Study on High-rise Steel Building Structure that Excels in Redundancy

by

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ABSTRACT

The collapse of the WTC towers may be taken to be an alert that a local failure can trigger a progressive collapse, and a landmark event that alerted the construction engineers to the importance of preventing progressive collapses of similar structures. Prevention of progressive collapse requires development of design technologies for frames that excel in redundancy. The Japan Iron and Steel Federation and the Japanese Society of Steel Construction established June this year the committee on "Study on Redundancy of High-Rise Steel Buildings". This paper discusses the high-rise steel building structures that excel in redundancy, and outlines the themes of this research project and then investigates analytically the resistance of the steel frames against heat induced by fire and loss of columns by accident such as an explosion and a impact of a crash considering structural parameters which are axial load utilization ratio of columns and structural type of frames, which are moment resistant frame, braced frame by hysteretic dampers and frame by hysteretic dampers and outrigger truss system. A non-linear analysis is conducted to estimate redundancy of various types of frames considering heat induced by fire and loss of columns. Based on the findings of the analysis, this paper advanced that 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 under 0.25, for prevention of progressive collapses.

KEYWORDS: collapse temperature, critical axial force ratio, high-rise steel building, redundancy, progressive collapse

1. INTRODUCTION

The direct causes for the collapse of the World Trade Center (WTC) Towers on September 11, 2001 were damages on the columns caused by aircraft impact, and large-scale fires. WTC1 and WTC2 kept standing 102 and 56 minutes after the impacts, respectively, during which periods many lives were saved. The great plastic deformation capacity or transferring load capacity of the steel structures reportedly saved a large number of human lives[1]. From the above the WTC towers might be considered to have some redundancy. However, the collapse of the WTC towers may be taken to be an alert that a local failure can trigger a progressive collapse, and a landmark event that alerted the construction engineers to the importance of preventing progressive collapses of similar structures.

Prevention of progressive collapse requires development of design technologies for frames that excel in redundancy. The Japan Iron and Steel Federation and the Japanese Society of Steel Construction established June this year the committee on “Study on Redundancy of High-Rise Steel Buildings”. For the purposes of improving safety of high-rise buildings, the committee has begun working on the following two themes. These are: (1) a study on collapse-control design, on the bases of Japan’s earthquake resistant and fire prevention technologies, and (2) quantification of redundancy of Japan’s high-rise steel building structures and a work to propose frames that excel in redundancy.

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2. OUTLINE OF RESEARCH THEMES OF THE COMMITTEE ON "REDUNDANCY OF HIGH-RISE STEEL BUILDINGS"

2.1 High-rise Steel Building Structure of High Redundancy

The WTC towers, both of them, could resist the intentionally created aircraft impacts and kept standing for some time, sparing time that permitted many people to evacuate. In this sense, the WTC towers should be credited with saving many lives. This indicates that the structures of two WTC towers had some redundancy. It is also true that the collapses that occurred some time afterward killed a large number of lives, including firemen who rushed to the towers for the trapped people's rescue. The process in which the collapse of the story that suffered from the aircraft impact lead to the collapse of the entire building may be interpreted as indicating that the structure did not have sufficient redundancy to prevent the total collapse.

Against such a background, necessity of designing buildings that could resist total collapse even in the case of accidental load actions caused as a result of an act of terrorism or accident is being discussed by various concerned parties. This committee considers that structures designed assuming increased external force by a large margin to resist total collapse are different from what the true redundant structure should be, though such structures in effect could be redundant structures. One example of redundant structures is a structure in which its redundant strength in aerodynamic design or seismic design, and its redundant strength at load carrying-capacity joints, both for the stationary load, combine to provide redundancy against the accidental load action created by an aircraft crash or explosion. In other words, the concept of redundant structure should evaluate redundant strength inherent in the subject structure, or should realize high-rise steel building structures excellent in redundancy with a proper selection of structural features, arrangement of members and proportioning of sections at a minimum of additional cost.

2.2 Objectives and Research Theme

The research committee has the following two objectives; namely, (1) Development of collapse control design standards or design recommendations based on the seismic and fire resistant design technologies of Japan, to prevent the progressive collapse; (2) Presentation of a recommendation for construction of high-rise steel building structures with adequate redundancy, based on the collapse control design standards or design recommendations.

The committee has established Structure Design Working Group and Fire-resistant Design Working Group to study the following themes on the basis of the results of studies shown in the preceding chapter. The study themes comprise those common to both groups and those unique to each group.

< Common themes >

1. Establishment of clear definition of the terms "local collapse" and "progressive collapse" as design terminology to be considered in design work
2. Evaluation of difference in redundancy according to the frame type (moment resistant frame, moment resistant frame with hat-bracing, moment resistant frame with hat-and-core-bracing, superframe structure, cf. Fig.1)
3. Proper redundancy of steel structure building from the viewpoint of risk management
4. Development of collapse control design recommendations or guidelines

< Structure Design Working Group >

5. Evaluation of resistance to the story collapse and to the loss of main structures to prevent total collapse
6. Evaluation of the effect of the load-carrying-capacity joint and full penetration welding of joints connecting columns on the above-mentioned stability of frames
7. Clarification of the behaviors of members under explosion and impact load conditions
8. Approach to redundancy of such space structures as shell, dome, etc.

< Fire-resistant Design Working Group >

9. Accuracy improvement of thermal analysis through acquisition of loaded heat test data on the cross-sections of large and thick members used for high-rise buildings, and numerical

simulation based on such data

10. Study on fire-resistive specification and construction of floor, and systems and methods for connecting the floor and the main building structure
11. Evaluation of fire resistance efficiency of high strength bolted connections
12. Study on technical measures for improvement of fire resistant redundancy (use of CFT structure, fire resistant steel and fire partition, etc.)

The Structure Design Working Group will divide the progressive collapse mode of seismically and aerodynamically designed high-rise steel structure building into two types as mentioned below, and analyze stability of frames to each collapse mode. The analysis will use the region for which the main member is conceived lost, strength and deformation capacity of joints as parameters, and obtain the critical axial force ratio of columns at the time of collapse for each type of frames. Further, the working group will clarify behaviors of members under explosion and impact load conditions, referring to the high-speed-loading test on the members and joints connecting columns and beams after the Hyogo-Ken-Nanbu Earthquake 1995 (Southern Hyogo Prefecture Earthquake). In this way, the working group will have clarified stability of members and joints against accidental loads. The working group will also study approaches to redundancy of such large space structure as shell, dome, etc., one of the main structure themes of this committee.

The Fire-resistant Design Working Group is promoting a study on the degree of redundancy (patterns of damage) of high-rise steel structure buildings of seismic and fire-resistant design, with the burning area as parameter. More specifically, the study aims to clarify structural stability at elevated temperatures suited to each frame type. In this study accuracy of evaluation on high temperature resistance of steel parts is important. Data on loaded heating tests on the cross-sections of thick steel members used for high-rise steel structure buildings are virtually nonexistent. Accordingly, the working group is conducting a loaded heat test on the standard heating temperature-time curve/hydrocarbon curve of ISO834 to acquire data. The working group intends to increase the accuracy of thermal analysis by conducting numerical simulations using these data.

The working group considers the effects of the fire-resistive specifications of floor, type of joint connecting the floor with the main structure, joint structure between columns and beams upon the redundancy of total structure very great. The working group studies the fire-resisting performance of the floors and high strength bolted connection in friction type used in Japan (rigid joint), and also presents recommendation on upgrading of redundancy.

The above study should identify members and connecting parts constituting key elements to securing good redundancy from both structural viewpoint and fire-resisting viewpoint. In effect, these key elements should be preferentially protected. The working group presents recommendations for designs of steel structure buildings excellent in redundancy, the objective the committee is expected to achieve, after some study on technical measures with suppressed incremental costs such as use of CFT columns, fire resistant steel, methods for installing fire partitions.

3. PREVENTION OF PROGRESSIVE COLLAPSE

3.1 Mode of Progressive Collapse

The committee classifies the modes of progressive collapses into two modes as shown in Figure 1, and is promoting researches into conditions for preventing progressive collapses for each of the modes. Mode 1 of Fig. 2 illustrates propagation of the story collapse to the lower stories. This mode represents a case when a collapse occurs on a certain story (called initial collapse story, can be more than one story), all stories above this story fall perpendicularly in a mass, and the vertical load resistant members (columns) of the story or stories just beneath cannot sustain the impact. Mode 2 represents progressive collapses that occur on stories above the initial collapse story. If a vertical element of the load bearing strength of the initial collapse story is lost and transferring of load occurs, and if the members adjacent to the lost vertical resistance can sustain the rearranged loads, the progressive collapse of this mode does not occur. This mode occurred to the exterior columns of the WTC towers. In the case of the WTC towers, a collapse of Mode 1 occurred following a collapse of Mode 2. The WTC tower case is being evaluated without clear distinction of these two modes, because these two modes of collapses occurred consecutively. In studying conditions for preventing collapses of these two modes from occurring, it is necessary to study such conditions, with the two modes clearly segregated.

3.2 Prevention of Mode 1 Progressive Collapse

Regarding prevention of Mode 1 progressive collapse, the following study will be done, considering robustness of Japan's seismically designed high-rise buildings, referring to the studies by Bažant et al [3].

For the sake of simplicity, let it be assumed here that a certain given story (initial collapse story) of a building collapses, and all stories above the initial collapse story fall in a rigid body. The question is whether or not the vertical load resistant members (columns) of the weakest story below the initial collapse story can withstand the load.

The story collapse occurs as a result of a loss of strength of vertical load resistant members (columns) by a fire, or a loss of vertical load resistant members (columns) themselves by an accident like explosion, buckling of vertical load resistant members (columns) as a result of a loss of such buckling-confining members as beams. In case the stories above do not fall in a rigid body but the dead load and the live load of the story immediately above the collapse story alone rest on the floor of the collapse story, the load carrying capacity of the collapse story should be examined. If the load carrying capacity is insufficient, or the oneness of the floor system is not secured, the floor of the collapse story falls and rests directly on the floor of the story immediately below the collapse story. In this case, the progressive collapse cannot be prevented. Therefore, it is assumed here that the ultimate load carrying capacity should be enough to support the load equivalent to those of two stories at least, and oneness of the floor is secured.

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Let M and g represent the mass of the entire portion above the initial collapse story and acceleration of gravity, and the load equivalent to Mg is placed on the collapse story before collapse occurs. The collapse story begins collapsing when the strength of its vertical load resistant members (columns) becomes equal to Mg. Let the average axial force ratio of columns of the story just beneath the initial collapse story be p, the vertical strength, Py, of the story just beneath the initial collapse story is given by Equation (1).

$$P_y = \frac{Mg}{p} \qquad (1)$$

If the length of the vertical load resistant members (columns) of the story just below the initial collapse story is decreased by, Δ_{max}, the decrease in potential energy, ΔE_h, is given by the following equation.

$$\Delta E_h = Mg(n \cdot h + \Delta \max) \qquad (2)$$

where n and h represent the number of collapse stories and story height, respectively.

The amount of energy that can be absorbed by the entire vertical load resistant members (columns) of the building, E_A, is given by the following equation.

$$E_A = Mg(\Delta_{\max} - \frac{1}{2}\Delta_y) + P_y(\Delta_{\max} - \frac{1}{2}\Delta_y) + E_e \quad (3)$$

where Δ_{max} represents the maximum displacement of the vertical load resistant members (columns) to the limit of maintaining the yield strength, Δ_y represents displacement of the deformation of the elastic limit of the vertical load resistant member (column), and E_e represents the elastic strain energy of vertical load resistant members (columns) of stories other than the weakest story. The first term of the right-hand side of Equation (3) represents the amount of energy absorbed before the initial collapse story begins collapsing, the second term the amount of energy the story just beneath can absorb. The critical deformation of the initial collapse story is assumed to be equal to the maximum deformation of the story just beneath the collapse floor, or $\Delta_{\max}$. If the following inequality holds, the story immediately beneath the collapse story does not collapse.

$$\Delta E_h < E_A \quad (4)$$

By substituting Equations (2) and (3), and, further, Equation (1) into the above equation, the following equation is obtained.

$$Mg(n \cdot h + \Delta \max) < Mg(\Delta_{\max} - \frac{1}{2}\Delta_y) + P_y(\Delta_{\max} - \frac{1}{2}\Delta_y) + E_e = Mg(\Delta_{\max} - \frac{1}{2}\Delta_y)\cdot(1+\frac{1}{p}) + E_e \quad (5)$$

By solving the above equation for $\Delta_{\max}$, the following equation is obtained.

$$\Delta_{\max} > ph[n + (1+\frac{1}{p})\frac{\Delta_y}{2h} - \frac{E_e}{Mgh}] \quad (6)$$

In Equation (5), let n = 1 assuming one story alone to collapse, and neglecting the term, Δ_y/h, because $\Delta_y/h \ll 1$, the following equation holds.

$$\Delta_{\max} > ph(1-\frac{E_e}{Mgh}) \quad (7)$$

This inequality gives the required capacity for deformation of the vertical load resistant members (columns) required to prevent the progressive collapse. The first term in the parenthesis of the above equation is the product of the average axial force ratio of columns of the initial collapse story, p, and the story height (falling distance), h. The second term is the ratio of the elastic strain energy of the vertical load resistant members (columns) of stories other than the weakest one to the decremental portion of the potential energy of the upper stories equivalent to the distance of fall. According to Bažant(2002) et al., this value is about 1/8.4. If the representative value of p for Japan's high-rise buildings is taken to be 0.2, the following definition of $\Delta_{\max}$ is obtained from Equation 7.

$$\Delta_{\max} > 0.2 \cdot (1-1/8.4)h \approx h/5.7 \quad (8)$$

Equation (8), indicates that the required capacity for deformation of the vertical load resistant members (columns) of the story just beneath the collapse story is 1/5.7 the story height. If plastic deformation is not limited to the story just beneath the collapse story but is extended to more than one story or considering the plastic deformation of seismic members such as hysteretic dampers, the required capacity for deformation becomes smaller than this value. To prevent the Mode 1 progressive collapse, the committee will first study the present states of Japan's high-rise steel structure buildings, and second see whether it is possible to make these buildings satisfy Equation (7), and conditions that achieve this at minimum costs.

3.3 Prevention of Mode 2 Progressive Collapse

The following pages compare a moment resistant frame equipped with hysteretic dampers installed for the purpose of reducing responses to seismic movements and a moment resistant frame, in the stability of entire frame structure against a local collapse, by means of static analysis. The study includes a static analysis simulating a condition in which main members are lost by an explosion and an analysis on stability of the frame at high temperatures, assuming the frame to be heated by a fire.

3.3.1 Structural Stability after Loss of Main Members or at the Time of Fire

This sub section presents an analysis of behaviors of the entire steel frame structure, of which columns have been lost, or buckled at high temperatures, of steel frame structures, of which such main members as columns and beams are lost or being exposed to heat of a fire. Here, steel frame structures, of which such main members as columns and beams have been lost are subjected to an ordinary statistical analysis against the vertical load, without the main members that have been assumed lost.

Regarding the steel frame structure exposed to the heat of a fire, collapse temperatures of the frames heated by the fire are calculated to study the structural stability at elevated temperatures of the steel frame structure.

Furumura et al[4]. have shown that plastification of members caused by temperature rise does not lead directly to the collapse of the entire frame structure. They also have shown that elevated temperatures far exceeding the allowable temperature in the Building Standard Law of Japan do not harm the structural soundness of steel frame structures. While 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[3]. Considering that, there is conceivably real need to study structural stability at elevated temperatures to prevent the progressive collapse against fire load.

This study regards the unstable process of the frame represents a process of snap through from a certain condition of static balance to another condition. In a stable process where the temperature of members can rise without causing buckling of columns, the behavior of frames can be analyzed from the standpoint of load control. While in an unstable process, temperature rise is suspended until the process of snap through is finished, the frame is analyzed from the standpoint of displacement control. This approach has made it possible to solve a number of problems on structural stability at elevated temperatures, and the ultimate states or collapse temperatures of the frame at the time of fires have been clarified. Regarding theories on dynamics governing behaviors of frames after column buckling at elevated temperatures, reference should be made to the literature[5][6].

The following section presents a study on the axial load utilization ratio that can alleviate the unstable conditions of frame caused by the loss of major members and instability of frames at elevated temperatures, and also structural forms that can prevent the Mode 2 progressive collapses.

3.3.2 Numerical Analysis Model and Method of Analysis

The numerical analysis designs four types of frames for 10 stories; namely, moment resistant frame, moment resistant frame with hat-bracing/with hat-core-bracing, super-frame structure (cf. Fig.1). The brace contemplates use of hysteretic dampers of buckling-restraint braces, increasingly used in Japan recently. The members are regarded as having been joined by load-carrying-capacity joints.

The process of collapse is studied for four assumed patterns of main member losses shown in Fig. 3 in six assumed types of fire shown in Fig. 4. Fig. 3 shows patterns of main member losses in super-frame structures, but the same patterns of losses (location where losses occur) are applied to other types of frames. Figure 4 shows the locations of fires using the moment resistant frame, but the same locations apply to other types of frame.

In Fig. 3, the design stationary loading is proportionally distributed, and axial load utilization ratios of columns at the time of collapse are calculated for each analysis case. In the cases for analysis assuming fires where the axial load utilization ratio of the interior column is $\bar{p} = N / A\sigma_y = 0.225, 0.3, 0.35, 0.4, 0.45$, collapse temperature is calculated for each case. For both cases assuming fires or losses of main members, 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 that of the interior columns. Also, uniformly distributed loadings on beams are set by the following normalized load, $\tilde{q}$.

$$\tilde{q} = \frac{ql^2}{16M_p} \qquad (9)$$

where l and M_p respectively represent length of span and full plastic bending moment of the beam. $\tilde{q}$ 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 $\bar{p} = 0.45$. 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 brace 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 ($\sigma_y = 235 N / mm^2$). Table 1 gives cross-sectional dimensions of the member and load conditions.

In calculating the collapse temperature, used an assumed condition of fires in which temperature of the portions indicated by bold lines of Fig. 4 is uniformly raised and other portions are held at the 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.

Collapse temperature due to fire is calculated with FEM, incorporating elasto-plasticity, linear expansion of the steel material at elevated temperatures, and finite displacements of frames. In this analysis, the yield strength and the stress-strain relationships[4] are assumed to follow the assumptions given in Table 2. The strength of the steel material declines as temperature rises, and the stress-strain relationships vary accordingly. Fig. 3 shows 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 lowering as the temperature exceeds 400°C, and becomes 1/3 and 1/7 the strength at the room temperature at 600°C and 700°C, respectively. Naturally, it follows that steel columns buckle under lower load at high temperatures than at the room temperature.

This analysis defines the total collapse as either of the following two conditions. One is a condition in which the vertical displacement of the burning story or of the story where the main members reached the story above ((a) of Fig. 9). The other is the condition where the analysis begins unable to obtain a convergent solution and unable to redistribute the load.

3.3.3 Result of Analysis and Discussion

Figs. 6 and 7 show the results of analysis for fire cases. Fig. 6 shows collapse temperatures produced by each analytical model versus scale of fire. Results are shown for two cases of axial load utilization ratio of the interior column, $\overline{p} = 0.225, 0.45$. It may be noted from the figure that, in case the load utilization ratio of the interior column is $\overline{p} = 0.225$, the collapse temperature is about the same, irrespective of the type of frame. In case the axial load utilization of the interior column is $\overline{p} = 0.45$, under relatively high axial force of ordinary load, the collapse temperatures are higher for the frame structures equipped with seismic members than for moment resistant frame, for localized fire scales of Cases 1 to 4, or analytical cases where exterior columns do not yield to buckling. However, in Cases 4-2 and 5 where large fires are so large that even the exterior columns yield to buckling, the collapse temperature does not vary with the frame structure. Fig. 7 shows fire collapse temperature of each case versus the frame structure. When the scale of fire is larger, the collapse temperature is higher for structures equipped with seismic members than for the moment resistant frame. Figure 10 shows the obtained relationship between the axial load utilization ratio of the interior column and collapse temperature for each frame structure. The bold line on the figure is a series of normalized numbers obtained by the stress values that give 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 curve for each frame structure and this bold lime may be considered to indicate the degree of redundancy of each structure. It may be noted that frame structures equipped with seismic members and frames with smaller axial load utilization ratios have greater allowances. It is also indicated that the frame structure with greater axial load utilization ratios and frame structures equipped with fewer seismic members are more in danger of total collapse as simulated by distorted structures shown on Fig. 9.

Regarding the analyses for cases assuming losses of major members (Case 2), Fig. 10 shows the distortion caused by the loss of three interior columns of the first story and the axial load utilization is $\overline{p} = 0.45$. In the case of moment resistant frame, the beam of the second story comes down 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 the brace, the collapse of the first story does not extend to the total collapse. Figure 13 and 14 show the values of axial load utilization ratio versus frame structure type and versus cases for main member loss, respectively. It is evident from Figs. 11 and 12 that the collapse critical axial force ratio is higher for the super-frame structure than for other structures. The moment resistant frame can withstand loss of a considerable number of vertical load 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 critical axial force ratio is larger than the moment resistant frame, indicating higher resistance of the former to loss of the vertical load supporting members.

As discussed above, it is possible to estimate resistance to the Mode 2 progressive collapse from the value of axial load utilization ratio of columns under ordinary load. Regarding the Mode 1 progressive collapse, Equation (7) of part 1 presents a function of axial load utilization ratio of column under ordinary load, and therefore permits estimation of resistance against progressive collapse from axial load utilization ratio of column under ordinary load.

The analysis presented here uses hypothetical models that use load-carrying capacity joints, and assumes the cross-sections of the members to be uniform with respect to the story height, to study stability against the progressive collapse. This analysis was done as a preliminary study. Hereafter, the committee intends to study such themes as effects of load-carrying capacity joints, for high-rise buildings constructed according to the A seismic Code of the Building Standard Law of Japan.

4. CONCLUSIONS

This paper outlines the joint research program of the Japan Iron and Steel Federation and the Japanese Society of Steel Construction, the committee on "Study on Redundancy of High-Rise Steel Buildings". The study broke down the progressive collapses into two modes, and conducted for both modes a study on the prevention of progressive collapses.

Japan's high-rise buildings designed to resist great earthquakes use a large number of braced members as seismic members. And very few are of moment resistant frame structure that is inferior in stability in case main members are lost by fires or accidents. Recently, hysteretic dampers, excellent in plastic deformation capacity and intended to absorb input earthquake energy, are increasingly used as seismic members. The seismic design also increases the ability to withstand progressive collapses. However, this is not the result of designs that intend to prevent progressive collapses. In other words, seismic design, fire resistant design and the design to prevent progressive collapses should be promoted concurrently. This may be interpreted as indicating that, by this approach, structures excellent not only in earthquake resistance but also in fire resistance and progressive collapse resistance may be constructed, with no great difficulty nor significant increase in cost.

And the study was done by numerical analyses of the effects of the reduction of strength of members as a result of fire and the loss of members by such accident as an explosion on redundancy of steel structure buildings, for different types of frame structure, with axial load utilization ratio of column under ordinary load as parameter.

Further studies are necessary to produce more detailed and practical results. Notwithstanding, the following conclusions have been obtained from the preliminary scope of this study.

(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) Super-frame structures reinforced for earthquake resistance by using hysteretic dampers has a higher stability of the entire structure against the local collapse than the moment resistant frame, in case the strength of member is reduced by fires or structural members are lost by such accidents as an explosion.

(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 far as the results of this study are concerned, a measure of the limit to the axial load utilization ratio of column under ordinary load is about 0.25, for prevention of progressive collapses.

The committee concludes this presentation by expressing the deepest condolence to more than 3,000 people who have been killed by the attach of terrorism to the WTC towers and their families, including 479 men and women of emergency services who sacrificed their lives in pursuit of their noble missions, and aircraft crew of 157 people. The committee also hopes that this research will contribute to the upgrading of safety of steel structures.

Table 1

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

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

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

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

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

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

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

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

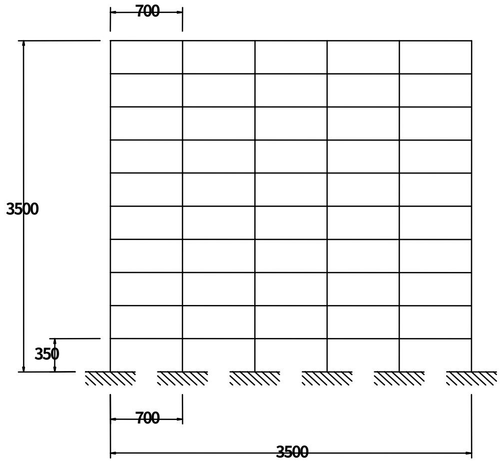
$$\sigma_{yt} = \left(1.001 - 3.592 \cdot 10^{-6} \cdot T^2\right) \cdot \sigma_{yRT} \qquad \sigma_{yRT} = 2.4t / cm^2$$

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

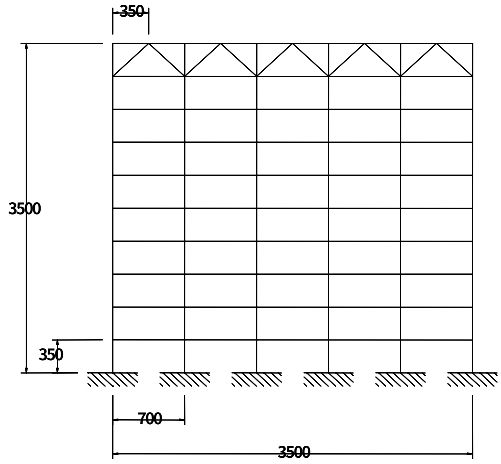
$$\sigma_{ot} = \begin{cases} where\, T \le 600°\mathrm{C} \\ \left(0.759 + 1.933 \cdot 10^{-4} \cdot T - 5.944 \cdot 10^{-6} \cdot T^2 + 2.179 \cdot 10^{-8} \cdot T^3 - 2.305 \cdot 10^{-11} \cdot T^4\right) \cdot \sigma_{yRT} \\ where\, T > 600°\mathrm{C} \\ \quad Line \text{ connecting} \left(600°\mathrm{C},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

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

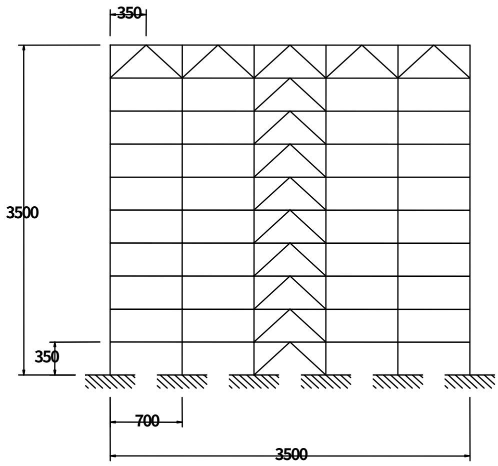
E_t：Young's modulus at T℃ σ_{yt}：Yield strength at T℃
E_{pt}：Plastic Modulus at T℃ σ_{ot}：Reference plastic stress at T℃
n_t：Shape parameter at T℃



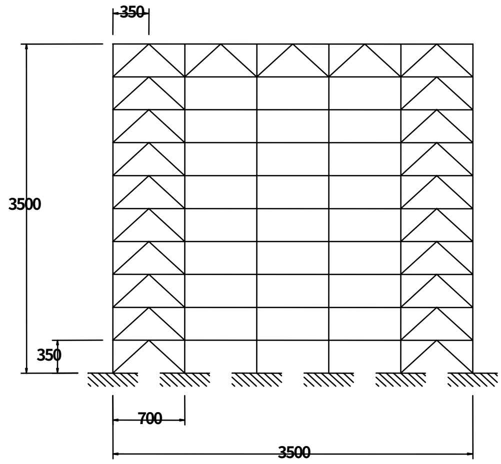
Moment resistant frame (Model 1)



Moment resistant frame with hat bracing (Model 2)

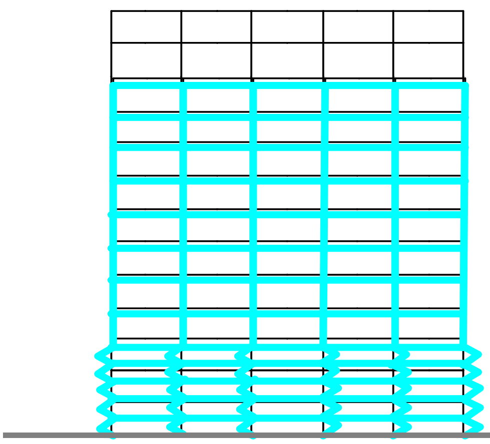


Moment resistant frame with hat-core-bracing (Model 3)

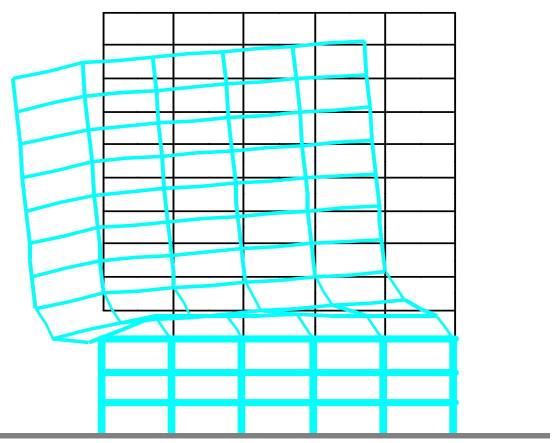


Super-frame (Model 4)

Fig. 1 Analytical Models



Progressive Collapse Mode I



Progressive Collapse Mode II

Fig. 2 Modes of progressive collapses

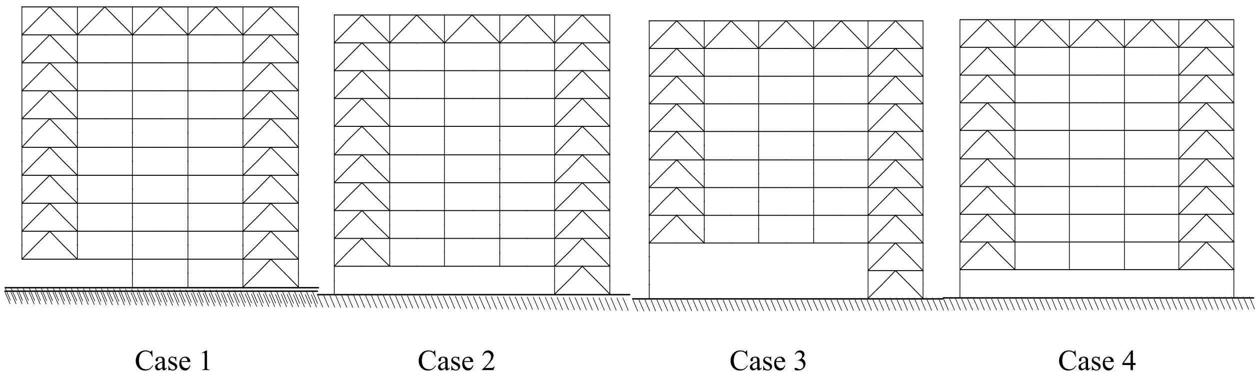


Fig. 3 Analytical models of main member losses

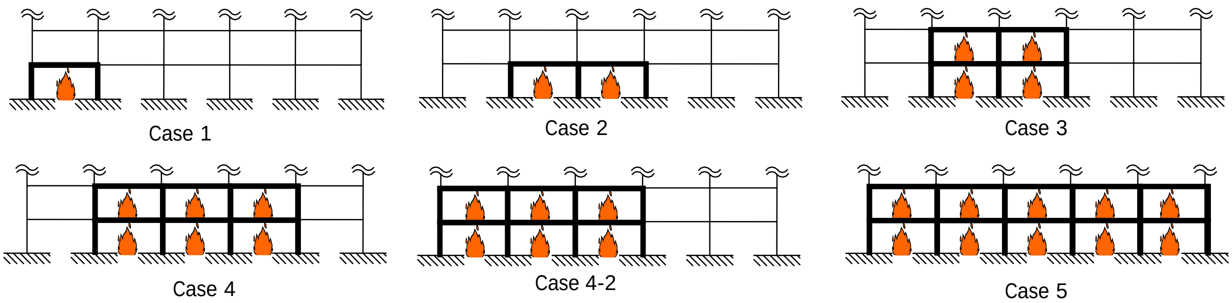


Fig. 4 Location of fires in analytical models

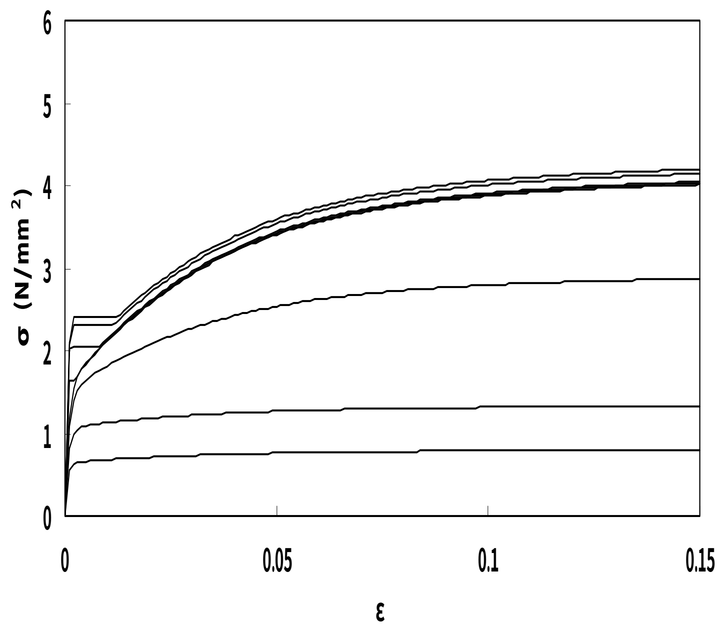


Fig. 5 Stress-strain relationship of SN400 at different temperature

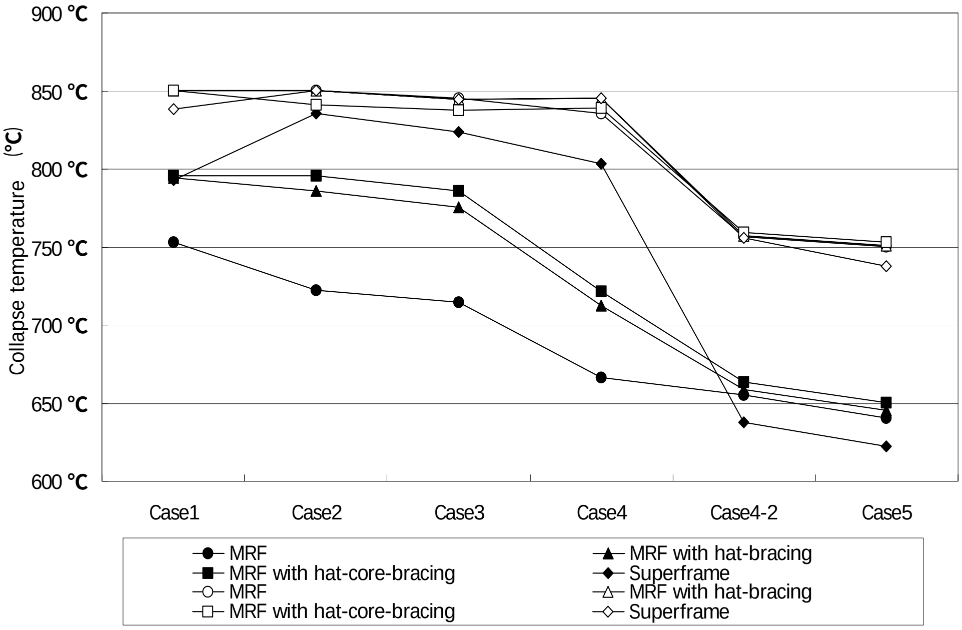


Fig. 6 Collapse temperatures produced by each analytical model versus scale of fire

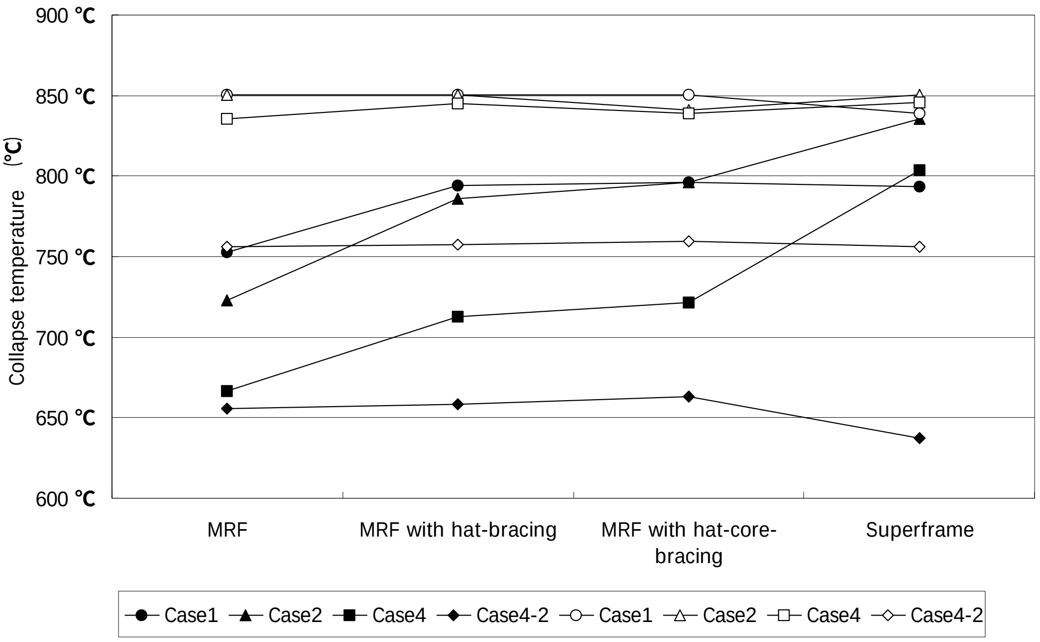


Fig. 7 Fire collapse temperature of each case versus the frame structure

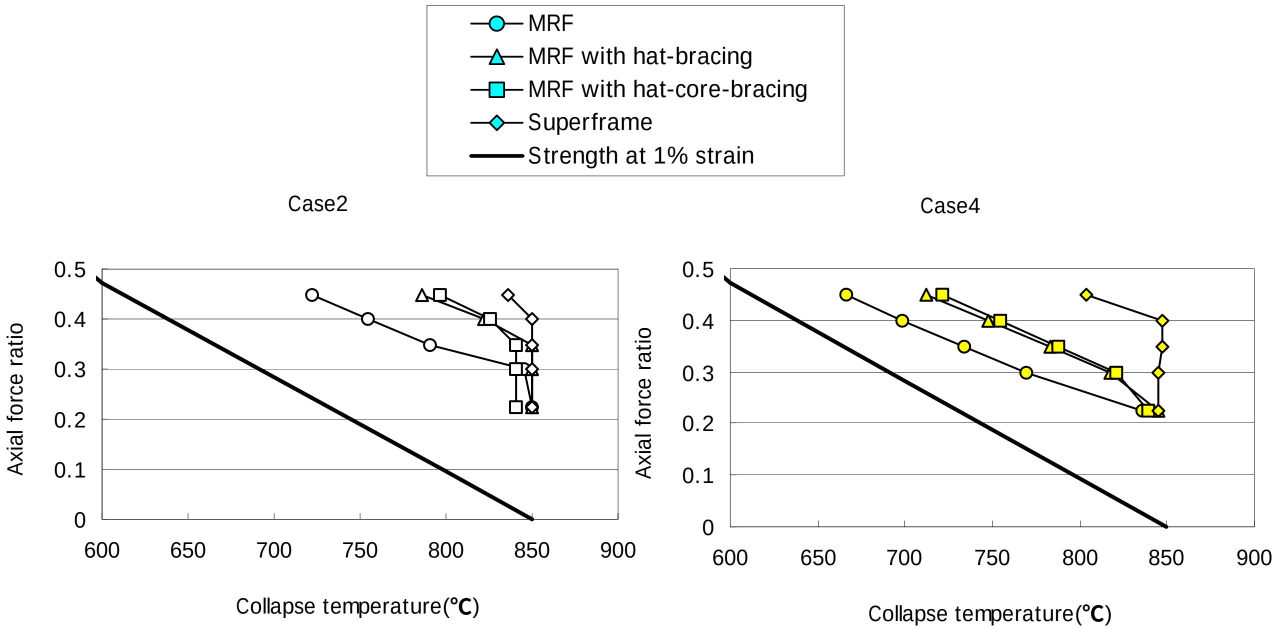
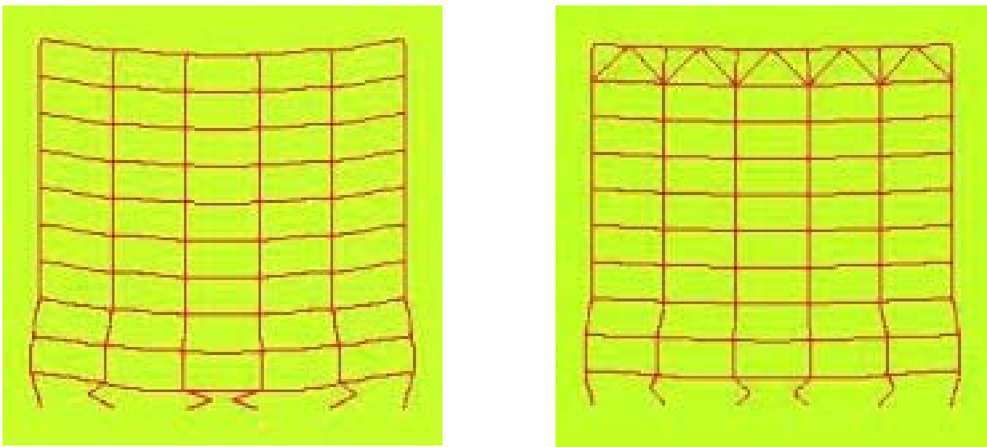
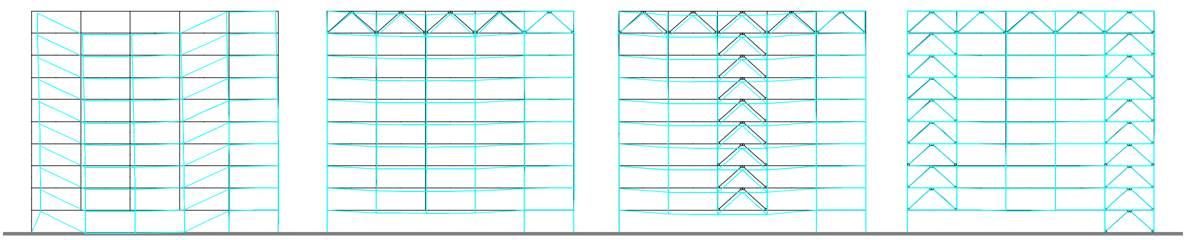


Fig. 8 Relationship between axial load utilization ratio of interior column and collapse temperature



(a)MRF (b)MRF with hat-bracing

Fig. 9 Distorted structures (fire case4: $\overline{p} = 0.45$)



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

Fig. 10 Distorted structures (less of major members case 2: $\overline{p} = 0.45$)

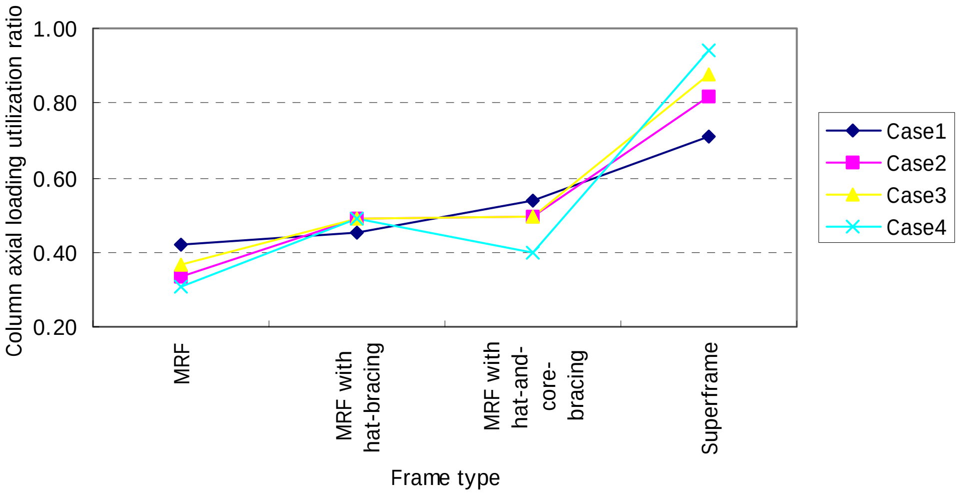


Fig. 11 Collapse axial load utilization ratio in case of loss of major members

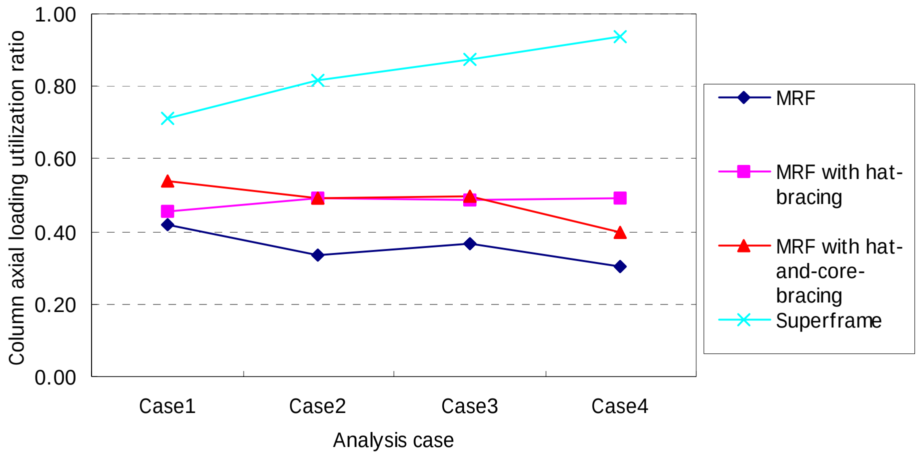


Fig. 12 Collapse axial load utilization ratio in case of loss of major members

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Acknowledgements

This research is part of a joint research program of the Japan Iron and Steel Federation and the Japanese Society of Steel Construction, the committee on "Study on Redundancy of High-Rise Steel Buildings” headed by Professor Akira Wada of Tokyo Institute of Technology. This paper incorporates various advice from committee members. Mr. Yukio Murakami of JFE Steel Corp. and Mr. Kei Nakagawa of JFE R&D Corporation contributed greatly to the numerical analysis. The authors express deepest appreciation to all men and women who generously extended cooperation to the development of this report.