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A Study on Progressive Collapse Behavior of Steel Structures Subjected to Fire Loads

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Abstract

Progressive collapse is one of the main reasons for the failure of structure. It occurs due to removal/ damage of a column or a shear wall by fire, blast or vehicle impact. In this study, aG+7 moment resisting steel frame residential building was analysed using ETABS to predict the sensitivity of the structure to progressive collapse due to fire loads. Columns at different levels were given a temperature of 550̊ C with reduced material properties and yield strength as per code IS 800. Progressive collapse load combination was adoptedas per GSA guidelines. Corner, edge, intermediate and re-entrant columns were removed separately at alternate storeys. The lower storeys were found to be more susceptible than the upper storeys. The structure may be redesigned to avoid progressive collapse, with a significant increase in steel consumption. This study can be useful for important structures..

Keywords: ETABS, Fire Load, GSA Guidelines, Moment Resistingsteel Frame, Progressive Collapse

1. Introduction

Progressive collapse occurs, when any one of the major structural load carrying members is removed suddenly from a building due to any unfavourable situation or condition and if the remaining structural elements are not capable of supporting the whole weight of the building. For example,if a column is damaged due to fire, man-made or natural hazards, the whole weight of the building (gravity load) inclusive of imposed loads are displaced to adjacent columns of the structure. If these adjacent columns are also not that much strong and stiff to carry the additional loads, they would have also been failed. As a consequence, the vertical load carrying elements may loose their strength and thus the massive collapse of the structure occurs. This failure usually occurs in a domino effect and precedes to a progressive collapse of the structure.

The progressive collapse behaviour of steel-frame buildings under fire load has been studied by lot of researchers for the past two decades.The General Services Administration (GSA) (2003) guidelines suggested some general expressions and conditions to predict the members which may be prone to the progressive collapse. These guidelines also recommended Demand Capacity Ratio (DCR) values to evaluate the intensity of damage of individual members of the structure due to progressive collapse.

The progressive collapse of an existing Hotel located in San Diego, California was examined both experimentally and analytically. The strain occurred due to the removal of the exterior columns from the building was measured experimentally with the help of strain gauges. From the Cardington full-scale fire tests, a dissimilarity has been observed between the real behavior under fire and the experimental behavior through standard furnace tests for the structural elements.

The reason is in real buildings, structural elements form part of a continuous assembly and therefore the higher temperature due to fire often remain localized, by

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receiving significant restraint from the sorrounded cool areas[1].

For the research purpose, a number of numerical models have been created to simulate the real behaviour of steel or steel-composite frame subjected to fire load. A two dimensional model has been developed to simulate the progressive collapse of multi-storey composite buildings. To study the behavior of three dimensional steel frames subjected to blast load and fire attack, a mix-element model has been designed which is capable of depicting the detailed behaviour of the member and instability of frames related with the effects of high-strain rate and fire temperature[2,3]. Later oncomputer programmes such as SAFIR and ABAQUS have been developed by the researchers to carry out the structural analysis of steel frames at elevated temperatures. From the literature survey, it seems to be very clear that these modeling strategies are effective only to describe the structural behaviour in which the fire loading time ranges from 0.5 to 4 hours.

In this regard, the present study investigates the progressive collapse of a moment resisting steel frame residential building (G+7) subjected to fire loading at different levels using ETABS software[5]. The Demand Capacity Ratio (DCR) valuesof the adjacent structural elements are calculated, when the columns at different levels have been failed due to the fire accidents. Based on the limit of DCR values given by GSA guidelines[7], the sustainability of the structure to progressive collapse is predicted.

The main objective of this study was to analyse the response of the steel structures due to a sudden loss of one or more columns under fire loadusing computational modeling in a stepwise manner. The first step in modeling is identification of the thermal loads on a structure due to fire. The second step is creating more appropriate interfaces to link the thermal and structural models to create an efficient computational modeling. The load combination was taken as per IS 875 part I and II. As recommended by GSA the columns were removed from corner, edge, intermediate and re-entrant corner separately at alternate storeys.In this study, structural behaviour of 8-storey moment resisting steel frame building (G+7)has been designed according to Indian codes. A nonlinear dynamic analysis procedure is recommended by the UFC 4-023-03 guideline, which provides technical guidance for mitigation of progressive collapse and thus to protect the structures.

2. Methodology

An8 storey 3D building was modeled (Figure 1) for this study in Extended 3D Analysis of Building System (ETABS 2015) software, which can performdesign and analysisof structures. Type of Building was a steel moment resisting space frame residential building with concrete slab. Plan of the model was irregular in shape with reentrant corners.The data used for analysis of building is shown in Table 1.

Table 1. Data used for analysis

Columns (Built up section)	**Depth**	**450 mm**
	Flange Width	**300 mm**
	Web thickness	**20 mm**
	Lange thickness	**32 mm**
Primary Beams	ISWB350	
Secondary Beams	ISMB300	
Slab Thickness	150 mm	
Zone	3	
Response Reduction Factor	4	
Important Factor	1	
Damping Ratio	0.02	
Time Period (X)	0.4236 seconds	
Time period (Y)	0.4955 seconds	

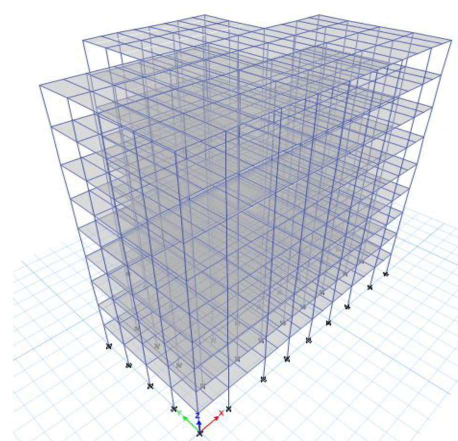

Figure 1. 3D Model of steel building.

The loads and loads combination were taken as per Indian Standard IS 875 (Parts 1 & 2). Live load was takenas 3 kN/m² on slab and dead load of wallwas taken as

12 kN/m on primary beam. The load combinations are shown in Table-2.

Table 2. Load combination

S. No.	Load combinations
1	1.5(DL + LL)
2	1.2(DL + LL ± EQ)
3	1.5(DL ± EQ)
4	0.9DL ± 1.5EQ

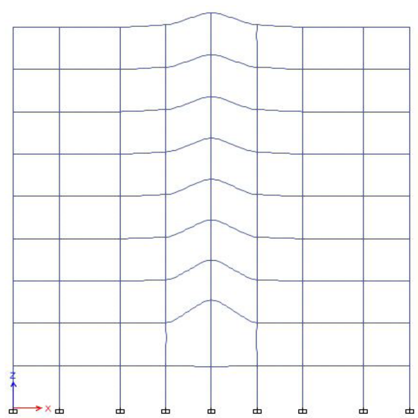

Figure 2. Initial stage of fire loading.

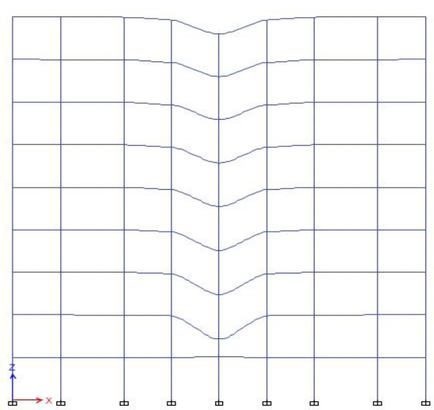

Figure 3. Final Stage of Fire Loading.

After giving a fire load on column in initial stage, steel column expands as per shown in Figure 2. As fire temperature was increased, the column loses its modulus of elasticity and rigidity. In the final stage i.e., when fire reachesthe melting point of steel, the column get collapsed as shown in Figure 3. In this paper temperature was taken as 550 °C.

3. Results

Temperature was given to column at different location of building as per GSA guideline. In this paper fire load was given at corner column, edge column, intermediate column and Re-entrant column of each ground floor, second column, fourth floor and sixth floor. As per GSA guideline the DCR of each element should be less than 2. If the DCR value exceeds 2, the progressive collapse willoccur. The progressive collapse gets started at members which are supported by column and also under fire load.

Figure 4 to Figure 7 represent initial deformed shape of building after giving fire load at corner column at ground floor, second column, fourth floor and sixth floor respectively. Figure 8 represents notation of corner column.

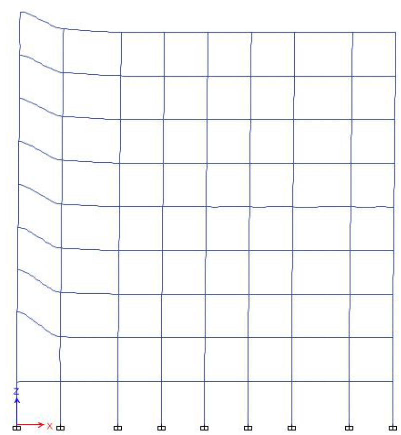

Figure 4. Deformedshape of ground floor corner column under fire.

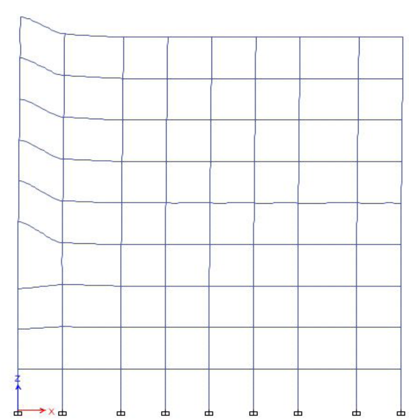

Figure 5. Deformed shape of second floor corner columnunder fire.

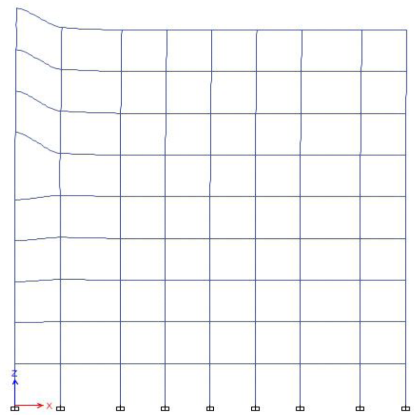

Figure 6. Deformed shape under fire load on corner column at fourth floor.

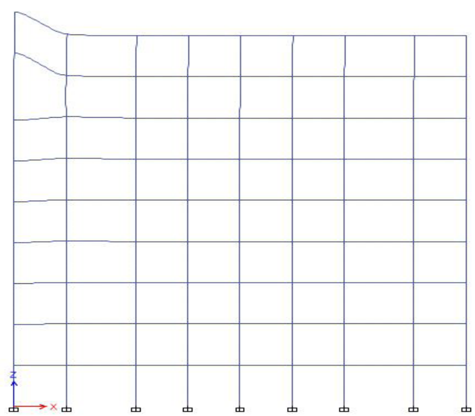

Figure 7. Deformed shape under fire load on corner column at sixth floor.

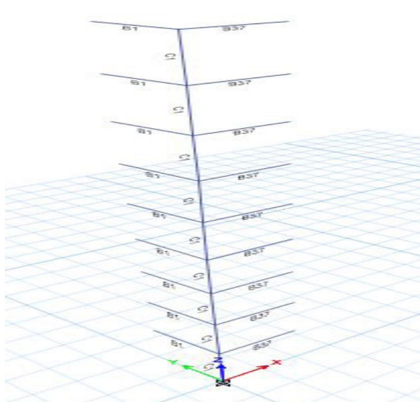

Figure 8. Notation of corner column.

Table 3. DCR values of specified floor corner members

Location	Members	DCR Values	
		Before Fire	After Fire
Ground Floor corner	Column C1	0.499	1.133
	Beam B37	0.331	1.266
	Beam B1	0.502	1.1533
II Floor Corner	Column C1	0.302	0.768
	Beam B37	0.377	1.178
	Beam B1	0.489	1.052
IV Floor Corner	Column C1	0.173	0.585
	Beam B37	0.291	0.996
	Beam B1	0.368	0.924
VI Floor Corner	Column C1	0.13	0.413
	Beam B37	0.137	0.913
	Beam B1	0.146	0.783

Table 3 shows DCR value of most affected elements at different floor corners. Since DCR values are within the limit, progressive collapse will not occur under fire load. Figure 9 and Figure 10 represents initial deformed shape of building after giving fire load at edge columns at ground floor level and fourth floor level respectively.

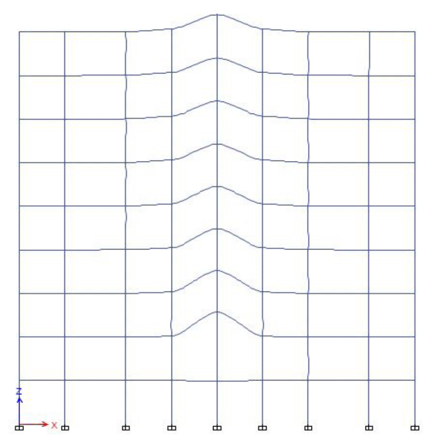

Figure 9. Deformed shape ofground floor edge column.

Table 4 shows that DCR value of most affected Floor Edge Members are also within the limit. So progressive collapse will not occur under fire load for these members. Figure 11 represents initial deformed shape of building after giving fire load at intermediate column at sixth floor. Figure 12 represents notation of intermediate column.

Table 5 shows that DCR value of most affected Floor intermediate Members are also within the limit. So progressive collapse will not occur under fire load for these members Figure 13 represents initial deformed shape of building after giving fire load at re-entrant column at ground floor. Figure 14 represents notation of re-entrant column.

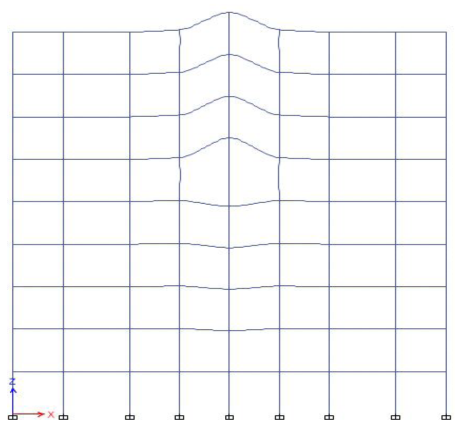

Figure 10. Deformed shape of fourth floor edge column.

Table 4. DCR Values of Specified Floor Edge Members

Location	Members	DCR Values	
		Before Fire	After Fire
Ground Floor	Beam B13	0.512	1.13
	Beam B40	0.322	1.352
	Beam B39	0.321	1.352
	Column C16	0.561	1.342
II Floor	Beam B13	0.502	1.028
	Beam B40	0.368	1.206
	Beam B39	0.367	1.19
	Column C16	0.341	0.873
IV Floor	Beam B13	0.382	0.9
	Beam B39	0.284	1.006
	Beam B40	0.242	1.006
	Column C16	0.237	0.641
VI Floor	Beam B13	0.163	0.766
	Beam B39	0.138	0.943
	Beam B40	0.138	0.943
	Column C16	0.137	0.413

Table 6 shows that DCR value of most affected Floor re-entrant members are also within the limit. So progressive collapse will not occur under fire load for these members also.

Table 5. DCR values of specified floor intermediate members

Location	Members	DCR Values	
		Before Fire	After Fire
Ground Floor	Beam B65	0.337	1.33
	Beam B20	0.448	1.463
	Beam B64	0.337	1.33
	Beam B19	0.43	1.463
	Column C24	0.515	1.342
II Floor	Beam B65	0.384	1.156
	Beam B20	0.435	1.179
	Beam B64	0.384	1.156
	Beam B19	0.43	1.178
	Column C24	0.442	0.9558
IV Floor	Beam B65	0.301	0.984
	Beam B20	0.333	1.04
	Beam B64	0.301	0.984
	Beam B19	0.333	1.04
	Column C24	0.346	0.683
VI Floor	Beam B65	0.155	0.928
	Beam B20	0.144	0.95
	Beam B64	0.155	0.949
	Beam B19	0.143	0.928
	Column C24	0.15	0.362

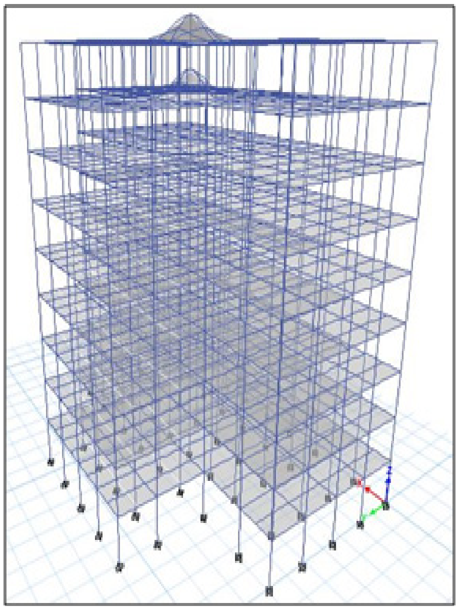

Figure 11. Deformed shape of sixth floor intermediate column.

Table 6. DCR values of specified floor re-entrant members

Location	Members	DCR Values	
		Before Fire	After Fire
Ground Floor	Beam B71	0.343	1.302
	Beam B77	0.447	1.072
	Beam B70	0.352	0.415
	Beam B12	0.448	1.204
	Column C15	0.562	1.05
Ii Floor	Beam B71	0.388	1.19
	Beam B77	0.464	1.08
	Beam B70	0.411	0.884
	Beam B12	0.44	1.133
	Column C15	0.425	0.889
Iv Floor	Beam B71	0.304	1.087
	Beam B77	0.383	0.969
	Beam B70	0.348	0.844
	Beam B12	0.334	1.071
	Column C15	0.331	0.76
VI Floor	Beam B71	0.156	0.935
	Beam B77	0.213	0.799
	Beam B70	0.226	0.716
	Beam B12	0.141	0.904
	Column C15	0.148	0.369

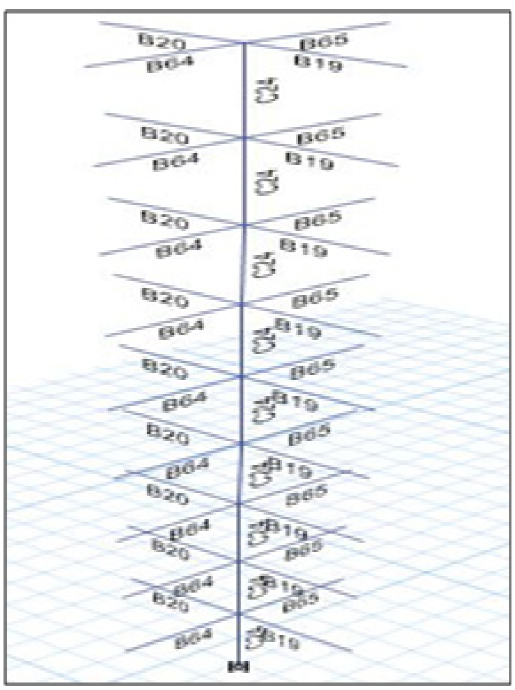

Figure 12. Notation of intermediate of the building.

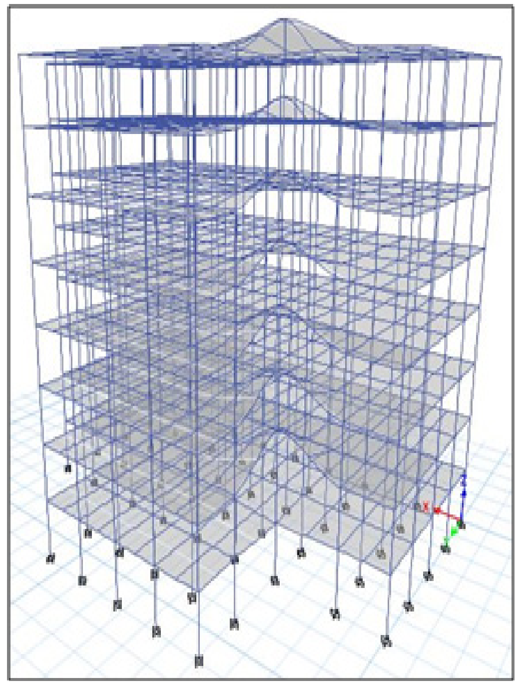

Figure 13. Deformed Shape of Ground Floor Re-entrant Corner Columns.

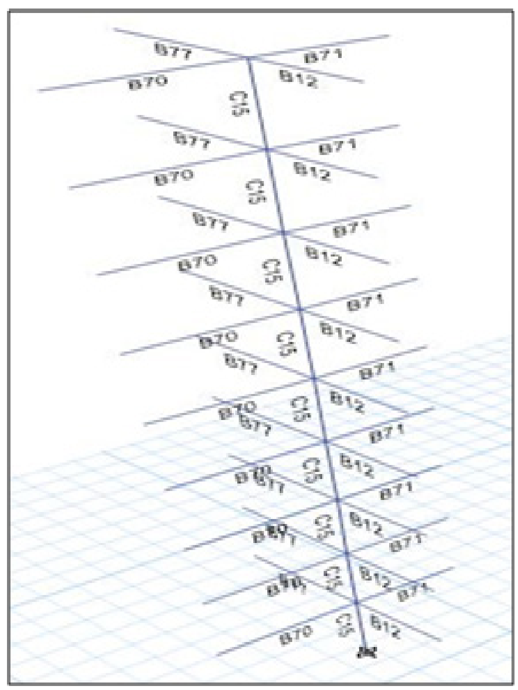

Figure 14. Notations of Re-entrant Column.

Table 7 shows comparison between intermediate and other located column at ground floor of building and Table 8 shows comparison between intermediate and other located beams at ground floor of building. From this, it has been inferred that members in the intermediate location were unsafe in the building and they are considered as critical members.

Table 7. Comparison between intermediate and other located columns

Column	Critical values in %
Re-entrant column	27.80
Corner column	16.36
Edge column	0

Table 8. Comparison between intermediate and other located beams

Beams	Critical values in %
Re-entrant beam	12.36
Corner beam	15.56
Edge beam	8.21

4. Conclusions

This study demonstrated the progressive collapse behavior of a steel frame building using ETABS software. In order to improve the progressive collapse resistance of structures in buildings and reduce the DCR values there are two possible options. One option is to use larger steel cross sections and the other option is to use more bracing. These two suggestions may lead to higher steel weight and may also cause more deformation after the columns affected by fire load. This paper shows that intermediate columnwas 27.8 % and 16.36% more critical when compared to re-entrant column and corner column respectively. Since DCR value of each element are within the limit 2 as per GSA guidelines, the building was safe against progressive collapse due to fire load.

5. References

1. Sasani M, Sagiroglu S. Progressive Collapse Resistance of Hotel San Diego. J Struct Eng. 2008 Mar; 134(3):478–88.
2. Izzuddin BA, Vlassis AG, Elghazouli AY, Nethercot DA. Progressive collapse of multi-storey buildings due to sudden column loss — Part I: Simplified assessment framework. Engineering Structures. 2008 Mar; 30(5):1308–18.
3. Izzuddin BA, Vlassis AG, Elghazouli AY, Nethercot DA. Progressive collapse of multi-storey buildings due to sudden column loss—Part II: Applicatiion. Engineering Structures. 2008; 30:1424–38.
4. Franssen, Marc J. SAFIR: A thermal/structural program for modeling structures under fire. Engineering Journal, AISC. 2005; 42(3):143–58.
5. Hosseini M, Fanaie N. Studying the Vulnerability of Steel Moment Resistant Frames Subjected to Progressive Collapse. Indian Journal of Science and Technology. 2014 Mar; 7(3):335–42.
6. Jiang J, Usmani A. Modeling of steel frame structures in Fire using OpenSees. Journal of Computers and Structures. 2013 Mar; 118:90–9.
7. United States General Services Administration (GSA). Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects. Washington, D.C. 2003.
8. Liew JYR. Survivability of steel frame structures subject to blast and fire. Journal of Constructional Steel Research. 2008 Aug; 64(7):854–66.
9. Hosseini M, Fanaie N, Yousefi AM. Studying the Vulnerability of Steel Moment Resistant Frames Subjected to Progressive Collapse. Indian Journal of Science and Technology. 2014 Mar; 7 (3):335–42.
10. Kirkpatrick S, MacNeill R, Smith J, Herrle K, Erekson M. Methodologies for Progressive Collapse Analysis. Structures Congress. 2009 Mar; 1–10.
11. McConnell J, Brown H. Evaluation of Missing Column Analyses in Progressive Collapse Design Codes. Structures Congress. 2009; 1–10.
12. Song B, Sezen H. Evaluation of an Existing Steel Frame Building against Progressive Collapse. Structures Congress. 2009; 1–8.
13. Liu R, Davison B, Tyas A. A Study of Progressive Collapse in Multi-Storey Steel Frames. Structures Congress. 2005; 1–9.
14. Mohamed OA, P.E. and M.ASCE. Progressive Collapse of Structures: Annotated Bibliography and Comparison of Codes and Standards. ASCE. 2006; 20(4):418–25.
15. Chen CH, Zhu YF, Yao Y, Huang Y, Long X. An evaluation method to predict progressive collapse resistance of steel frame structures. Journal of Constructional Steel Research. 2016; 122:238–50
16. Agarwal A, Varma AH. Fire induced progressive collapse of steel building structures: The role of interior gravity columns. Journal of Constructional Steel Research. 2014; 122:129–40.
17. Sun R, Huang Z, Burgess IW. Progressive collapse analysis of steel structures under fire conditions. Engineering Structures. 2011; 34:400–13.
18. Sun R, Burgess IW, Huang Z, Dong G. Progressive failure modelling and ductility demand of steel beam-to-column connections in fire. Engineering Structures. 2015; 89:66–78.