Title: **A Study on the Redundancy of High-Rise Steel Buildings Due to the Effect of Induced Heat and Loss of Vertical Structural Members**

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A STUDY ON THE REDUNDANCY OF HIGH-RISE STEEL BUILDINGS DUE TO THE EFFECT OF INDUCED HEAT AND LOSS OF VERTICAL STRUCTURAL MEMBERS

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Abstract

This paper presents a theoretical investigation of the redundancy capacity of steel frames due to heat induced by fire and loss of columns due to impacts from explosions and plane crash. A non-linear analysis was used to estimate the redundancy capacity through examination of the axial load utilisation ratio of columns. Investigations were carried out on moment resistant frames, braced frame with hysteretic dampers and outrigger truss system. It is concluded that steel frames designed with load-carrying capacity joints can withstand large-scale fire. For any loss of vertical load bearing members to contribute to progressive collapse, it was found that the axial load utilisation ratio in members was found to be greater than 0.25. Therefore, for the prevention of progressive collapse, the axial forces in vertical load bearing members must be held below this limit.

Keywords: high-rise steel building, redundancy, progressive collapse, structural stability, collapse temperature, critical axial force ratio

1. Introduction

The work described herein is part of a larger investigation into the studies of redundancy in high-rise steel structures, described in a preceding article of this conference. The studies conducted above categorised progressive collapse into two modes according to the nature of structural failure.

Failure mode 1 illustrates the propagation of the story collapse to the lower stories of the building. This mode represents a case when a collapse occurs on one particular story (called initial collapse story, which can be more than one story), leading to stories above it to fall perpendicularly in a mass, due to failure of columns on the stories below it to sustain the impacts. Mode 2 represents progressive collapse that occur on stories above the initial collapse story. If the vertical load bearingelement(s) of a story is lost and redistribution of forces to adjacent elements take place and able to sustain the load then, progressive collapse will not occur. This particular mode occurred to the exterior columns of the WTC towers. In the case of the WTC towers, it is believed that failure Mode 2 occurred first and followed by Mode 2. The WTC case is analyzed without clear distinction between the occurrence of these two modes as the modes occurred consecutively.

This paper examine the conditions that are required to prevent a Mode 2 progressive collapse due to loss of structural members possibly through reduction in strength by fires or due to accidents or explosions. Numerical analyses were conducted to evaluate the redundancy capacity of high-rise steel structures through evaluation of the axial load utilization ratios of columns.

Investigations were carried out for moment resistant frames namely, frames with hat-bracing, frames with core bracing and super frame structure, Fig 1. Moment resistant frames equipped with dampers (for reducing seismic responses) were analysed statically and the stability of frames being examined against local collapse. The theoretical analyses simulate a condition where main members are lost due to an explosion and high temperatures induced by fire.

2. Prevention of Progressive Collapse

2.1 Prevention of Mode 2 Progressive Collapse

Investigations were carried out for moment resistant frames namely, frames with hat-bracing, frames with core bracing and super frame structure, Fig 1. Moment resistant frames equipped with dampers (for reducing seismic responses) were analysed statically and the stability of frames being examined against local collapse. The theoretical analyses simulate a condition where main members are lost due to an explosion and high temperatures induced by fire.

2.1.1 Structural Stability due to Loss of Structural Members

The analyses on the behavior of steel structural frames in which columns and beams have been damaged, lost or buckled at high temperatures are presented. The steel frames were analysed by conventional statistic analysis without the main members which have been assumed lost.

In the case of steel frames exposed to fire, the collapse temperatures of the frames due to the heat are calculated in order to examine the structural stability at elevated temperatures.

Furumura et al (1981) have shown that the plasticity of members caused by temperature rise does not lead directly to the collapse of the entire frame structure. They have also shown that elevated temperatures, far exceeding the allowable temperature stated in the Building Standard Law of Japan, do not harm the structural soundness of steel frame structures. On the other hand, the collapse of the WTC towers indicates a possibility that, if a very large portion of the frame structure is exposed to high temperatures, the frame structure can eventually become totally and destructively unstable (Bazant, et. al, 2002). Considering this point then there is a conceivable need to study structural stability at elevated temperatures to prevent progressive collapse against fire load.

The study described herein considers the transition of frame instability from one condition to another condition of static balance. In a stable condition, where the temperature of members rise without causing buckling of columns, the behavior of frames can be analyzed through load control. In the unstable condition, the temperature rise is suspended until the transition stage is completed, and thereafter the frame is analyzed through displacement control. This approach enables solving of problems for structural stability at elevated temperatures and the ultimate state or collapse temperatures of the frame due to fire. Previous works on dynamic theories governing the behavior of frames due to buckling of columns at elevated temperatures were presented by Suzuki (1995, 2002).

The following sections present a study on the axial load utilization ratio of members due to lost of members and heat generated by fire at elevated temperatures, their respective state of stability and recommendations on structural forms that can prevent Mode 2 progressive collapse.

2.1.2 Numerical Analysis Model and Method of Analysis

Numerical analyses on a 10 story buildings based on four types of frames namely, moment resistant frame, moment resistant frame with hat-bracing and with hat-core-bracing and a super frame structure (Fig. 1). Buckling-restraint bracing were employed in the study as these are increasingly used in Japan recently. The members are considered jointed with load-carrying capacity joints.

The process of collapse was analysed for four assumed configurations with members losses as shown in Fig.1 together with six fire conditions as illustrated in Fig. 2. Fig. 1 shows the configuration of member losses in the structures. The same configuration are applied to other types of frames. Fig. 2 shows the locations of fire for analysis conducted on moment resistant frame. The same configurations are applied to other types of frames.

In Fig. 1, the designed static loading is proportionally distributed, and axial load utilization ratios of columns at the time of collapse are calculated for each case of analysis. In cases for fire analysis where the axial load utilization ratio of the interior column is $\overline{p} = N / A\sigma_y = 0.225, 0.3, 0.35, 0.4, 0.45$, the collapse temperature has to be calculated for each case. For cases assuming fires or losses of main members, the slenderness ratio of columns is set at $\lambda = l / i = 25.5$ and the column load utilization ratio of exterior columns is set at 1/2 of that of the interior columns. Also, uniformly distributed loadings on beams are set by the following normalized load, $\tilde{q}$.

$$\tilde{q} = \frac{ql^2}{16M_p} \tag{1}$$

where l and M_p represent length of span and full plastic bending moment of the beam, respectively. $\tilde{q}$ indicates the load factor with respect to the collapse load of the fix-supported beams under a uniform load, and $\tilde{q} = 0.15$ when $\overline{p} = 0.45$. This means that a load equivalent to 15% of the ultimate collapse load under uniform load is normally placed on the beam.

The amount of bracing used is the same for the moment resistant frame with hat-and-core-bracing and the superframe structure. The steel materials are all JIS G 3136 SN400 ($\sigma_y = 235N / mm^2$). Table 1 shows the cross-sectional dimensions of the members and load conditions.

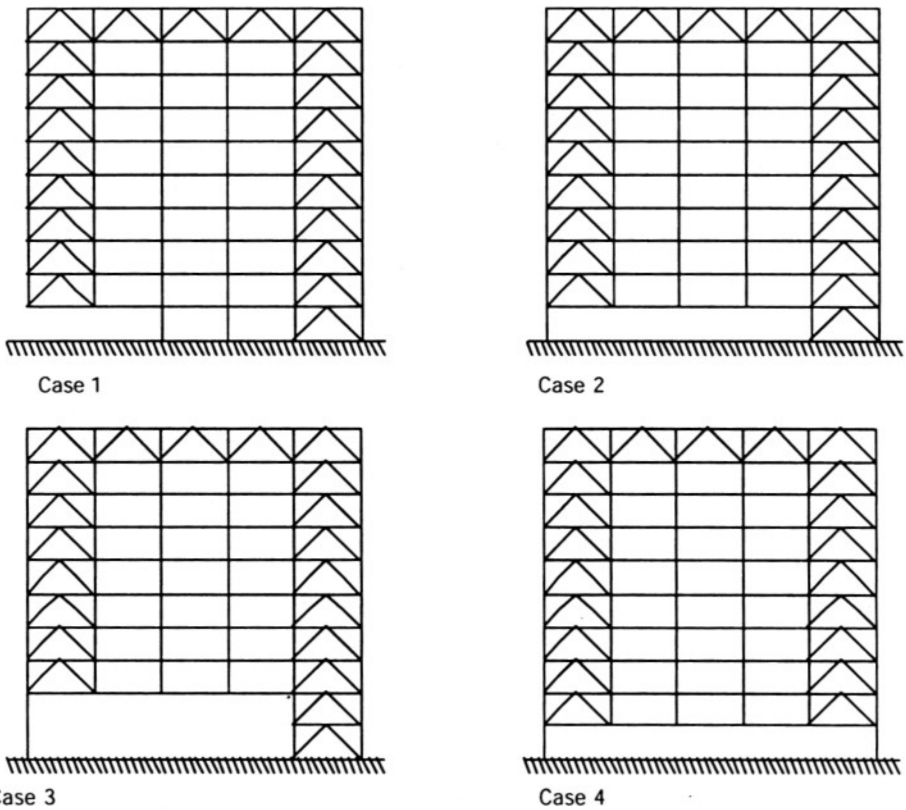


Fig. 1 Analytical models of frames with member losses

In calculating the collapse temperature, an assumed fire condition with constantly increasing temperature for the portion within the bold boundaries shown in Fig. 1 is used. The remaining portions are held at room temperature. The rate of thermal expansion of steel, α, is assumed not to vary with the temperature but to remain constant at $\alpha = 12 \times 10^{-6}$ / °C.

Table 1: Sections and load conditions

Section of frame	
•Beam	•H-596x199x10x15•for all structure•
•Column	••-350x350x16•all•
•Hat-bracing	•H-200x200x8x12•all•
•Hat-and-core bracing	•H-200x200x10x15•Hat-and-core-bracing• H-200x200x8x12•Superframe•
Load conditions	
•Normalized loading on beam	• $\tilde{q} = \dfrac{ql^2}{16M_p} = 0.15$
•Column force ratio(The lowest story)	• $\overline{p} = N / A\sigma_y = 0.225, 0.3, 0.35, 0.45$

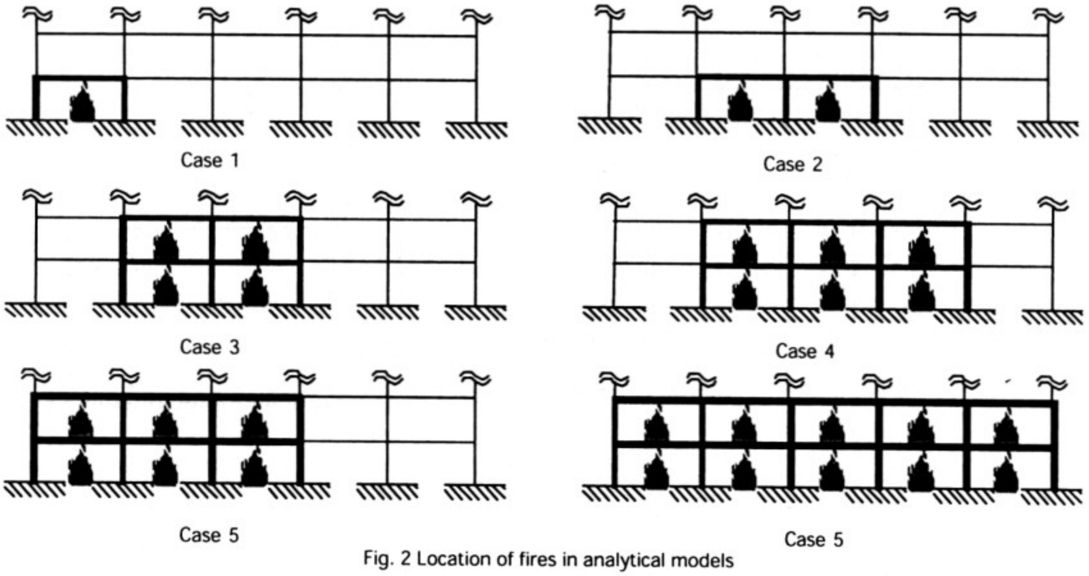


Fig. 2 Location of fires in analytical models

The collapse temperature due to fire is calculated using the Finite Element Method incorporating the elasto-plasticity, linear expansion of the steel material at elevated temperatures, and finite displacements of frames. The analyses assumes the yield strength and stress-strain relationships (Furumura 1981) as given in Table 2. In any typical case, the strength of the steel declines as temperature rises, and the stress-strain relationships vary accordingly. Fig. 3 shows the stress-strain relationships of a structural steel, JIS G 3136 SN400, at different temperatures. It may be noted from the figure that the strength of the steel begins to reduce as the temperature exceeds 400°C and continuously reduce to 1/3 and 1/7 of the strength at 600°C and 700°C, respectively. Accordingly, at higher temperature the steel columns will buckle under lower axial load than at room temperature.

This analysis defines the total collapse of frames for two conditions:

(i) whereby the vertical displacement was calculated for the story /whole story and
(ii) where the analysis is unable to reach convergence and therefore unable to redistribute the load. These will be discussed in detailed in section 2.1.3

Table 2 Assumption for formulating yield strength and stress-strain relationship of SN400

$$\sigma(\varepsilon, T) = \max\left\{\sigma^{(1)}(\varepsilon, T), \sigma^{(2)}(\varepsilon, T)\right\}$$

$$\sigma^{(1)}(\varepsilon, T) = \min\left\{E_t \cdot \varepsilon, \sigma_{yt}\right\}$$

$$\sigma^{(2)}(\varepsilon, T) = f^{(1)}(\varepsilon, T) + f^{(2)}(\varepsilon, T)$$

$$f^{(1)}(\varepsilon, T) = \frac{E_{lt} \cdot \varepsilon}{\left\{1 + \left(\frac{E_{lt}}{\sigma_{ot}} \cdot \varepsilon\right)^n\right\}^{\frac{1}{n}}} \quad , \quad f^{(2)}(\varepsilon, T) = \frac{E_{pt} \cdot \varepsilon}{\sqrt{1 + \left(\frac{\varepsilon}{\varepsilon_2}\right)^2}}$$

$$E_{lt} = E_t - E_{pt}$$

$$E_t = \left(1.0 - 0.905 \cdot 10^{-6} \cdot T^2\right) \cdot E_{RT} \qquad E_{RT} = 2100t\ /\ cm^2$$

$$\sigma_{yt} = \left(1.001 - 3.592 \cdot 10^{-6} \cdot T^2\right) \cdot \sigma_{yRT} \qquad \sigma_{yRT} = 2.4t\ /\ cm^2$$

$$E_{pt} = Line\ \text{connecting}\left(0℃, 50.0t\ /\ cm^2\right), \left(400℃, 50.0t\ /\ cm^2\right), \left(600℃, 5.0t\ /\ cm^2\right) and \left(850℃, 0.0t\ /\ cm^2\right)$$

$$\sigma_{ot} = \begin{cases} where T \le 600℃ \\ \left(0.759 + 1.933 \cdot 10^{-4} \cdot T - 5.944 \cdot 10^{-6} \cdot T^2 + 2.179 \cdot 10^{-8} \cdot T^3 - 2.305 \cdot 10^{-11} \cdot T^4\right) \cdot \sigma_{yRT} \\ where T > 600℃ \\ \quad Line\ \text{connecting}\left(600℃, \sigma_{ot}(600℃)\right) and \left(850℃, 0.0t\ /\ cm^2\right) \end{cases}$$

$$\varepsilon_2 = 0.05 \qquad n_t = 1.7$$

Where, E_t•Young's modulus at T• •$_{yt}$•Yield strength at T•
E_{pt}•Plastic Modulus at T• •$_{ot}$•Reference plastic stress at T•
n_t•Shape parameter at T•

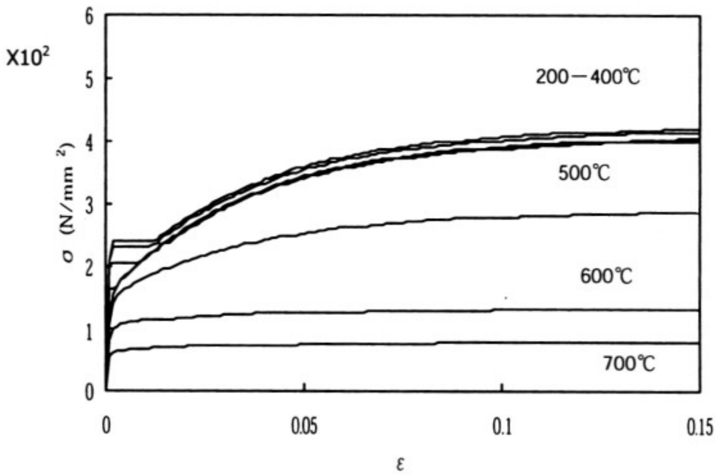


Fig. 3 Stress-strain relationship of SN400 at different temperature

2.1.3 Result of Analysis and Discussion

Figs. 4 and 5 show the results of the analysis for fire cases 1-5. The variation of the collapse temperature with the scale of fire for the models are shown in Fig. 4. The results displayed are for two cases of axial load utilization ratio of the interior column, $\overline{p} = 0.225, 0.45$.

It is evident that the collapse temperature is about the same magnitude when the load utilisation ratio of the interior column is $\overline{p} = 0.225$, irrespective of the frame types. In the case where the axial load utilization of the interior column is $\overline{p} = 0.45$ i.e. under relatively high axial force the collapse temperatures are higher for frame structures equipped with seismic members compared to moment resistant frame or for localized fire such as in Cases 1 to 4, or where exterior columns do not yield. However, in Cases 4-2 and 5 where the fire is severe causing the exterior columns to yield and buckle, it appears that the collapse temperature does not vary with the frame structure.

Fig. 5 shows the fire collapse temperature of each fire for the different frame structure. When the scale of fire increases, the collapse temperature will be higher for structures equipped with seismic members than for the moment resistant frame. Figure 6 shows the relationship between the axial load utilization ratio of the interior column and collapse temperature for each frame structure derived from these studies. The bold line in Fig 6 is a series of normalized numbers obtained from the stress values to provide one percent strain on the stress-strain curve for each assumed temperature divided by the yield stress ($235N/mm^2$) at room temperature. The distance between the curves and this bold line for each frame structure may be considered to indicate the degree of redundancy of each structure. It may be noted that frame structures equipped with seismic members and those frames with smaller axial load utilization ratios display greater allowances. The studies indicated that a frame structure with greater axial load utilization ratios and frame structures equipped with fewer seismic members are more in danger of total collapse as illustrated by the distortion of structures in Fig. 7.

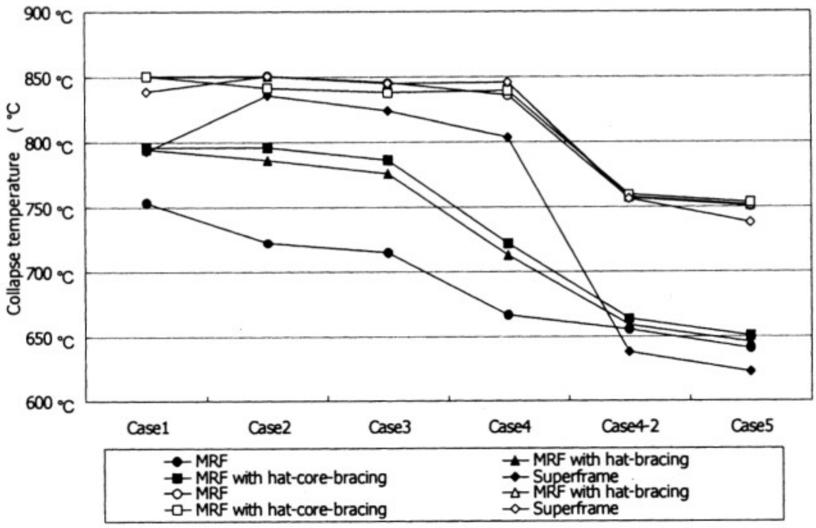


Fig. 4 Variation of collapse temperatures with scale of fire

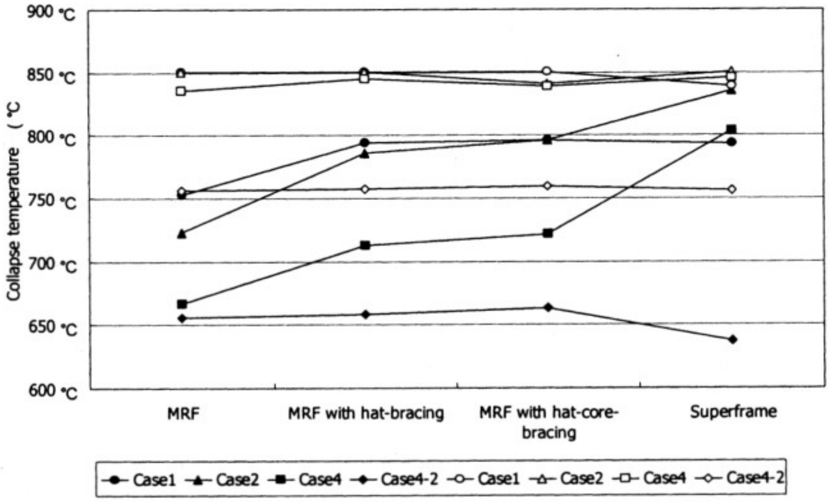


Fig. 5 Variation of collapse temperature for fire cases and frame types

Fig. 8 shows the distortion of structures due to the losses of major members i.e. three interior columns of the first story (Case 2) at which the axial load utilization is $\overline{p} = 0.45$. In the case of moment resistant frame, the beam of the second story collapse to the ground, meaning that the collapse of the first story leads to the total collapse. By contrast, despite the loss of interior columns and bracing members, the collapse of the first story does not lead to total collapse. Fig. 9 and 10 show the values of axial load utilization ratio for frame structures and cases for member losses. It is evident that the collapse axial force ratio is higher for the superframe structure than for other structures. The moment resistant frame can withstand the loss of a considerable number of vertical supporting members, provided that the axial load utilization is not greater than 0.3. In the case of moment resistant frame with hat-bracing and with hat-and-core-bracing, the collapse axial force ratio is larger than the moment resistant frame, thus indicating higher resistance of the former structure against losses of vertical load supporting members.

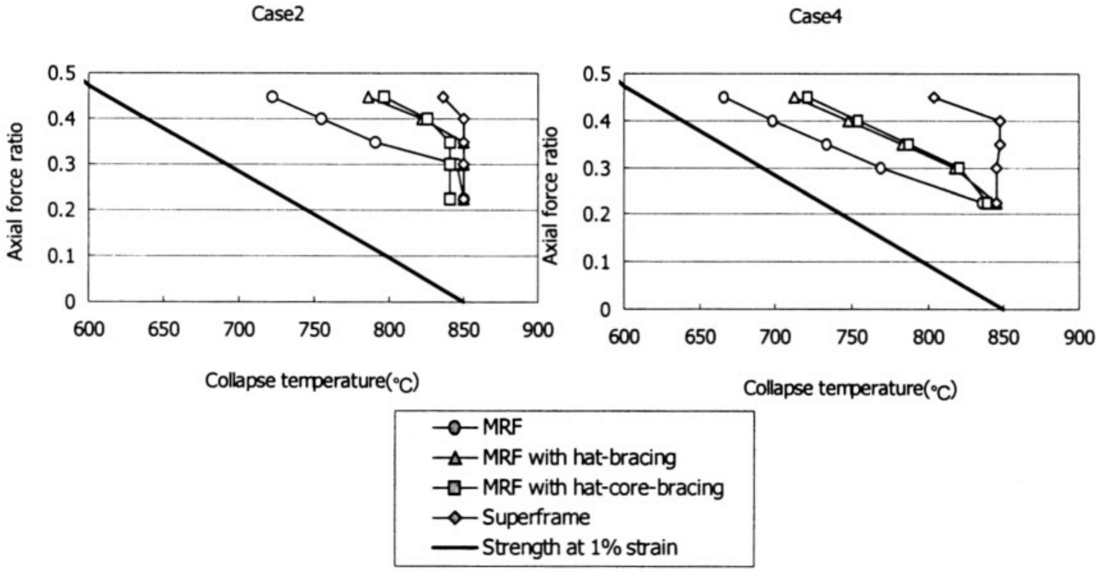
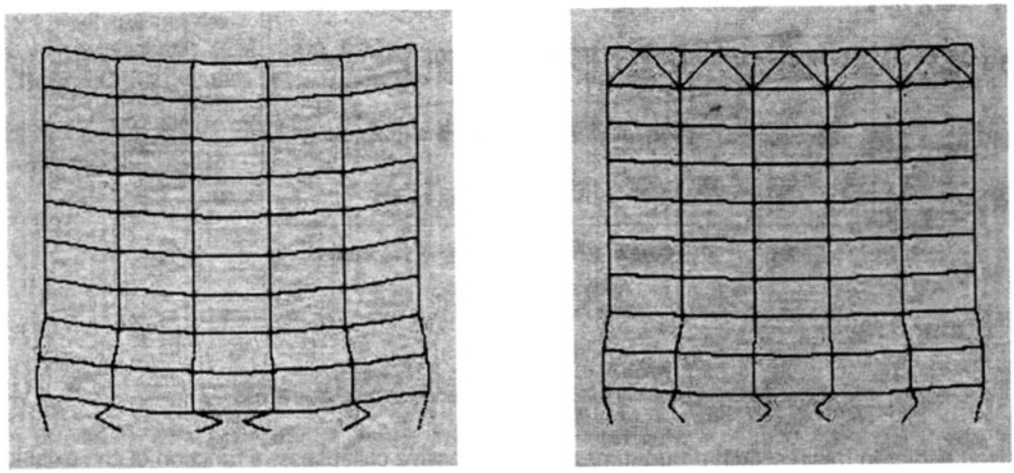


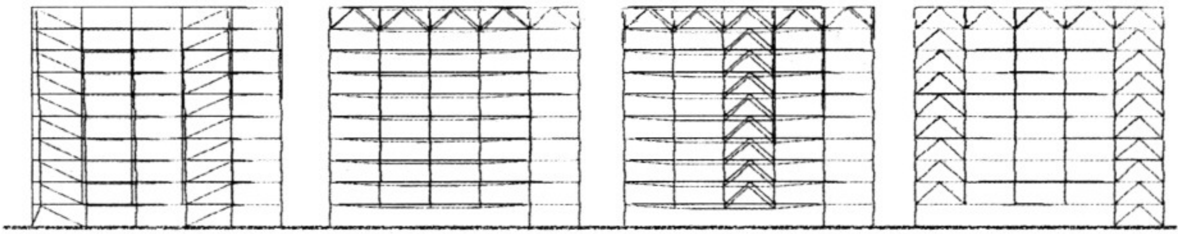
Fig. 6 Relationship between axial load utilization ratio of interior column and collapse temperature



(a) MRF

(b) MRF with hat-bracing

Fig. 7 Distorted structures (fire case4: $\overline{p} = 0.45$)



(a)MRF (b)MRF with hat-bracing (c)MRF with hat-core bracing (d)Superframe

Fig. 8 Distorted structures (less of major members case 2: $\overline{p} = 0.45$)

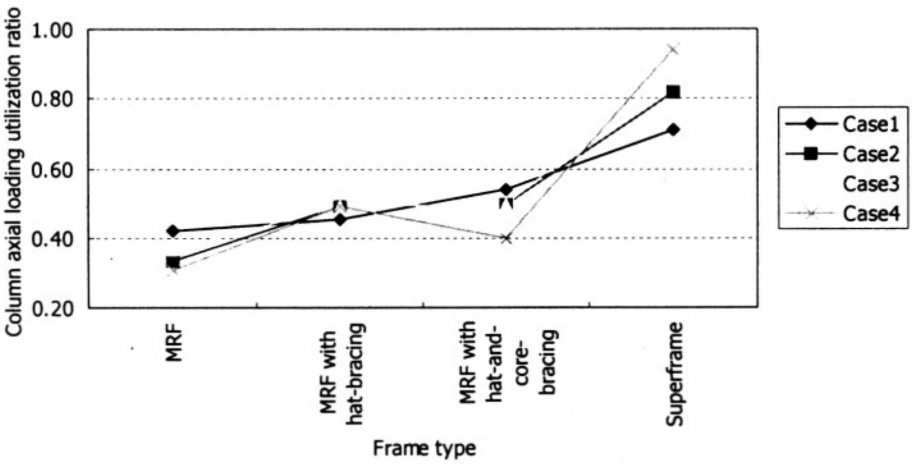


Fig. 9 Collapse axial load utilization for frame types

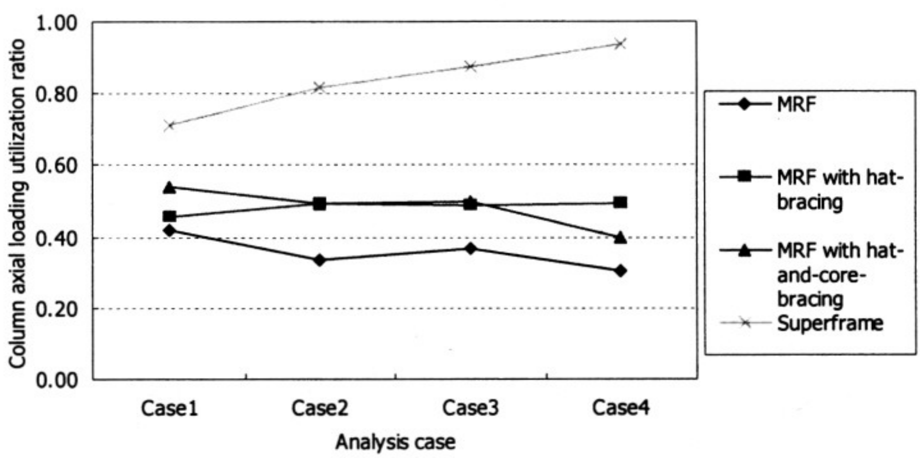


Fig. 10 Collapse axial load utilization ratio for case of analysis

As derived earlier in this research programme, the progressive collapse is a function of the axial load utilization ratio of column under ordinary loading hence, a prediction whether collapse in Mode 1 or Mode 2 will occur can be made through analysis of axial load utilization ratio.

The analysis presented herein uses hypothetical models based on load-carrying capacity joints, and uniform cross-sections of members throughout the building height in studying stability against the progressive collapse. These analyses were conducted as a preliminary study. Hereafter, the committee plans to study the effects of load-carrying capacity joints, for high-rise buildings constructed in accordance to the Seismic Code of the Building Standards in Japan.

Conclusions

An investigation was conducted to examine the structural redundancy of structural steel frames due to the effects of fire and member losses due to accidental impacts and explosions. The axial load utilization ratio of column under ordinary load was used as a parameter to evaluate cases and evaluation of mode for progressive collapse. Further studies are necessary to produce detailed results for practical applications. Nevertheless, the following conclusions have been obtained from the preliminary studies carried out.

(1) Steel frame structures using load-carrying capacity joints can withstand large-scale fires and loss of vertical load resistant members, if the axial load utilization ratio of columns is held low.

(2) Superframe structures designed for earthquake resistance has a higher stability against local collapse than moment resistant frames, when the strength of member is reduced by fires or structural members are lost.

(3) The axial load utilization ratio of column under ordinary load can be an effective parameter in the study for prevention of progressive collapses. As concluded from this study, the axial load utilization ratio of column under ordinary load can be limited to approximately 0.25 for prevention of progressive collapse.

Acknowledgements

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