave differently and highlight the role of the bracing (Figure 15).

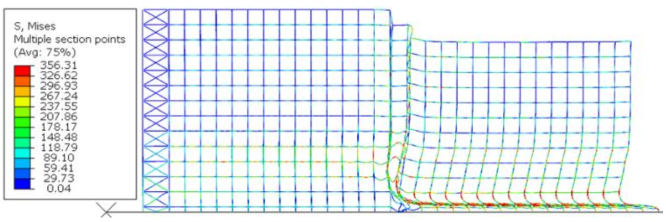

Figure 15 - T=436s, the collapse of the right side (braceless) for scenario #3 modified

Scenario #4 did not lead to a global collapse; the reason must be found in the redundancy of the down-aisle scheme of an automated rack-supported warehouse and the high level of connection. Moreover, bracing towers column are usually the strongest elements, as they bear the extra-axial force from horizontal actions and the cross-diagonal scheme. A generic representation of the internal forces in a down-aisle scenario is reported in Figure 16.

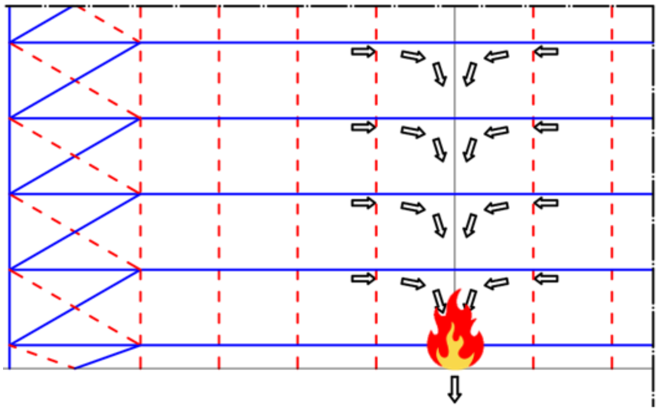

Figure 16 - Internal forces in a down-aisle scenario. Dashed red lines mean compression, and blue lines mean tension. The column involved is drawn in black.

Introducing a redistribution factor β, as

$$\beta(t) = F(t)/F_{initial}, \qquad (1)$$

where $F(t)$ is an internal force/moment in the upright and $F_{initial}$ is the initial axial force in pre-fire conditions; Figure 17, Figure 18, and Figure 19 report the values of the β coefficient for every upright at the end of the cooling phase.

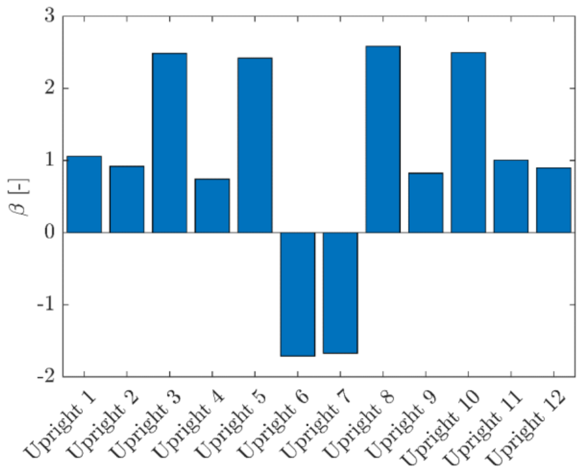

Figure 17 - β factor at the end of the analysis, scenario #2.

From the analysis of Figure 17, it is possible to note that high tensile forces develop in uprights 6 and 7. This behavior is due to the cooling phase after the fire. In the latter, temperatures in uprights 5, 6, 7, and 8 start decreasing, and the thermal expansion is recovered, leading to a decrease in the upright's length. However, since uprights 6 and 7 have passed the yielding point and have entered the inelastic field during heating, the final length of the column, after the complete recovery of the thermal expansion, is shorter than in pre-fire conditions. As a result, a tensile force develops in the uprights 6 and 7, and the latter pulls on the entire cross-section. Therefore uprights 6 and 7 can eventually transfer a force to the other vertical members larger than the load they were initially carrying.

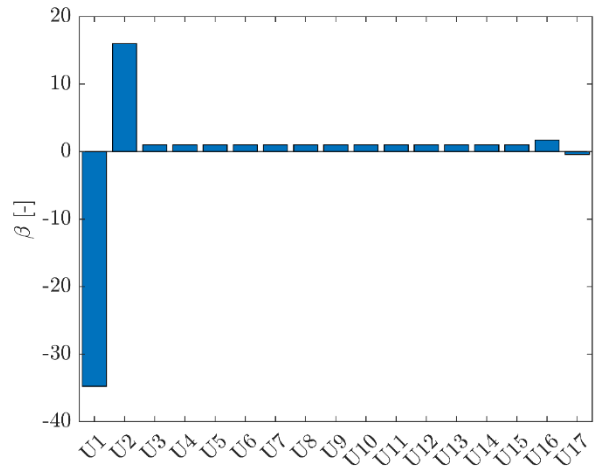

Figure 18 - β factor at the end of the analysis, scenario #3. U17 is the upright attacked by the fire. The figure reports just half of the uprights because the β factors are almost symmetrical

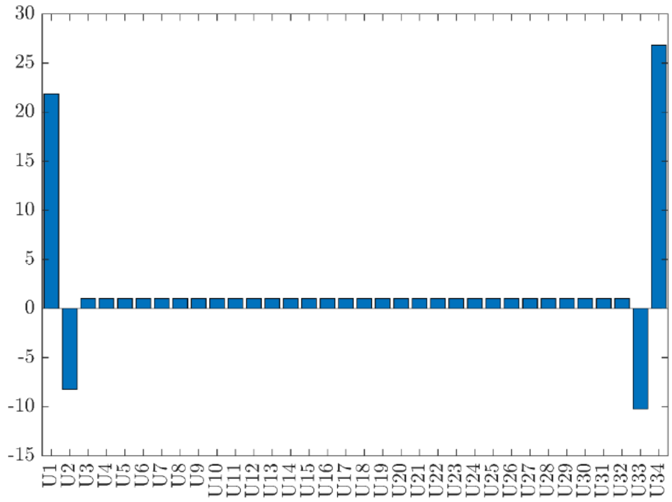

Figure 19 - β factor at the end of the analysis, scenario #4.

6 Concluding Remarks

This paper explored the fire-induced progressive collapse of automated rack-supported warehouses and tried to find the key features involved. It is worth noting that the intention of the calculations in this paper is neither to consider all possible variations of fire nor to describe the actual state of the construction after failure. Nevertheless, the calculation results can support the intended description of possible failure progression. The main results can be summarized as follows:

- An analysis procedure based on an implicit/explicit scheme has been developed to replace less approximate a posteriori estimation of kinematics based on the last step of an implicit framework. The procedure allows for overcoming numerical instabilities that affect the analysis when buckling, high strain rates, or unstable conditions develop during a fire;

- 2D cross-aisle analyses allowed for identifying the role of the roof-truss in redistributing forces that develop during a collapse. This could be a starting point for a new design method to avoid the progressive collapse of ARSW involving a roof-truss strengthening or using elements working as fuses in fire conditions.
- 2D down-aisle analyses highlighted the crucial role of the bracing towers even in case of fire, as its elevated lateral stiffness permits resisting the pulling forces born after localized failures and avoiding a progressive collapse.
- As the pallet beams working in tension can undergo high stresses, their beam-to-column bolted joints must be able to resist high shear forces. Otherwise, the ARSW will face a localized collapse as the beams start disconnecting from the column isolating the attacked column by the bracing system.

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