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Authors: Yukio Murakami, Deputy Manager, JFE Steel Corporation
Mitsumasa Fushimi, Manager, Nippon Steel Corp
Hiroaki Suzuki, Professor, University of Tsukuba

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Thermal Deformation Analyses of High-rise Steel Buildings

Y. Murakami[1], M. Fushimi[2], H. Suzuki[3]

1 Deputy Manager, Construction Engineering Service Dept, JFE Steel Corporation
2 Manager, Construction & Architectural Materials Development Division, Nippon Steel Corporation
3 Professor, Faculty of Engineering, The University of Tsukuba

Abstract

The collapse of the World Trade Center (WTC) towers can be cited as a warning that one local failure can trigger progressive collapse. The collapse of the WTC is regarded as a landmark event that alerted construction engineers to the importance of preventing progressive collapses in similar structures. After being struck, WTC1 and WTC2 remained standing for 102 minutes and 56 minutes, respectively, which was long enough to save many lives. Drawing from this event, the members of the Committee to Study the Redundancy of High-Rise Steel Buildings, which was established within the Japanese Society of Steel Construction in June 2002, commenced a research program to develop a progressive collapse control design method based on the seismic and fire-resistant design techniques of Japan and to investigate the redundancy of high-rise steel buildings. As a part of this research program, several thermal deformation analyses have been carried out on a high-rise steel structure of actual design. This paper presents the results of these studies.

Keywords: high-rise steel building, redundancy, thermal deformation analysis, collapse temperature, load redistribution

1. Introduction

The direct cause of the collapse of the World Trade Center towers on September 11, 2001 is attributable to column damage and to large-scale fires caused by the crashed airplanes. In spite of the fire damage, WTC1 and WTC2 stood for 102 minutes and 56 minutes, respectively. It is reported that the large plastic deformation capacity or load redistribution capacity inherent in steel structures led to many lives being saved. However, their collapse was a warning with regard to progressive collapse and a momentous event that gave notice to construction engineers about how important it is to suppress such collapse.

The Japan Iron and Steel Federation established the Committee to Study the Redundancy of High-Rise Steel Buildings in June 2002 within the Japanese Society of Steel Construction with the aim of enhancing the safety of high-rise steel buildings. As one link in a series of research programs conducted by the committee, thermal deformation analyses targeted at high-rise steel buildings were carried out. Generally, as countermeasures against fire, the external forces and the criteria associated with these forces are assumed, and then fire-resistant designs are carried out or fireproofing protections are determined to meet these criteria. This paper discusses the results of thermal deformation analyses targeting unassumed disturbances that exceed generally assumed criteria and at the same time discusses the effect of frame system and fire area on the redundancy of steel buildings. This paper also discusses a simple method to assess frame collapse, which is proposed based on thermal deformation analysis results, and its validity.

Yukio Murakami, Master Eng.
Deputy Manager, Construction Engineering Service Dept.,
JFE Steel Corporation
2-3, Uchisaiwai-cho 2-Chome, Chiyoda-ku, Tokyo 100-0011, Japan
Tel: 81-3-3597-4129, Fax: 81-3-3597-3825
e-mail: yu-murakami@jfe-steel.co.jp

2. Outline of Thermal Deformation Analysis

First, an outline of the thermal deformation analyses applied in the paper is described. The method used in this research is finite element analysis (SUZUKI, 1995), (SUZUKI et al, 2003) that takes into account high-temperature elasto-plasticity and the thermal expansion of steel products as well as the finite displacement of structural members. Meanwhile, because the research does not take into account the creep performance of steel or the local buckling of columns, actual frame behavior cannot be completely traced. But within the contextual range of this paper, we assume that the analysis demonstrates sufficient accuracy. In the following, particulars common to analyses are described.

2.1 Buildings Targeted for Analysis

The targeted structure was an office building with 27 stories above ground, a maximum height of about 130 m and column spans of 6.4 m. It was experimentally designed as a steel moment resistant frame (MRF) structure with a standard floor plan that was 57.6 m x 24.5 m. For the analyses, a plane frame was extracted from the building and a 9-span model was used as shown in Fig. 1.

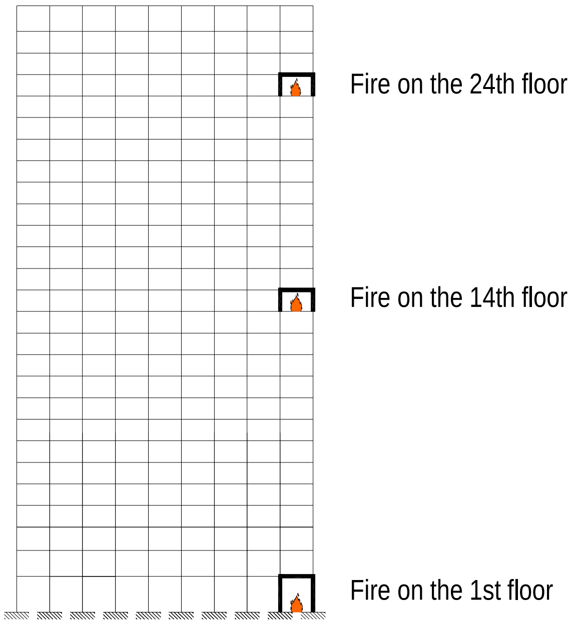


Fig. 1. Analysis Model and Fire Case (Floor Where Fire Occurs)

2.2 Particulars Regarding Structural Members

- Columns: Built-up box section columns were used with a material quality of SN490. Two kinds of sections were used: 750×750 mm and 650×650 mm, with plate thicknesses ranging from 25 mm to 40 mm.
- Beams: Built-up H-section beams are used with a material quality of SN490. The beam height is 850 mm for standard floors and 1,000~1,050 mm for low-rise sections. Flange plate thickness is 25~32 mm.

2.3 Axial Force Ratio of Columns

When the axial force ratio of columns is defined as the ratio of stationary vertical load to column yield strength, the ratio is 0.008~0.307. The ratio is smaller for side columns that show larger axial force deviation during horizontal loading.

3. Analytical Results

3.1 Effects of Fire Area

In this section, the effect of fire area on the redundancy of steel frames is described, based on the results of thermal deformation analyses in which fire areas are taken as parameters.

The parameters thus set assume the occurrence of three fires on each of three floors, as shown in Fig. 1—Case (fire on 24th floor) where fire occurs on the 24th floor above ground; Case (fire on 14th floor) where fire occurs on the 14th floor above ground; and Case (fire on 1st floor) where fire occurs on the lowest floor. Then, as shown in Fig. 2, the spread of fire area was taken as another parameter by assuming eight sets of cases for analysis—Side Case: Cases 1~4, a series of cases where areas around the edges of a floor are subjected to fire; Center Case: Cases 5~7, a series of cases where the area in the middle of a floor is subjected to fire; Case 8 where an entire floor is subjected to fire. As a result, a total of 24 cases were analyzed.

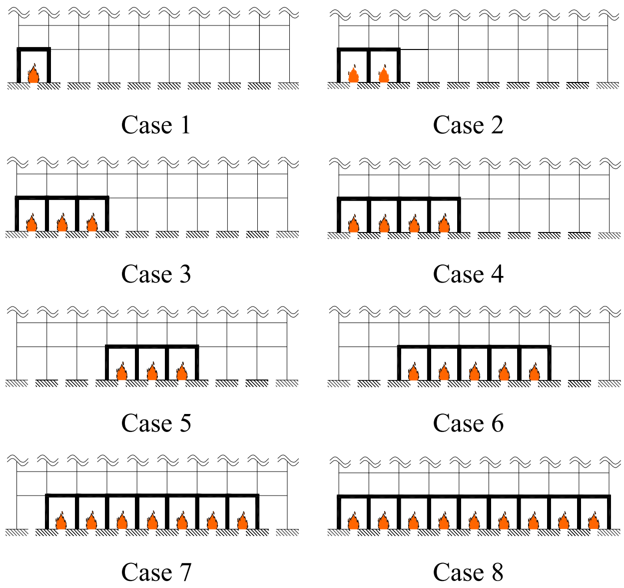


Fig. 2. Fire Case (Fire Area)

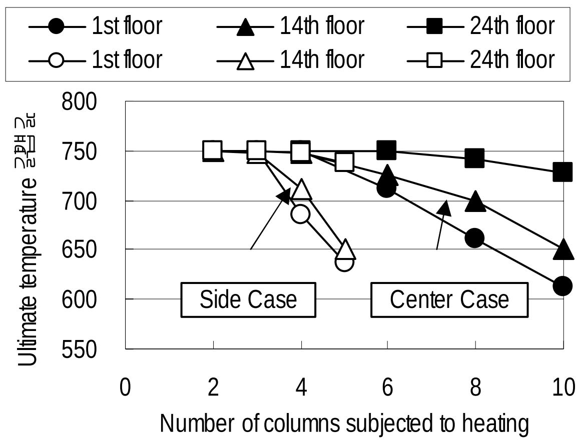
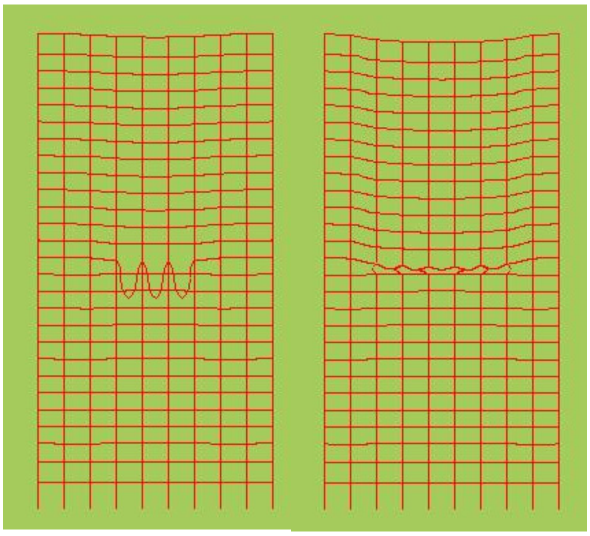


Fig. 3. Relation between Ultimate Temperature of Frame and Fire Area

Fig. 3 summarizes the relation between the ultimate temperature of frames (obtained from analytical results) and the number of columns subjected to heating by fires occurring in the area for each case according to the analysis parameters. In the figure, white markers show the series (Side Case) where areas at the edge of the floor are subjected to fire; the black markers indicate the series (Center Case) where the area in the middle of the floor is subjected to fire. Regarding the high-temperature characteristics of the steel products used in analysis, when the temperature of steel reaches 750ºC, the yield strength becomes zero; consequently, when the temperature of the member subjected to heating reaches 750ºC, the analysis discontinues. As a result, it can be considered that a building will not suffer entire collapse in cases where the ultimate temperature of frame reaches 750ºC. In Side Cases where a fire occurs on the lowest floor and more than 3 spans are subjected to fire or more than 4 columns are subjected to heating, there is the possibility that an entire building collapse might occur. For Center Cases where more than 5 spans are subjected to fire in the middle of a floor or more than 6 columns are subjected to heating, there is the possibility of an entire building collapse. Further, when comparing Side Cases to Center Cases, even at identical ultimate temperatures, collapse occurs in Side Cases with fewer columns subjected to heating than in Center Cases. From this, it is understood that entire collapse is more likely to occur in cases where fire occurs near the edges of a floor rather than in the middle, and that entire collapse is more likely to occur when side columns or corner columns are subjected to unassumed fire loads.

Fig. 4 shows the frame deformation (at 10 times actual displacement) that occurred near the time of collapse in Cases 5 and 6 and Fig. 5 shows the relation between the maximum deflection of beams subjected to heating and the temperature of those members (black circle shows analytical result for Case 5, and white circle for Case 6). Fig. 5 uses a solid line as a reference to show the limit value ($1^2/400d$: l=span length, d=beam depth) for beam deflection in the fire-resistant test prescribed in ISO834. As can be seen from Figs. 4 and 5, there is a high probability of entire collapse in Case 6; but, with regard to Case 5 (the frame is supposed to remain in local collapse), excess deflection of the beams was observed, and thus it is considered that beam collapse had occurred. Further, regarding other cases where the frame was supposed to remain in local collapse, beam deflection was prominent in every case; therefore, it can be regarded that local collapse is equal to beam collapse. Beam deflection in Case 5 far surpassed the criteria in common fire-resistant design, which means an example of behavior investigating when subjected to unassumed disturbances.



Case 5 Case 6

Fig. 4. Deformation Figure

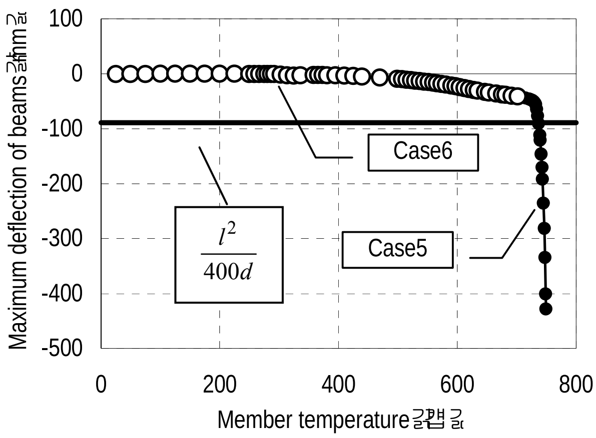


Fig. 5. Maximum Deflection of Beams Subjected to Heating

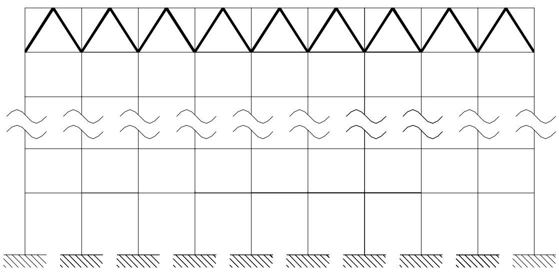


Fig. 6. Arrangement of Hat Trusses

3.2 Effects of Frame System

In this section, description is made of the effect of frame system on the redundancy of steel building frames, based on the results of thermal deformation analyses when frame system was set as a parameter.

There are two target frame systems: MRF structures and hat-truss structures. Analyses were made to examine the effect exerted by hat-truss. As for MRF structures, descriptions provided in the previous section are applied. Meanwhile, regarding hat-truss structures, as shown in Fig. 6, an analytical model is used in which only one diagonal member is added to a MRF structure, and the diagonal member is added only to the topmost floor. Further, taking into account that the purpose of this examination is to determine the effect of hat-truss, the dimensions of the respective members other than the diagonal member to be added, the loading conditions, etc. are to be identical to those used in the analytical model for MRF structures. The sectional dimensions of the diagonal member were calculated using a method to check load redistribution capacity, one of several simple methods to assess frame collapse that will be introduced in the following section, and on the assumption that any of the total axial force of columns subjected to fire heating that cannot be redistributed by the MRF structure will be redistributed by hat-truss. Regarding the fire area, this analysis assumed for the hat-truss structure the same 24 cases as in the previous section.

Fig. 7 shows the relation between the axial force ratio of interior columns subjected to fire heating and the ultimate temperature of frame for Cases 6 and 7 (white circle for the MRF structure and black circle for the hat-truss structure), which was selected as a representative relation after arrangement of the relations obtained for many cases. The figure shows the axial force ratio at room temperature before being subjected to heating. Regarding Case 6, the effect that the installation of the hat truss had on the ultimate temperature can clearly be observed. In the MRF structure, from examining the ultimate temperature, it is understood that the frame led to entire collapse in every case. On the other hand, in the hat-truss structure, because the temperature of members subjected to heating reached 750°C in every case of the axial force ratio (≒0.1~0.3) thus analyzed, it is understood that the frame remains in local collapse. Meanwhile, regarding Case 7, it was observed that installation of a hat truss had only a slight effect on the ultimate temperature; even in the hat-truss structure, the ultimate temperature reached about 715°C in the vicinity of an axial force ratio of 0.25 (fire on the 14th floor) and reached about 660°C in the vicinity of an axial force ratio of 0.3 (fire on the 1st floor); therefore, it is understood that the frame led to entire collapse in both cases. The following can be cited as the cause for entire collapse: while load redistribution capacity was improved by the addition of a hat truss, columns at room temperatures that were not subjected to fire heating, in particular columns adjacent to the area of the fire, could not bear the redistributed axial force in terms of structural stability after redistribution.

Fig. 8 shows an example of frame deformation (at 10 times actual displacement) that occurred around the time of collapse in Case 6 of a fire on the 14th floor; the figure shows a comparison between MRF and hat-truss structures. As can be seen from the deformation conditions in Fig. 8, it is assumed that entire collapse occurred in the MRF structure (an ultimate temperature of 726°C was obtained as the result of analysis); but, in the hat-truss structure where the temperature of members heated by fire reached 750°C, it is understood that the frame remained in local collapse. From this, it is understood that the effect of a hat truss on improving the load redistribution capacity of frames as a whole is demonstrated.

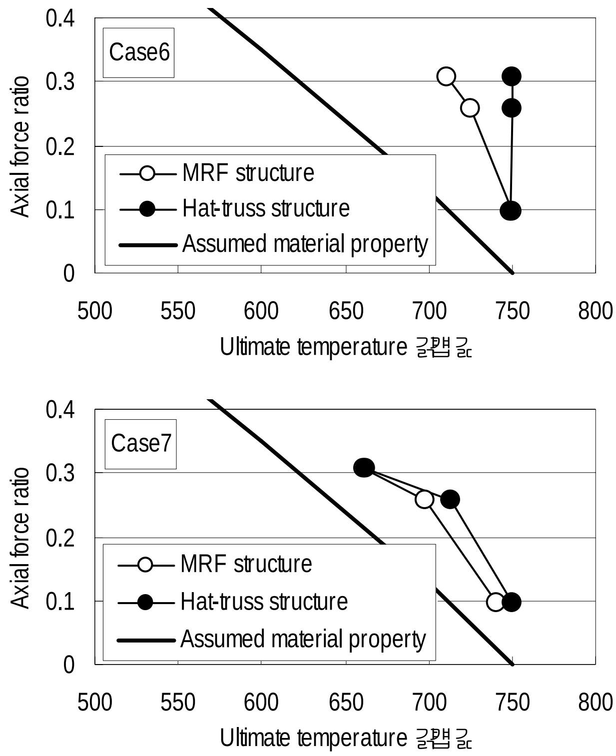


Fig. 7. Relation between Axial Force Ratio of Columns and Ultimate Temperature

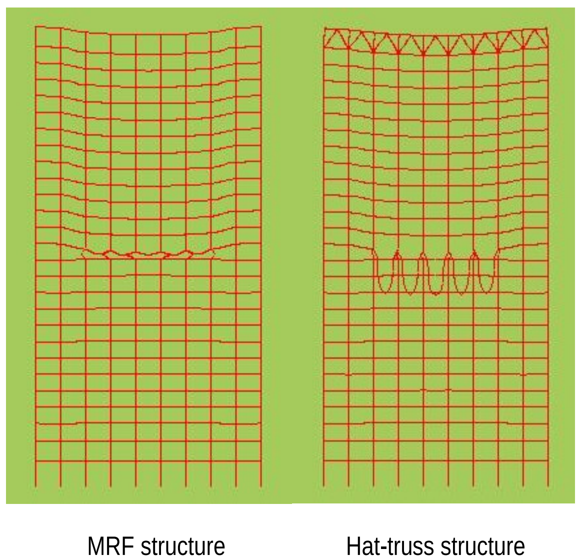
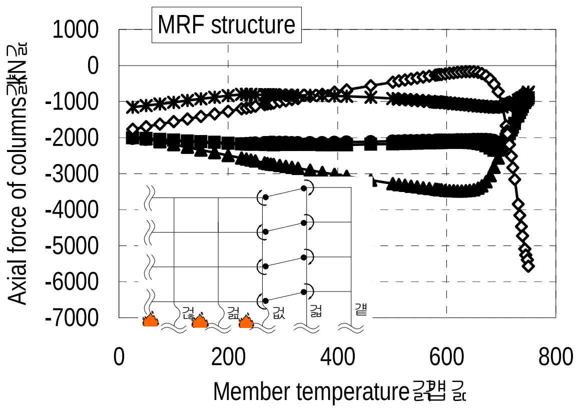


Fig. 8. Deformation Figure (Case 6: Fire at 14th Floor)

Fig. 9 shows the representative relation between the column axial force on the first floor and the temperature of members subjected to fire in Case 6 of a fire on the 24th floor (upper figure for MRF structure, lower figure for hat-truss structure). It is understood from the figure that the pattern of change in column axial force that accompanied the rise in member temperatures is nearly identical for both MRF and hat-truss structures.

When examining Fig. 9, it is understood that uniform load redistribution is not always made to columns at room temperature (marked with ◇ or ⋆ in the figure), regardless of whether or not a hat truss has been installed. This trend is commonly observed in fire analysis cases other than the case in Fig. 9. Accordingly, it can be said that redistributed axial force is borne at room temperature mainly by the columns (marked with ◇ in Fig. 9) that are adjacent to the fire area in both MRF and hat-truss structures.



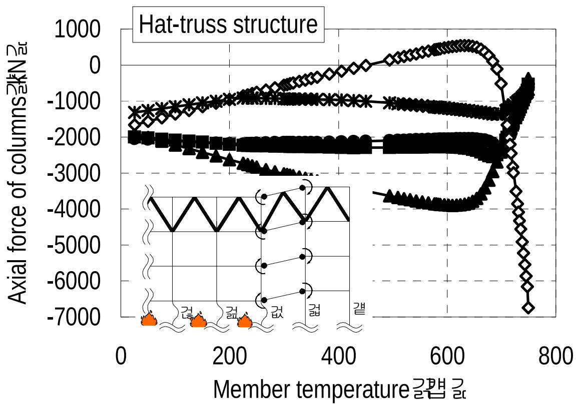
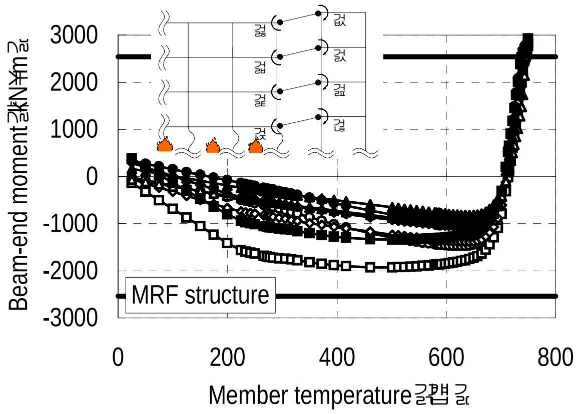


Fig. 9. Relation between Axial Force of Columns and Member Temperature (Case 6: Fire at 24th Floor)

Fig. 10 shows as a representative example the relation between the moment of room-temperature beam end (assumed to contribute to load redistribution) on the floors above the fire floor and the temperature of members subjected to fire heating in Case 6 of the fire on the 24th floor. The upper figure relates to the MRF structure, and the lower figure to the hat-truss structure. The solid bold line in the figure shows the calculated value of the plastic moments of beams at room temperature. In the figure, the code of moments is illustrated assuming that the direction (clockwise) shown in the figure is set as plus. In Fig. 10, the beam-end moment initially increases to the minus side due to thermal expansion of the columns subjected to heating. The moment reverses itself in the vicinity of the temperature at which the columns subjected to heating buckle and then increases to the plus side. It is understood that the process whereby the increase of moments to the plus side after reversing itself indicates the condition for load redistribution. This trend is identical for both MRF and hat-truss structures. In any case, it is understood that the moment resistance of beam ends on the floors above the fire floor works towards load redistribution, and accordingly it is understood that the strength and deformation performance of joints (ex. beam-to-column connections) become important. Further, in frames where beam ends have pin connections, it is considered difficult to distribute load using beams at room temperature on the floor above the fire floor, and thus it is necessary to install a hat truss, belt truss or other load redistribution frame so as to take into account the redundancy of steel buildings.



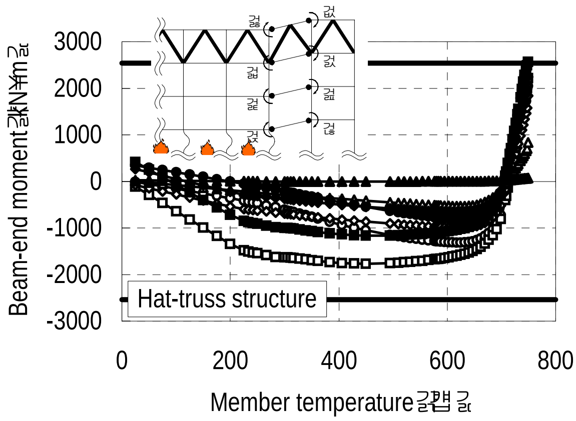


Fig. 10. Relation between Beam-end Moment and Member Temperature (Case 6: Fire at 24th Floor)

4. Simple Method to Assess Frame Collapse

In this section, a simple method for assessing frame collapse (SUZUKI et al, 2003) is proposed that is based on the aforementioned results of analysis.

The procedure of this simple method to assess frame collapse is as follows:

1) Check load redistribution capacity and

2) Check strength after load redistribution

In the following, these respective checking methods are described.

[Check Load Redistribution Capacity]

The load redistribution capacity is checked by assuming the following items obtained from the results of analysis and the collapse mechanism shown in Fig. 11.

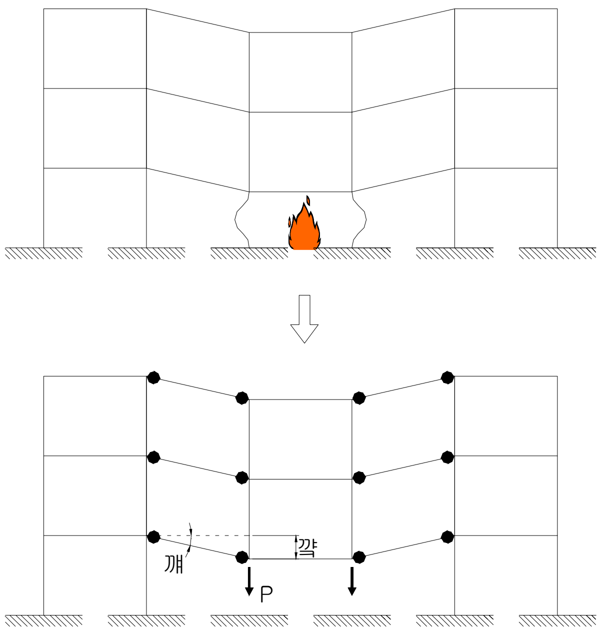


Fig. 11. Assumption of Collapse Mechanism

- In cases where column members were subjected to fire heating and lost load bearing capacity due to high-temperature buckling, the axial force previously borne by the columns is redistributed to other members at room temperature.
- The beams adjacent to the fire area and on the floors above the fire floor contribute to the redistribution of axial force.
- Because of the simple collapse assessment method that is applied, the residual strength of columns after high-temperature buckling is not taken into account.

When the MRF structure and the fire area shown in Fig. 11 are taken as an example, P in the figure indicates the axial force that was borne by the columns causing high-temperature buckling (that are subjected to fire heating) before the occurrence of fire. The axial force P together with the virtual displacement δ constructs the external work. Further, the plastic moment of room-temperature beams and the virtual rotation angle θ construct the internal work. Because the condition under which the frame does not collapse is external work<internal work, the following equation holds.

$$\sum_{j=1}^{n} P_j \cdot \delta < \sum_{i=1}^{N} {}_bM_{pi} \cdot \theta_i \tag{1}$$

Where

n: Number of columns that lose load bearing capacity due to high-temperature buckling (that are subjected to heating)

P_j: Axial force of columns that cannot bear load due to high-temperature buckling

N: Number of plastic hinges of beams that contribute to load redistribution

${}_bM_{pi}$: Plastic moment at room temperature of beams that contribute to load redistribution

Further, when each span (l) is assumed to be equal, Equation (1) could be transformed to Equation (2).

$$\sum_{j=1}^{n} P_j \cdot < \sum_{i=1}^{N} \frac{{}_bM_{pi}}{l} \tag{2}$$

When a certain fire occurred and if Equation (2) is satisfied, it is understood that load redistribution is possible with the beams on the floors above the fire floor.

[Check Strength after Load Redistribution]

If the axial force of the columns that lose load redistribution capacity due to high-temperature buckling cannot be borne by all the columns at room temperature, the frame will collapse. Accordingly, it is necessary to assess whether or not the redistributed load (column axial force) can be borne by the columns at room temperature. As shown in Fig. 12, all distributed axial forces are to be borne by the columns at room temperature adjacent to the fire area and the axial force after load redistribution is to be borne by the columns at room temperature, for which the following equation must be satisfied.

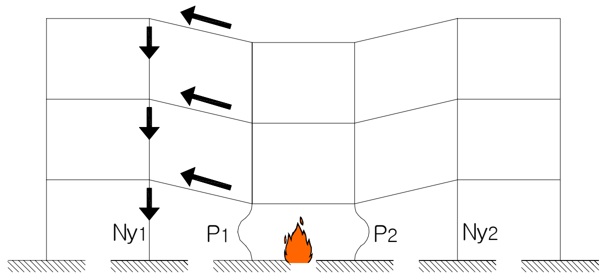


Fig. 12. Flow of Load Redistribution

$$\sum_{j=1}^{n} P_j \cdot < \sum_{i=1}^{N} Nyi \tag{3}$$

Where

P_j: Axial force borne at room temperature by columns that lose load bearing capacity due to buckling (that are subjected to heating)

N_{yi}: Yield strength of room-temperature columns adjacent to columns that lose load bearing capacity due to buckling (that are subjected to heating)

n: Number of columns that lose load bearing capacity due to buckling (that are subjected to heating)

N: Number of columns adjacent to columns that lose load bearing capacity due to buckling (that are subjected to heating)
(In cases where fire occurs at the end of a floor: N=1; in cases where fire occurs in the middle of a floor: N=2)

Table 1 shows each analyzed case in which the above concepts are incorporated. The following are assumed to examine Table 1: It is understood that the frame remains in local collapse in cases where the temperature of members subjected to heating reaches 750°C and that entire collapse occurs in cases where the ultimate temperature of frame is less than 750°C. These are shown in the "assumed collapse type" in Table 1.

The following can be understood from assessments of the MRF structure, shown in Table 1.

1) Regarding Case 1, both redistribution capacity and strength after redistribution can be adequately checked employing the proposed simple collapse assessment method ($\Sigma N < \Sigma Q$: Remaining in local collapse)—which also accords with the analysis results.
2) Except for Case 1, in cases (Cases 2, 5 and 6 of a fire on the 24th floor) where the frame remains in local collapse in the analysis results, the maximum value for the ratio of redistribution capacity ($\Sigma N < \Sigma Q$) is 2.04 (Case 6 of the fire at 24th floor); and, it is assumed that load redistribution is possible up to the level at which $\Sigma N < \Sigma Q$ is approximately 2. The attributable reasons to be considered include the effect (increase of ΣQ) of strain hardening on the strength of the beam (the load redistribution member) and the effect (increase of ΣQ) that the shear resistance of the beam web (which is ignored in the proposed assessment method) has on redistribution capacity.
3) Of the cases where the frame remains in local collapse, it is only Case 2 of a fire on the 1st floor where strength after redistribution cannot be adequately examined using the proposed simple collapse assessment method ($\Sigma N < \Sigma Q$). However, in this case, because $\Sigma N / N_y$ is 1.21, it is assumed that entire collapse does not occur even after load redistribution up to the level of approximately $\Sigma N / N_y$ =1.2. The attributable reasons to be considered include the effect (increase of N_y) of strain hardening on the strength of columns at room temperature and the load redistribution (decrease of ΣN) to room-temperature columns other than those columns that are assumed to bear the axial force after redistribution.
4) When 2) and 3) above are taken into account and the reason for the collapse in cases where entire collapse occurred in the analysis results is assumed, the following results are obtained.

● Fire on the 24th floor: Cases 3, 4 and 7
→Shortage of load redistribution capacity
● Fire on the 14th floor: Cases 3, 4, 6 and 7
→Shortage of strength after stress redistribution
● Fire on the 1st floor: Cases 3, 4, 6 and 7
→Shortage of strength after stress redistribution

The following can be understood from the assessment results as to the hat-truss structure, shown in Table 1.

5) Regarding cases (fire on the 24th floor: Cases3, 4 and 7) where entire collapse is assumed to occur in the MRF structure due to load distribution capacity, the frame remains in local collapse in the hat-truss structure as a result of the improvement in load redistribution capacity provided by the hat truss.
6) While entire collapse occurs in the MRF structure in cases other than those above, there is one case (Case 6 of fire on the 14th and 1st floors) where the frame remains in local collapse due to the installation of a hat truss. This may be because the axial force to be redistributed by use of the hat truss was distributed, to a certain extent, to room-temperature columns other than those columns adjacent to the fire area.

When the analysis results above are examined and safety as a design equation is assessed, it is understood that the proposed simple collapse assessment method is valid.

5. Conclusion

Thermal deformation analyses were made on high-rise steel buildings using parameters set for fire area and frame system. As a result, the following became clear.

1) The ultimate temperature of frame is largely affected by the number of columns and the structural section subjected to heating; and, the cases where side or corner columns are subjected to heating, rather than center columns, become critical.
2) Installation of a hat truss is effective in improving load redistribution capacity as a countermeasure against entire frame collapse.
3) If fire is confined to a certain area, the frame does not suffer entire collapse even when unassumed events such as the failure of fire protection occur.
4) In order to prevent entire frame collapse by means of load redistribution, it is necessary to secure sufficient strength of joints and high deformation capacity of members.

In addition, a simple collapse assessment method for frames was proposed based on thermal deformation analysis results, and its validity was confirmed.

Table 1. Calculation Results

Fire case	Assumed fire floor	MRF structure		Examination of redistribution capacity			Examination of strength after redistribution			Hat-truss structure	
		Ultimate temperature	Assumed collapse type	ΣN (t)	ΣQ (t)	ΣN / ΣQ	ΣN (t)	Ny (t)	ΣN / Ny	Ultimate temperature	Assumed collapse type
Case1	24F	750℃	Local	318	324	0.98	318	1,876	0.17	750℃	Local
	14F	750℃	Local	927	1,138	0.81	927	1,685	0.55	750℃	Local
	1F	750℃	Local	1,803	2,430	0.74	1,803	2,540	0.71	750℃	Local
Case2	24F	750℃	Local	538	324	1.66	538	1,876	0.29	750℃	Local
	14F	750℃	Local	1,579	1,138	1.39	1,579	1,685	0.94	750℃	Local
	1F	750℃	Local	3,075	2,430	1.27	3,075	2,540	1.21	750℃	Local
Case3	24F	748℃	Entire	759	324	2.35	759	1,876	0.40	750℃	Local
	14F	712℃	Entire	2,231	1,138	1.96	2,231	1,685	1.32	736℃	Entire
	1F	684℃	Entire	4,346	2,430	1.79	4,346	2,540	1.71	697℃	Entire
Case4	24F	738℃	Entire	979	324	3.03	979	1,876	0.52	750℃	Local
	14F	650℃	Entire	2,884	1,138	2.53	2,884	1,685	1.71	684℃	Entire
	1F	636℃	Entire	5,618	2,430	2.31	5,618	2,540	2.21	638℃	Entire
Case5	24F	750℃	Local	882	647	1.36	882	3,752	0.24	750℃	Local
	14F	750℃	Local	2,610	2,277	1.15	2,610	3,369	0.77	750℃	Local
	1F	750℃	Local	5,086	4,861	1.05	5,086	5,079	1.00	750℃	Local
Case6	24F	750℃	Local	1,323	647	2.04	1,323	3,752	0.35	750℃	Local
	14F	726℃	Entire	3,915	2,277	1.72	3,915	3,369	1.16	750℃	Local
	1F	711℃	Entire	7,629	4,861	1.57	7,629	5,079	1.50	750℃	Local
Case7	24F	741℃	Entire	1,764	647	2.73	1,764	3,999	0.44	750℃	Local
	14F	698℃	Entire	5,220	2,277	2.29	5,220	3,645	1.43	714℃	Entire
	1F	661℃	Entire	10,171	4,861	2.09	10,171	6,558	1.55	662℃	Entire
Case8	24F	728℃	Entire	1,959	647	3.03	1,959	–	–	729℃	Entire
	14F	651℃	Entire	5,768	2,277	2.53	5,768	–	–	651℃	Entire
	1F	613℃	Entire	11,235	4,861	2.31	11,235	–	–	613℃	Entire

ΣN : Accumulated axial force borne at room temperature by columns subjected to fire

ΣQ : $= 2¥ \sum_{i=1}^{n} {}_b M_p / l$ 갉Case1 ? 4갉?? 갌?? 2¥ 2¥ $\sum_{i=1}^{n} {}_b M_p / l$ 갉Case5 ? 8갉

n: Number of hinges, ${}_bM_p$: Plastic moment of beams at room temperature, l : Span

Ny: Yield strength of room-temperature columns adjacent to fire area

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