Journal of Mines, Metals and Fuels, *71(9): 1235-1249; 2023. DOI: 10.18311/jmmf/2023/35443*

Print ISSN : 0022-2755

Journal of Mines, Metals and Fuels

Contents available at: *www.informaticsjournals.com/index.php/jmmf*

A Numerical Study on the Fire Induced Collapse of a Real Life Warehouse Structure Based on Post-Fire NDT Results

Devjit Acharjee* and Debasish Bandyopadhyay

Department of Construction Engineering, Jadavpur University, Kolkata - 7000106, West Bengal, India; devjita.construction.rs@jadavpuruniversity.in

Abstract

Fires being a Low-Probability High-Consequence problem, can have a big impact on the structures due to their occurrence; irrespective of the construction materials. Even, the damage might lead to structural collapse. Such failures have warned to check the impact of fire on structural elements, which is frequently overlooked in popular design guidelines. Warehouses are used for the storage of different types of commodities; however, the vulnerability of these structures during fire event depends on the storage materials and other factors. Such a warehouse with steel roof trusses supported over Reinforced Cement Concrete (RCC) frame, located at Kolkata, suffered one of such severe fire-induced damage; and the roof truss system along with some portions of the brick masonry walls got totally collapsed. The information about the structure including its design data is presented along with the site observations after the fire incident, and the results of various Non-Destructive Tests (NDTs) performed over RCC frame elements. A severe fire of 8-hour duration led to a huge increase in temperature, which is considered to be the main reason of the progressive collapse of the entire warehouse structure. To better understand the failure of members due to temperature load, the present study aims to analyse a numerical model of the damaged structure in the Finite Element (FE) Framework. The roof truss, RCC frames and brick masonry walls are modelled; and temperature load is applied along with self-weight load to check the effect of incremental temperature on various structural responses of the warehouse. The temperature-dependent material properties are considered in the analyses; as applicable. Results have shown that there is a significant effect of temperature load, which gets worse with increasing temperature. The connection between roof truss and columns also contributes to the extent of damage in the truss supported over RCC frames; as evident from the numerical analysis. The present study seems to present a clear view about the fire-induced structural collapse of a real-life warehouse structure.

Keywords: *Finite Element Analysis, Fire-Induced Damage, Non-Destructive Test, Temperature Load, Warehouse*

1.0 Introduction

Fire is an unplanned and unintentional situation that affects all types of structures, from buildings and bridges[1] to warehouses[2,3] irrespective of the materials[1,4-6]. Thus, it is considered to be one of the Low-Probability High-Consequence (LPHC) problems in the field of engineering; and the mechanism of the disasters due to fire needs to be carefully monitored. One of such real-life fire events and the further collapse of a warehouse structure is presented in this current research paper; using numerical analysis based on post-fire NDT results and temperature-dependent properties of steel.

A bag storage warehouse with steel roof trusses supported over a Reinforced Cement Concrete (RCC) frame, located in Kolkata, suffered a severe fire of nearly

**Author for correspondence*

Figure 1. Failure of steel roof truss.

Figure 2. Collapse of brick masonry wall.

Figure 3. Economic loss due to the burning of the storage material.

8 hrs.; and the roof truss system (Figure 1) along with some portions of the brick masonry (Figure 2) walls got totally collapsed. Although it did not claim any life loss; the inflammable storage materials were totally burnt out and caused a huge economic loss (Figure 3).

Generally, fire-induced damage and its further structural collapse is a complicated thermo-mechanical behavior of the overall structure under the collapse limit state[7]. Such a behavior includes load redistributions in structural systems, substantial deformations of structural members, and the degradation of materials due to high temperatures. Understanding the general mechanical behavior of the warehouse structure during fire seems to be significantly important using a thorough numerical investigation. However, such a real-life scientific investigation is rarely presented[8,9]. Thus, the current paper aims to present a case study on the said warehouse, based on FE analysis and post-fire NDT results; which

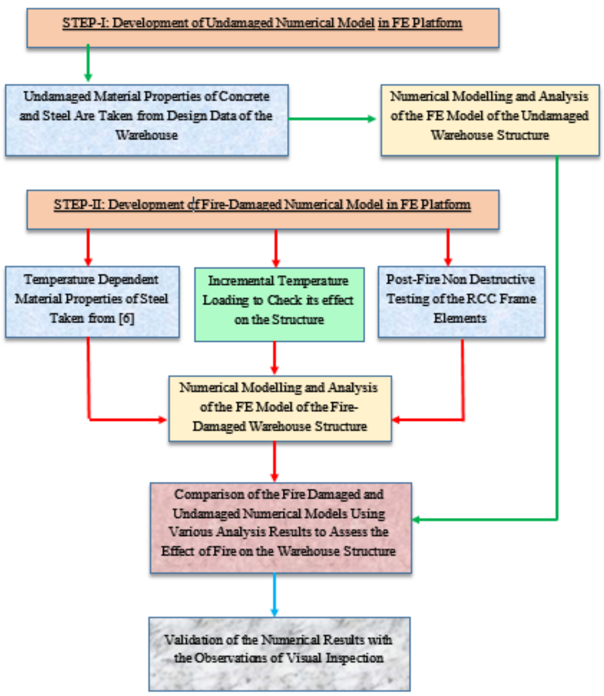

Figure 4. Flowchart of the methodology.

can provide useful engineering lessons learnt from the fire-induced structural damage.

The goal of the current study is to investigate numerical models of undamaged and damaged structures in the Finite Element (FE) Framework in order to better understand the failure of the members as a result of temperature load. To test the impact of incremental temperature on the various structural responses of the warehouse; the roof truss, RCC frames, and brick masonry walls are simulated considering temperature-dependent material properties and an incremental temperature load is imposed in addition to the self-weight and other loads, as applicable. Methodology of the current paper is presented below by the means of a flowchart (Figure 4).

The results demonstrate a considerable impact of temperature load that worsens as the temperature rises. The connection between the roof truss and columns also has a role in the degree of damage suffered by the truss members supported over RCC frames. The current investigation appears to provide a clear picture of the structural collapse of the warehouse caused by fire.

2.0 Non-Destructive Testing

Generally, concrete as a building material is durable to fire, but might be deteriorated due to fire. Thus, a thorough post-fire assessment of concrete structural elements is essential to determine the condition of the considered structure. In the present case study, Non-Destructive Testing (NDTs) like the Schmidt Rebound Hammer Test (Figure 5) and Ultrasonic Pulse Velocity Test (Figure 6) confirming to relevant IS codes[10,11] were conducted on the fire-damaged RCC structural members to better understand the material properties after the fire event. Although these test results are not directly correlated with fire-induced damage, but are used to evaluate the present mechanical characteristics of concrete (the strength of the concrete is evaluated from the Hammer test, and the modulus of elasticity is derived from the UPV test results).

During visual inspection of the warehouse structure, considerable damage of truss joints and the connection between the roof truss and the columns were observed

Figure 5. Rebound hammer test of the RCC frame members.

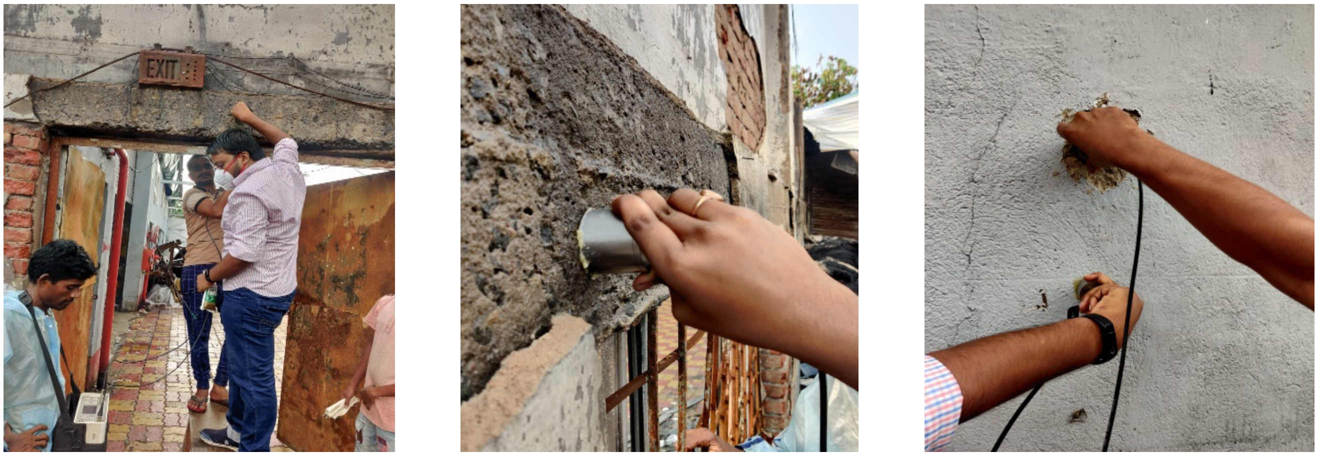

Figure 6. Ultrasonic pulse velocity test of the RCC frame members.

Figure 7. Damage of the truss joint and connection between the roof truss and the columns.

(Figure 7); which is mainly due to the temperature load.

3.0 Numerical Modelling

The numerical models of the considered real-life Warehouse Structure are developed as per the actual dimensions (Figure 8) in the Finite Element platform considering initial and fire-damaged material properties (temperature-dependent material properties of steel and damaged mechanical properties of concrete). The Finite Element models consist of space frame elements (for RCC beams and columns, purlins, and different trusses) and 2-dimensional thin shell elements (for brick walls and tin roofing sheets, without considering shear deformation) for numerical analysis. To assess the effect of different extent of fixities of the connections between roof truss and RCC columns on the damage of the steel roof trusses, two types of boundary conditions are considered for analysis; i.e. pinned and fixed.

3.1 Material Specifications

The undamaged material properties of the RCC structural members and roof trusses are taken from the design data, and those are given below. Brick Masonry Walls and Tin Roof Sheet are also considered in the numerical modeling, to assess the global effect of temperature loading on the warehouse structure; in a better manner. The mechanical properties of brick masonry walls are calculated from the available literatures[12,13]. The modulus of elasticity of Tin Roof Sheet is taken as 50000 MPa, as available in[14].

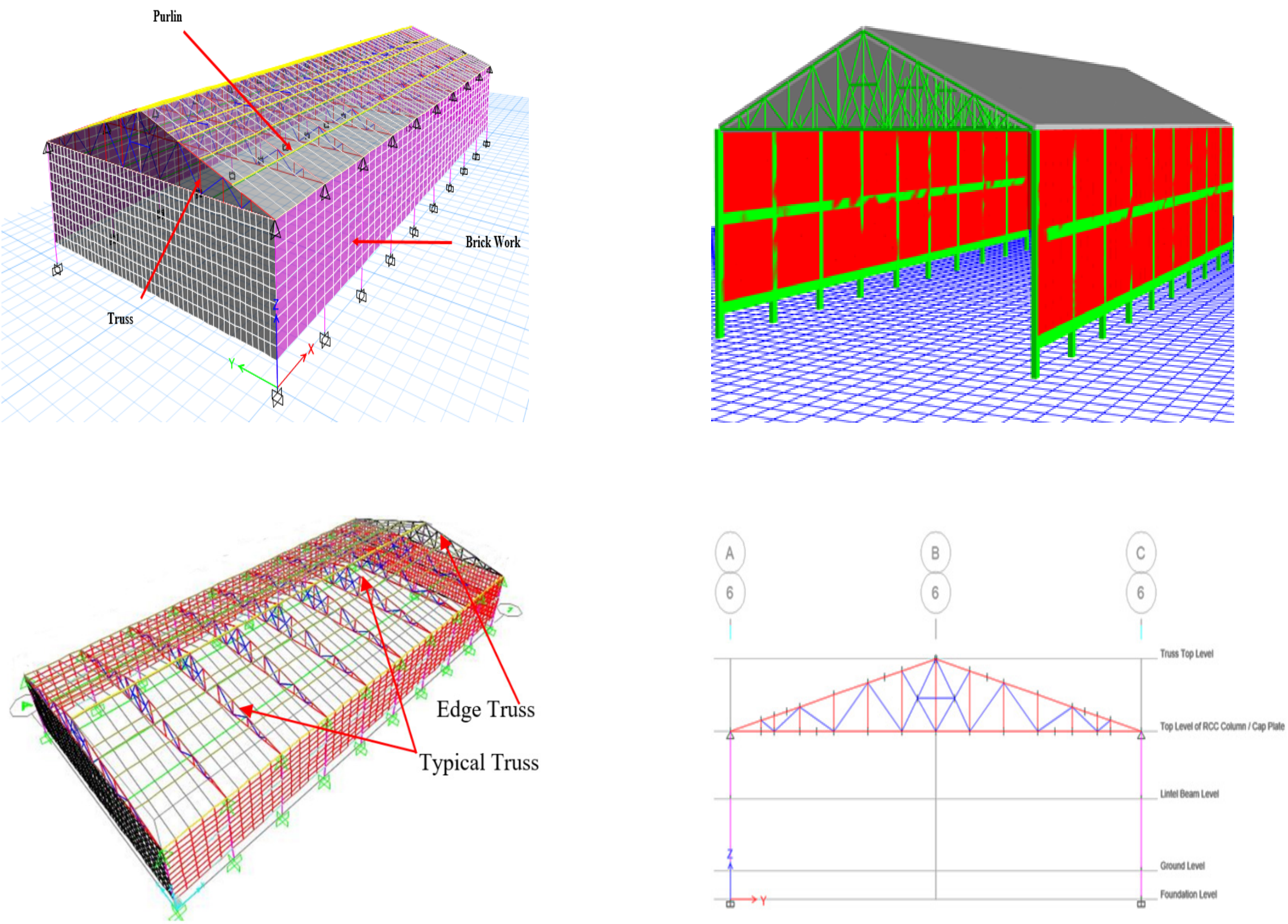

Figure 8. Isometric and front elevation views of a numerical model of the warehouse.

Table 1. Undamaged material properties of the warehouse

Materials	Grade and mechanical characteristics of materials
Concrete	M 20 (with characteristic strength of 20 MPa)
Steel Reinforcement	Fe 415 (with minimum yield and tensile strength of 415 MPa and 485 MPa respectively)
Steel Truss Members	Fe 345 (with minimum yield and tensile strength of 345 MPa and 450 MPa respectively)
Brick Masonry Wall	Modulus of Elasticity of 3850 MPa and Compressive Strength of 13.79 MPa
Tin Roof Sheet	Modulus of Elasticity of 50000 MPa

3.1.1 Mechanical Properties of Fire Damaged Concrete

The fire-damaged material properties of concrete are derived from the results of NDTs performed. The strength of damaged concrete is evaluated from the Rebound Hammer test[11] and the dynamic modulus of elasticity[15] is derived from the UPV test results. Dynamic modulus of elasticity of concrete is calculated using the following formula (as per IS 516 (Part 5/Sec 1) codal provision)[10]:

$$E = \frac{\rho(1+\mu)(1\text{-}2\mu)}{(1\text{-}\mu)}V^2 \qquad (1)$$

Where, E = dynamic young's modulus of elasticity (in MPa), ρ = density (in kg/m^3), V = pulse velocity (in m/sec).

The static modulus of elasticity is calculated as per the formula[16],

$$E = E_d/1.16 \qquad (2)$$

The measured NDT data (Rebound Hammer Test Data) has shown that the strength reduction in concrete members was from 18 to 55 % maximum, for the worst case. This strength reduction is duly considered in the numerical modeling.

3.1.2 Variation of Mechanical Properties of Steel with Temperature

The temperature-dependent material properties are taken from IS 800[17] for steel sections. The following considerations regarding the effect of temperature on steel's yield stress and modulus of elasticity are made:

$$\frac{f_y(T)}{f_y(20)} = \frac{905-T}{905} \le 1.0 \qquad (3)$$

where, $f_y(T)$ = yield s tress of steel at T^{o}C, $f_y(20)$ = yield stress of steel at 20^o C, T(^{o}C) = temperature of the steel. For T< 215°C, no reduction needs to be considered.

$$\frac{E(T)}{E(20)} = 1.0 + \left[\frac{T}{2000\left[\ln\left(\frac{T}{1100}\right)\right]}\right]$$

when, $0^o C < T \le 600^o C$ (4)

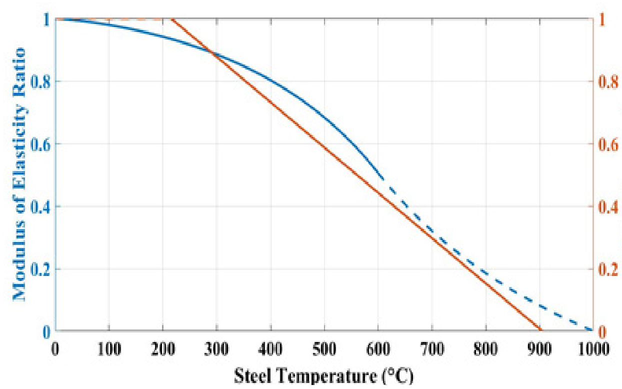

Figure 9. Variation of mechanical properties of steel with temperature[17].

Table 2. Section properties of different elements

Sections	Width (mm)	Depth (mm)	Outer Diameter (mm)	Thickness (mm)
Beams	250	300	NA	NA
Columns	250	375	NA	NA
Purlins	NA	NA	75	4
Ties	NA	NA	75	4
Diagonals	NA	NA	62.5	3
Verticals	NA	NA	75	4
Rafters	NA	NA	75	4
Brick Wall	NA	NA	NA	250
Roofing Sheet	NA	NA	NA	3

The relationships are shown by Curve 1 and 2 in Figure 9.

3.2 Section Details

The section properties of various elements of the warehouse are tabulated in Table 2.

3.3 Considered Loads for Analysis

Loads for analyzing the FE models as per IS codes[15,18-21] are given in Tables 3 to 5.

3.4 Temperature Load Considered for the Numerical Analysis

In the present study, the effect of fire damage is evaluated by applying the temperature load, that the structure is assumed to be subjected to. The warehouse was subjected to a fire of 8-hour duration; a maximum difference of temperature in the inner and outer surfaces of the said structure is taken as 600°C[22,23]. To check the adverse effect of incremental temperature loading, intermediate temperatures of 200°C and 400°C are also considered.

Table 3. Various gravity loads applied in numerical models

Codal Stipulations used1[18,19]	Type of Loading	Considered Load
IS 875 (Part 1): Dead Loads-Unit Weights of Building Materials and Stored Materials	Dead Load (DL)	Self-weight of the elements.
IS 875 (Part 2)	Live Load (LL) on Inaccessible Roof Slabs	0.75 kN/m^2

Table 4. Wind load calculation as per IS 875 (Part 3): 2015

Parameters	Values
Basic Wind Speed, V_b for Kolkata	50 m/s
Risk coefficient, K_1	1
Terrain category	III
Topography factor, K_3	1
K_4	1
Design wind speed, V_z	V_b x K_1x K_2x K_3x K_4
Design wind pressure, P_z	(0.6 xV_z^2)

Table 5. Seismic load calculation as per IS 1893 (Part 1):2016

Parameters	Values
Importance Factor for Warehouses	I = 1.5
Response Reduction Factor for OMRF Structures	R = 3
Soil Type	Medium Stiff
Seismic Zone, for Kolkata	IV
Seismic Mass	100% of DL

4.0 Results and Discussions

This section of the current article presents and discusses the various structural responses based on the temperature load analysis[21] results of the fire-damaged real-life warehouse. To evaluate the impact of incremental temperature load on the said structure, those responses like Interaction ratio and deflection of roof truss members and design demand of reinforcements[24] of RCC columns are compared.

4.1 Comparative Study of Interaction Ratio of Roof Truss Members

Although, the truss members are idealized as tension-compression members only; roof trusses are subjected to combined axial compression and biaxial bending, thus satisfying the following interaction relationships:

$$\frac{P}{P_{dy}} + K_y \frac{C_{my} M_y}{M_{dy}} + K_{LT} \frac{M_z}{M_{dz}} \leq 1.0 \qquad (5)$$

and

$$\frac{P}{P_{dz}} + 0.6 K_y \frac{C_{my} M_y}{M_{dy}} + K_z \frac{C_{mz} M_z}{M_{dz}} \leq 1.0 \qquad (6)$$

Where, the notations are of usual meanings (refer to Clause 9.3.2.2 of IS 800)[17].

Values of interaction ratios for various types of truss members are given below:

The results of the analyses, as presented in the above tables; depict that the interaction ratios of all types of truss elements increase with incremental temperature. Higher the interaction ratio, the greater the load that a truss element is subjected to. The detrimental effect of temperature loading is higher for the edge trusses than that of the typical trusses. The rafters and bottom chord members are more prone to damage due to temperature loads, while the diagonal elements are observed to be prone to damage less than that of the rafters and bottom chord members; but higher than that of the vertical elements. The roof trusses supported over fixed joints

Table 6. The interaction ratio of various types of elements of pin-supported typical trusses

No Temp Load		200°C Temp Load		400°C Temp Load		600°C Temp Load	
Member	**Max Int. Ratio**	**Member**	**Max Int. Ratio**	**Member**	**Max Int. Ratio**	**Member**	**Max Int. Ratio**
Rafter	**0.339**	**Rafter**	**All > 1**	**Rafter**	**All > 1**	**Rafter**	**All > 1**
Diagonal	0.142	Diagonal	For 33% members, the ratio is > 1; others pass with maximum of 0.663	Diagonal	For 50% members, the ratio is > 1; others pass with maximum of 0.922	Diagonal	For 90% members, the ratio is > 1; others pass with maximum of 0.848
Vertical	0.07	Vertical	0.466	Vertical	For 30% members, the ratio is > 1; others pass with maximum of 0.407	Vertical	For 30% members, the ratio is > 1; others pass with maximum of 0.658
Bottom Chord	0.116	Bottom Chord	All > 1	Bottom Chord	All > 1	Bottom Chord	All > 1

Table 7. Interaction ratio of various types of elements of pin-supported edge trusses

No Temp Load		**200°C Temp Load**		**400°C Temp Load**		**600°C Temp Load**	
Member	**Max Int. Ratio**	**Member**	**Max Int. Ratio**	**Member**	**Max Int. Ratio**	**Member**	**Max Int. Ratio**
Rafter	**0.306**	**Rafter**	**All > 1**	**Rafter**	**All > 1**	**Rafter**	**All > 1**
Diagonal	0.101	Diagonal	For 33% members, the ratio is > 1; others pass with maximum of 0.7	Diagonal	For 60% members, the ratio is > 1; others pass with maximum of 0.924	Diagonal	For 90% members, the ratio is > 1; others pass with maximum of 0.848
Vertical	0.134	Vertical	0.555	Vertical	For 70% members, the ratio is > 1; others pass with maximum of 0.96	Vertical	For 93% members, the ratio is > 1; others pass with maximum of 0.719
Bottom Chord	0.11	Bottom Chord	All > 1	Bottom Chord	All > 1	Bottom Chord	All > 1

Table 8. The interaction ratio of various types of elements of fixed-supported typical trusses

No Temp Load		**200°C Temp Load**		**400°C Temp Load**		**600°C Temp Load**	
Member	**Max Int. Ratio**	**Member**	**Max Int. Ratio**	**Member**	**Max Int. Ratio**	**Member**	**Max Int. Ratio**
Rafter	**0.630**	**Rafter**	**All > 1**	**Rafter**	**All > 1**	**Rafter**	**All > 1**
Diagonal	0.318	Diagonal	For 45% members, the ratio is > 1; others pass with maximum of 0.840	Diagonal	For 70% members, the ratio is > 1; others pass with maximum of 0.947	Diagonal	All > 1
Vertical	0.205	Vertical	0.616	Vertical	For 50% members, the ratio is > 1; others pass with maximum of 0.549	Vertical	For 60% members, the ratio is > 1; others pass with maximum of 0.834
Bottom Chord	0.277	Bottom Chord	All > 1	Bottom Chord	All > 1	Bottom Chord	All > 1

Table 9. Interaction ratio of various types of elements of fixed supported edge trusses

No Temp Load		200°C Temp Load		400°C Temp Load		600°C Temp Load	
Member	**Max Int. Ratio**	**Member**	**Max Int. Ratio**	**Member**	**Max Int. Ratio**	**Member**	**Max Int. Ratio**
Rafter	**0.577**	**Rafter**	**All > 1**	**Rafter**	**All > 1**	**Rafter**	**All > 1**
Diagonal	0.254	Diagonal	For 48% members, the ratio is > 1; others pass with maximum of 0.889	Diagonal	For 85% members, the ratio is > 1; others pass with a maximum of 0.98	Diagonal	All > 1
Vertical	0.306	Vertical	0.709	Vertical	For 90% members, the ratio is > 1; others pass with a maximum of 0.985	Vertical	For 96% members, the ratio is > 1; others pass with maximum of 0.929
Bottom Chord	0.268	Bottom Chord	All > 1	Bottom Chord	All > 1	Bottom Chord	All > 1

have developed more stresses than the trusses supported over pinned joints, as evident from the interaction ratios of the truss elements. This suggests that the higher the fixity between the trusses and the RCC columns, higher the damage of the truss members; as observed from the results.

4.2 Comparative Study of Deflection of Roof Truss Members

For each of the edge trusses as well as the typical trusses, two nodes are selected to compare the deflections (Figure 10) for incremental temperature loading

Table 10. Defections of the selected nodes of pin-supported typical trusses

Sl. No.	Load Combination	Deflection (mm)		Node No.	Temp (°C)
		Global Y	Global Z		
1	DL+LL	0	-6.922	1	NA
2	DL+LL	0	-7.003	2	
3	DL+TL	0.025	-61.957	1	200
4	DL+TL	0	-56.326	2	
5	DL+TL	0.026	-141.638	1	400
6	DL+TL	0	-127.567	2	
7	DL+TL	0.026	-189.861	1	600
8	DL+TL	0	-173.234	2	

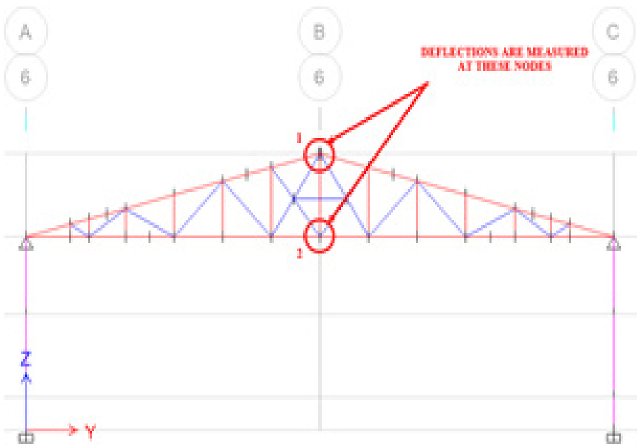

Figure 10. Nodes selected for comparing the deflections.

considering different types of support conditions of trusses. For numerical models without temperature load; a combination of dead load and live load is considered for checking the deflections. For the FE models considering temperature load; a combination of dead load and temperature load is considered. The results, as presented in Table 10 to Table 13; suggests that the deflection gets increased with incremental temperature, and the edge trusses are more vulnerable than that of the typical trusses during fire hazard. Again, less deflection of trusses over fixed support represents the development of higher reaction force; thus more prone to damage.

Table 11. Defections of the selected nodes of fixed-supported typical trusses

Sl. No.	Load Combination	Deflection (mm)		Node No.	Temp (°C)
		Global Y	Global Z		
1	DL+LL	0	-4.944	1	NA
2	DL+LL	0	-5.002	2	
3	DL+TL	0.025	-40.601	1	200
4	DL+TL	0	-36.911	2	
5	DL+TL	0.026	-92.817	1	400
6	DL+TL	0	-83.596	2	
7	DL+TL	0.026	-124.417	1	600
8	DL+TL	0	-113.522	2	

Table 12. Defections of the selected nodes of pin-supported edge trusses

Sl. No.	Load Combination	Deflection (mm)		Node No.	Temp (°C)
		Global Y	Global Z		
1	DL+LL	0	-7.388	1	NA
2	DL+LL	0	-7.474	2	
3	DL+TL	0.027	-66.275	1	200
4	DL+TL	0	-60.250	2	
5	DL+TL	0.028	-151.534	1	400
6	DL+TL	0	-136.478	2	
7	DL+TL	0.028	-203.132	1	600
8	DL+TL	0	-185.341	2	

Table 13. Defections of the selected nodes of fixed supported edge trusses

Sl. No.	Load Combination	Deflection (mm)		Node No.	Temp (°C)
		Global Y	Global Z		
1	DL+LL	0	-4.917	1	NA
2	DL+LL	0	-4.975	2	
3	DL+TL	0.027	-40.574	1	200
4	DL+TL	0	-36.884	2	
5	DL+TL	0.028	-92.79	1	400
6	DL+TL	0	-83.569	2	
7	DL+TL	0.028	-124.39	1	600
8	DL+TL	0	-113.495	2	

4.3 Comparative Study of Design Demand of Reinforcement of RCC Columns

Design demand of reinforcement of the RCC columns[24] helps to study the effect of incremental temperature load (from limit state of collapse perspective); as it indicates the maximum design force that the RCC columns are subjected to. The significant increase in reinforcement demand due to the incremental temperature refers to the development of distress and subsequent vulnerability in the lateral load-resisting system of the structure, while subjected to the devastating fire. For a certain temperature, the values of maximum reinforcement demand amongst the columns are given in Table 14.

4.4 Comparative Study of Brick Wall Stress

The stress contour plots for a certain brick wall of the warehouse are shown in Figure 11 to Figure 14; with different load combinations. The wall stress has increased

Table 14. Design demand of reinforcement of RCC columns

Sl. No.	Temp (°C)	Max. Design Reinforcement Required (%)
1	NA	0.9
2	200	4.35
3	400	>6
4	600	>6

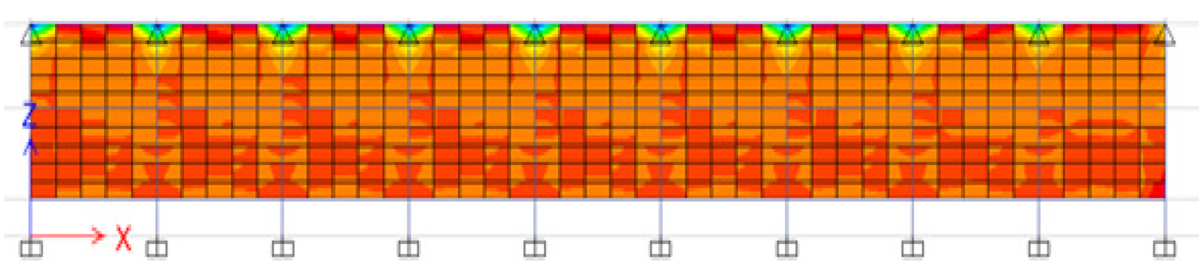

Figure 11. Stress contour of brick walls for load combination DL+LL.

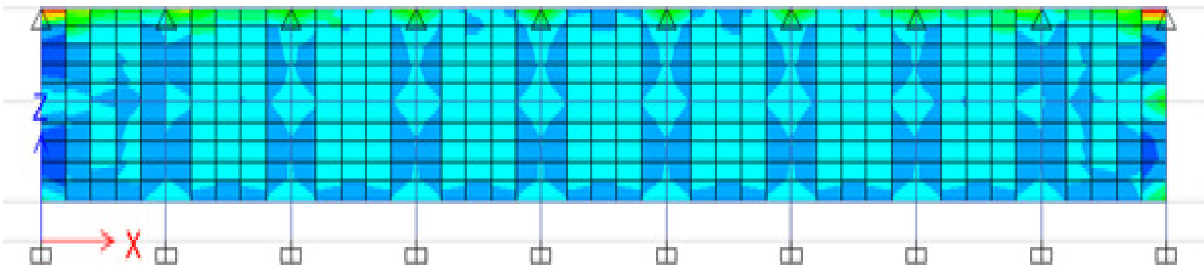

Figure 12. Stress contour of brick walls for load combination DL+TL (200oC).

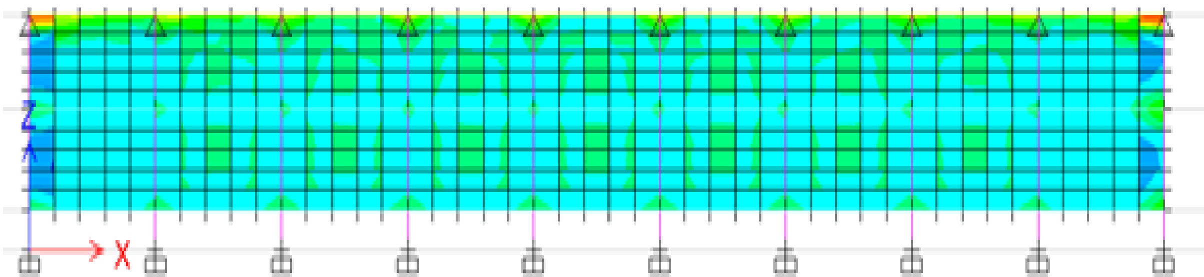

Figure 13. Stress contour of brick walls for load combination DL+TL (400oC).

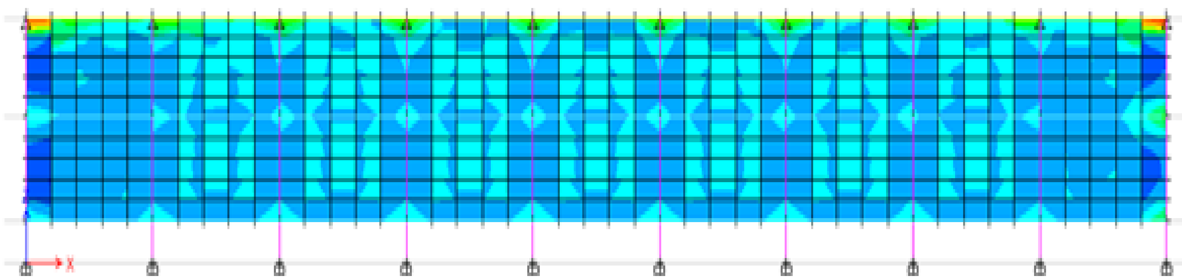

Figure 14. Stress contour of brick walls for load combination DL+TL (600oC).

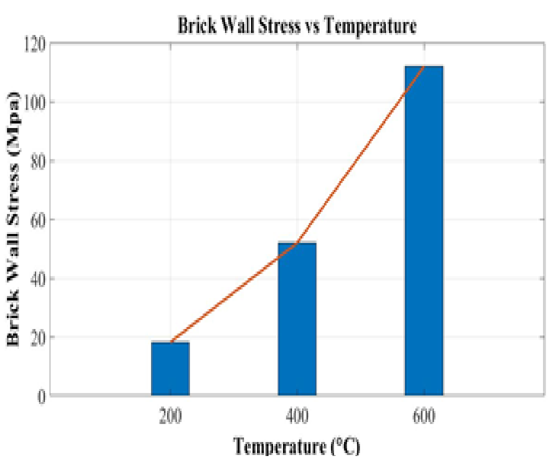

Figure 15. Brick wall stress vs temperature plot of warehouse subjected to fire hazard.

significantly (around 100 times) as the temperature load is considered during analysis. The maximum value of wall stress increases with incremental temperature (Figure 15). Such higher stress could result in cracks and further collapse (through fracture[25]) of the brick walls; as it happened during the specified case.

4.5 Validation of the Numerical Results

During visual inspection of the collapsed warehouse, it is observed that the vertical truss members are less prone to damage than the diagonals, rafters and bottom chords during the fire (Figure 16). From the present numerical study; it is quite evident that the interaction ratio of the vertical members is significantly lower than the

Figure 16. Damage extent of different truss members observed during visual inspection.

diagonals, rafters and bottom chords; when subjected to fire. The finite element analysis results are in tune with the observations of visual inspection; and thus are validated.

5.0 Conclusions

The present study is based on the Temperature Load Analysis of FE models of a real-life warehouse with steel trusses supported over RCC frames; to understand the global failure of the warehouse structure due to fire. Various parameters like interaction ratios and deflections of different truss elements over different types of joint fixities and design demand of reinforcements of RCC columns are compared with incremental temperature loading, and the presented results converge to the conclusions given below:

- There is a significant impact of temperature load that gets even worse with incremental temperature.
- The interaction ratio of edge truss elements are observed to be higher, thus leading to be more dangerous than the typical (intermediate) trusses during fire hazard.
- The deflection of the roof truss members also depicts that edge trusses are more vulnerable during fire.
- For a specific truss system, it has been found that the bottom chords and rafters are most affected; whereas the vertical elements are least affected due to fire hazard, as per the interaction ratios evaluated from the analysis.
- The degree of damage sustained by truss members supported over the RCC frames is influenced by the fixity of connection between roof truss and columns.
- Higher the fixity, greater the development of stresses; and thus the damage extent is high. It is observed from both the results based on interaction ratio, as well as deflection of the trusses.
- The stress contour plots for a specific brick wall has been presented; and it is observed that the stress is increased significantly with incremental temperature, finally leading to the collapse of that wall.
- Design reinforcement demand of RCC columns is also investigated to check the effect of incremental temperature. The increase in design reinforcement demand indicates about development of a higher design force in RCC columns.

Thus, the detailed finite element analysis based on NDT results seems to be significant in presenting a clear picture of the distress suffered by the warehouse structure before its collapse due to fire. The proposed study might be subsequently extended to evaluate different vulnerability indices of these types of fire-damaged structures.

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