Study of Structural Redundancy of High-Rise Steel Building Due to the Effect of Heat and Loss of Vertical Structural Members

by
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ABSTRACT
The collapse of the WTC towers may be taken as a warning that a local failure can trigger progressive collapse. It is also a reminder to structural engineers that they need to gain a better understanding of this structural phenomenon in order to prevent similar incidents in future. The Japanese Iron and Steel Federation together with the Japanese Society of Steel Construction established "The Committee on the Study on Structural Redundancy of High-Rise Steel Buildings" in June 2002 in an attempt to study and provide a better understanding of progressive collapse. This paper discusses the structural redundancy of high-rise steel building structures and issues raised by the proposed research project. Also, it presents a theoretical investigation of the redundancy characteristics of steel frames due to heat caused by fire and loss of columns due to impacts from explosions and plane crashes. A non-linear analysis was used to estimate the redundancy characteristics through examination of the axial load utilization ratio of columns. Investigations were carried out on moment resistant frames, braced frames with hysteretic dampers, and outrigger truss systems. It is concluded that steel frames designed with load-carrying capacity joints can withstand major fires. For any loss of vertical load-bearing members to contribute to progressive collapse, it was found that the axial load utilization ratio in members had 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: collapse temperature, high-rise steel building, structural redundancy, progressive collapse

1. INTRODUCTION

The direct causes of the collapse of the World Trade Center (WTC) Towers on September 11, 2001 were damage to the columns caused by the impact of the two jetliners, and the major ensuing fires. WTC1 and WTC2 remained standing for 102 and 56 minutes after the impacts, respectively, during which periods many lives were saved. The great plastic deformation capacity or load transferring 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 structural redundancy. However, the collapse of the WTC towers may be taken to be a warning that a local failure can trigger a progressive collapse, and a landmark event that should impress on construction engineers the importance of preventing the progressive collapse of similar structures.

Prevention of progressive collapse requires development of design technologies for frames that have excellent structural redundancy. The Japan Iron and Steel

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Federation and the Japanese Society of Steel Construction established in June 2002 a committee that would carry out a "Study on Structural 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: (1) a study on collapse-control design, on the basis of Japan's earthquake resistant and fire prevention technologies, and (2) quantification of structural redundancy of Japan's high-rise steel buildings and the proposal of frames that have excellent structural redundancy. This paper discusses highly redundant and high-rise steel building structures and summarizes issues that should be examined in this research project. Furthermore, in order to quantify redundancy differences among high-rise steel building structures, it numerically examines conditions to prevent progressive collapse that results from fire-induced loss of member strength and loss of structural members after explosions and other serious accidents, using the parameters of column axial load utilization ratios at ordinary loading and frame types (moment resistant frame (MRF), moment resistant frame with hat-bracing, moment resistant frame with hat-and-core-bracing, super-frame structure). Additionally, it verifies numerical and quantified redundancy against loss and fire damage of columns in actual high-rise steel buildings that were designed in conformity with the seismic code of the Building Standard Law in Japan.

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

2.1 High-rise Steel Buildings Structure with High Structural Redundancy

Both of the WTC towers were able to withstand the impact of the jetliners crashed into them and kept standing for some time, permitting many people to evacuate. In this sense, the WTC towers should be credited with saving many lives. This indicates that the two WTC towers had some structural 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 to rescue the trapped people. The process by which the collapse of the story that suffered the impact led to the collapse of the entire building may be interpreted as indicating that the structure did not have sufficient structural redundancy to prevent the collapse of the entire building.

Against such a background, the necessity of designing buildings able to withstand total collapse in the event of a sudden and massive accidental load caused by an act of terrorism or accident is being discussed by various concerned parties. This committee considers that structures designed with the capacity to withstand total collapse by a large margin against external force are different from what a true redundant structure should be, though such structures in effect could be regarded as redundant structures. One example of a redundant structure is a structure where its redundant strength in terms of aerodynamic design or seismic design, and its redundant strength at load carrying-capacity joints, both for ordinary loads, combine to provide structural redundancy against a sudden and massive accidental load created by an aircraft crash or explosion. In other words, the concept of redundant structure should include redundant strength inherent in the subject structure, or should lead to the realization of high-rise steel buildings with excellent structural redundancy through proper selection of structural features, arrangement of members and proportioning of sections at a minimum additional cost.

2.2 Objectives and Research Theme

The 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 progressive collapse; (2) Presentation of a recommendation for construction of high-rise steel buildings with adequate structural redundancy, based on the collapse control design standards or design recommendations.

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

< Common themes >

1. Establishment of a clear definition of the terms “local collapse” and “progressive collapse” as design terminology to be considered in design work
2. Evaluation of differences in structural redundancy according to the frame type (moment resistant frame (MRF), moment resistant frame with hat-bracing, moment resistant frame with hat-and-core-bracing, super-frame structure, cf. Fig.1)
3. Proper structural redundancy of steel 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 story collapse and to the loss of main structural members 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 behavior of members under explosion and impact load conditions
8. Approach to structural redundancy of such space structures as shells, domes, etc.

< **Fire-resistant Design Working Group** >

9. Improved accuracy 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 of fire-resistant specifications and floor construction, 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 of technical measures to improve fire resistant redundancy (use of CFT structure, fire resistant steel and fire partitions, etc.)

The Structure Design Working Group will divide the progressive collapse mode of seismically and aerodynamically designed high-rise steel buildings into two types as mentioned below, and analyze the stability of frames for each collapse mode. The analysis will use the region in which the main member is believed to be 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 frame. Further, the working group will clarify the behavior of members subjected to 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 (Great Hanshin-Awaji Earthquake). In this way, the working group will clarify the stability of members and joints against accidental loads. The working group will also study approaches to structural redundancy of such large space structures as shells, domes, etc., one of the main structure themes of this committee.

The Fire-resistant Design Working Group is promoting a study on the degree of structural redundancy (patterns of damage) of high-rise steel buildings having seismic and fire-resistant design, with the burning area as a parameter. More specifically, the study aims to clarify the structural stability at elevated temperatures for each frame type. In this study, the accuracy of the 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 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 in order 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-resistance specifications of the floor, the type of joint connecting the floor with the main structure, and the joint structure between columns and beams on the structural redundancy of total structure to be very great. The working group is studying the fire-resistance performance of the floors and high strength friction type bolted connections used in Japan (rigid joint), and also presents recommendations on upgrading structural redundancy.

The above study will identify members and connecting parts constituting the key elements required to secure good redundancy from both structural and fire-resistance viewpoints. In effect, these key elements should be preferentially protected. The working group presents recommendations for the design of steel buildings with superior structural redundancy after study on technical measures that reduce incremental costs, such as use of CFT columns, fire resistant steel, and methods for installing fire partitions.

3. PREVENTION OF PROGRESSIVE COLLAPSE

3.1 Mode of Progressive Collapse

The committee classifies progressive collapse into two modes as shown in Fig. 1, and is promoting research into the conditions that will prevent progressive collapse for each of the modes. Mode 1 of Fig. 1 illustrates propagation of story collapse to the lower stories. This mode represents the case where a collapse occurs on a certain story (called initial collapse story - it 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 load transfer 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 Mode 1 collapse occurred following the Mode 2 collapse. The WTC tower case is being evaluated without a clear distinction being made between these two modes, because these two modes of collapse occurred consecutively. In studying conditions that will prevent these two collapse modes from occurring, it is necessary to study such conditions, with the two modes clearly differentiated. This paper omits a review of Mode 1 progressive collapse, though when a certain layer of a high-rise building is collapsed and then a combination of this initially collapsed layer and its upper floors fall rigidly, we might determine whether vertical load resistant members of the most fragile layer of lower floors in the collapsed layer can withstand this drop, considering the robustness of seismic design high-rise buildings in Japan and referring to studies conducted by Bazant 3) et al. The study will focus on the prevention of Mode 2 progressive collapse.

3.2 Prevention of Mode 2 Progressive Collapse

The following pages compare a super-frame structure equipped with hysteretic dampers installed to reduce response to seismic movements and a moment resistant frame, in terms of the stability of the entire frame structure against a local collapse, by means of static analysis. The study includes a static analysis simulating the condition in which main members are lost as the result of an explosion and an analysis on stability of the frame at high temperatures, based on the assumption that the frame is heated by a fire.

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

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

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

Furumura et al[4]. have shown that plasticity 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 compromise the structural integrity of steel frame structures. On the other hand, the collapse of the WTC towers indicates the 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]. Thus, there is clearly a real need to study structural stability at elevated temperatures in order to prevent progressive collapse as a result of fire load.

This study regards the unstable process affecting the frame as a process of snap through from a condition of static balance into 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. However, in an unstable process, temperature rise is suspended until the process of snap through is finished, and the frame is analyzed from the standpoint of displacement control. This approach has made it possible to solve a number of structural stability problems that occur at elevated temperatures, and the ultimate states or collapse temperatures of the frame at the time of fire have been clarified. Regarding theories on the dynamics governing the behavior 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 the frame caused by the loss of major members and the instability of frames at elevated temperatures, and also structural forms that can prevent Mode 2 progressive collapse.

3.2.2 Numerical Analysis Model and Method of Analysis

Numerical analysis can be used in the design of four types of frames for 10-story buildings; namely, moment resistant frame, moment resistant frame with hat-bracing or hat-core-bracing, and super-frame structure (cf. Fig.2). We assumed that hysteretic dampers of buckling-restraint brace type, increasingly used in Japan recently, are used. The members are regarded as having been joined by load-carrying-capacity joints.

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

In Fig.3, the design ordinary loading is proportionally distributed, and axial load utilization ratios of columns at the time of collapse are calculated for each case. In cases of fire where the axial load utilization ratio of the interior column is,

$\bar{p} = N/A\sigma_y$ =0.25,0.3,0.35,0.4,0.45, the collapse temperature is calculated for each case. For both cases fires or losses of main members are assumed, the slenderness ratio of columns is set at $\lambda = \ell/i$ =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, $\tilde{q}$

$$\tilde{q} = \frac{q\ell^2}{16M_p} \qquad (1)$$

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 a 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 super-frame structure. The steel materials are all JIS G 3136 SN400 (σ_y=235N/mm^2). Table 1 gives cross-sectional dimensions of the member and load conditions.

In calculating the collapse temperature, we assumed that the temperature of the portions indicated by the bold lines in Fig. 4 is uniformly raised and other 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 12×10^{-6}°C.

Collapse temperature 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[5] 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. 5 shows stress-strain relationships of structural steel, JIS G 3136 SN400, at different temperatures. It may be noted from the figure that the strength of the steel begins to decline as the temperature exceeds 400°C, and becomes 1/3 and 1/7 the strength at room temperature at 600°C and 700°C, respectively. Naturally, it follows that steel columns buckle under lower loads at high temperatures than at 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 were lost reached the story above ((a) of Fig. 10). The other is the condition where the analysis is unable to obtain a convergent solution and unable to redistribute the load.

3.2.3 Results of Analysis and Discussion

Fig. 6 and Fig.7 show the results of analysis for fire cases. Fig. 6 shows collapse temperatures produced by each analytical model versus the scale of the fire. Results are shown for two cases where the axial load utilization ratio of the interior column is $\bar{p}=0.225, 0.45$. It may be noted from Fig. 8 that, in the case where the load utilization ratio of the interior column is $\bar{p}=0.225$, the collapse temperature is about the same, irrespective of the type of frame. In the case where the axial load utilization of the interior column is $\bar{p}=0.45$, under a relatively high axial force of ordinary load, the collapse temperatures are higher for frame structures equipped with seismic members than for the 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 massive that even the exterior columns yield to buckling, the collapse temperature does not vary with the frame structure. Fig. 7 shows the fire collapse temperature for each case versus the frame structure. When the scale of the fire is larger, the collapse temperature is higher for frame structures equipped with seismic members than for the moment resistant frame. Fig. 8 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 ($235 N/mm^2$) at room temperature. The distance between the curve for each frame structure and this bold line may be considered to indicate the degree of structural 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 also indicates 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 the distorted structures shown in Fig. 9.

Regarding analyses of cases assuming the loss of major members (Case 2), Fig. 10 shows the distortion caused by the loss of three interior first story columns where the axial load utilization is $\bar{p}=0.45$. In the case of the moment resistant frame, the beam of the second story comes down to the ground, meaning that the collapse of the first story leads to total collapse. By contrast, despite the loss of interior columns and the bracing, the collapse of the first story does not result in total collapse. Fig. 11 and Fig.12 show the values for axial load utilization ratio versus frame structure type and versus cases for main member loss, respectively. It is evident from Fig. 11 and Fig.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 the 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 the 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 Mode 2 progressive collapse from the value of the axial load utilization ratio of columns under ordinary load.

3.3 Analysis of the behavior of actual Japanese high-rise steel buildings against unexpected external force (loss of members)

Next, taking an example of an over 60m high-rise steel office building that was designed in conformity with the seismic code of the Building Standard Law in Japan, we estimated numerical redundancy against excitation (local fracture etc.) that is not assumed in the design, and identified its characteristics.

3.3.1 Outline of target models

Designing the building structures in Japan, a country prone to frequent earthquakes, covers seismic and wind loads as well as ordinary vertical loads. Member cross-sections of columns and beams that make up the structures often depend on these horizontal loads. Fig. 13 shows a floor plan and an elevation of a target building. The target is an office building that has 27 levels above the ground, a maximum height of about 130 m, a basic column span of 6.4 m and a steel moment resistant frame structure. A typical floor has a plane shape of one-sided core type and an area of 57.6 m×24.5 m. A column-span in a longitudinal direction and a beam-span extend 6.4 m and 17.5 m, respectively. Beams make up an office without columns. All main frame cross-sections will be determined based on seismic and wind loads. A column has a built-up box cross-section that includes two types of 750×750 and 650×650, and a thickness of 25 to 45 mm. It is made of JIS G 3136 SN490C steel.

The girder has a built-up H or roll H section. Its height on a standard floor is 850 mm while it is 1,000 to 1,500 mm on the lower and top floors. Flange thickness is 25 to 32 mm. The flange is made entirely of JIS G 3136 SN490B steel. The beam is made of roll H-shaped JIS G 3036 SS400 steel. In addition, the slab is made of RC produced with deck plate permanent forms as shown in Fig. 14. Lightweight Class 1 concrete is used for the slab.

Connection methods are described next. A column-to-column joint is connected by full face field butt welding and is a full strength joint. On the other hand, columns and beams are connected by high strength friction type bolted connections on which a beam flange is field welded and a gusset plate is used for a web.

Assuming that a column axial load utilization ratio is defined as the ratio of ordinary vertical load to column ultimate axial strength, the force ratio will be approximately 0.1 to 0.35 (if it is defined as the ratio of ordinary vertical load to column yield strength, it will be 0.0008 to 0.307.), and is smaller at a corner column where axial force varies greatly under a horizontal load.

3.3.2 Analysis model and method

In order to develop an analysis model, columns and girders were modeled as beam elements to link the elements for a 3-D model. A material non-linear is determined from a bilinear $\sigma - \varepsilon$ relation for each nodal point shown in Fig. 15. In this case, it was decided that the yield point would be 1.1 times the specification material strength.

A non-linear static incremental analysis was made for the following four cases based on NASTRAN, a general-purpose analysis program, taking the locations where columns were lost as a parameter.

Case 1: Loss of first-floor center columns, Case 2: Loss of first-floor corner columns, Case 3: Loss of 20th floor center columns, Case 4: Loss of 20th floor corner columns

More specifically, columns were removed one by one to determine a collapse critical state, i.e. the state where stationary axial force cannot be maintained any longer.

3.3.3 Analysis Results

Analysis results for each case are shown as follows:

(Case 1) Frames were stable after the loss of 6 center columns. Plastic hinge occurred at each end of the girders in the center of the 1st to 19th floors, which formed a beam sideway's mechanism. In the next step, the frames became unstable after the loss of 8 center columns.

(Case 2) Frames were stable after the loss of 5 corner columns. Plastic hinge occurred at each end of the girders in the corner of the 1st to 13th floors, which formed a beam sideway's mechanism. In the next step, the frames became unstable after the loss of 6 corner columns.

(Case 3) Frames were stable after the loss of 8 center columns. Plastic hinge occurred at each end of the girders on the 20th to roof floors, which formed a beam sideway's mechanism. In the next step, the frames became unstable after the loss of the 10 center columns.

(Case 4) Frames were stable after the loss of 7 corner columns. Plastic hinge occurred at each end of the girders on 20th to roof floors, which formed a beam sideway's mechanism. In the next step, the frames became unstable after the loss of 8 corner columns.

Deformation at collapse in Case 1 is shown in Fig. 16. In addition, redistribution of ordinary loads induced by lost columns, that was estimated based on the axial force exerted on the first floor under the collapse critical state in Case 1, is shown in Fig. 17. Assuming that the total of shear force that was applied to the 2nd to roof floor beams on row Y1 (section of X8-X9) and to those beams on rows X6-X8 (section of Y1-Y4) equals the ordinary vertical load that was redistributed to the frames perpendicular to the same cross section and in the same cross section, respectively and which the columns should have borne, the ratio of shear force of both frames reached about 7:3. Vertical load is redistributed to the frames perpendicular to the same cross section via a 17.5 m long span beam. For this reason, though the vertical force redistributed is smaller than that to the frames in the same cross section via a 6.4 m uniform span beam, a long span beam can be found to contribute to also redistribution of ordinary vertical load and produce promising three-dimensional effects during great deformation.

3.3.4 Discussion and Summary

After conducting an analysis on the massive deformation occasioned by the loss of columns in an actual 27-story one side core type office building with 6.4 m-column grids and 17.5 m long span beams, the building was confirmed to be able to support ordinary vertical loads following loss of 15 to 20% of all columns. We next consider analysis results in the following 1),2),3).

1) Allowance degree of column and beam member cross-sections

Members of this building structure, as described above, have a low ratio of column axial force to ordinary vertical load, approximately 0.1 to 0.35. Even a 17.5 m long span beam has a low ratio of end long-term bending moment to full plastic moment, approximately 0.2 to 0.3. These allowance degrees are main contributors, in that the frames could be maintained even after the loss of a substantial number of columns.

2) Structure type and three-dimensional effects

All columns and beams serve as elements resistant to horizontal load, so all column-to-beam connections are rigidly jointed to form a mechanism by which seismic elements are distributed to the overall building. This structure type enables three-dimensional load redistribution.

3) Difference between effects of loss of center and corner columns

When columns are lost, their upper moment resistant frames redistribute loads that the columns bore. When center and corner columns are lost, the loads are redistributed by both-end support frames and cantilever frames, respectively. This analysis did not show a great difference between these frames, but the cantilever frames could not support the vertical load earlier than the both-end support frames.

This analysis assumes that member and column/beam connections have sufficient plastic deformation capacity. It goes without saying that the prevention of brittle fracture against this presumption will be a condition for designing a highly structurally redundant building.

In this paper, the effects of static loading to the members were examined. Dynamic effects of the instantaneous loss of columns should be reviewed in the future.

3.4 Analysis of the behavior of actual Japanese high-rise steel buildings against unexpected fire

Here, taking the same example as in 3.3, we conducted an analysis on fire response against an external force that exceeds the criteria for standard fire resistance design, and identified characteristics.

3.4.1 Analysis model and method

In order to develop an analysis model for the same building as in 3.3, a 2-D model was selected that covers the Y1 row shown in Figs. 13 and 19. We decided to select an equation that was described in the “Guidelines for Steel Structure Fire Resistance Design” of the Architectural Institute of Japan as a steel $\sigma - \varepsilon$ relation for fire response analysis shown in Figs.18.

Our own analysis program was used to conduct fire response analysis on a total of 24 cases where a combination of a fire floor and a layer direction fire extension is taken as a parameter, shown in Figs.19 and 20.

Fire floor: First floor (3-hour fire resistance), 14th (2-hour fire resistance), 24th (1-hour fire resistance)

Layer direction fire extension: Proximity to layer ends (4 cases), proximity to layer center (3 cases), overall layer (1 case)

3.4.2 Analysis results

In Fig. 21, we summarized the relation between the obtained frame collapse temperature and the number of columns that were heated by fire in a block defined for each case in the analysis parameters. Steel that was used for fire response analysis has high-temperature characteristics where its strength declines to 0 at 750°C, so the analysis is terminated when a frame member is heated to a temperature of 750°C. For this reason, it can be considered that a frame will not totally collapse when the collapse temperature of the frame reaches 750°C. Hence, when a fire breaks out in the lowest layer, as in the side fire case, the heating of three spans from the side and 6 columns or more may cause total frame collapse.

3.4.3 Discussion and Summary

We next consider analysis results in the following 1),2),3).

1) Effects of layer direction fire spread difference on collapse temperature

In the center fire case, the heating of 5 spans in the center and 6 columns or more may cause total frame collapse. In addition, comparing the side fire case with the center fire case, the heating of fewer columns in the former than in the latter caused frame collapse even at the same frame collapse temperature. It was found that when a fire breaks out at the side instead of the center, namely, where an unexpected fire load is applied to perimeter columns or corner columns, this may easily lead to total collapse.

Fig. 22 shows deformation of a frame in the proximity of its collapse in Case 5 and Case 6 on the 14^{th} floor. According to the analysis results, it can be assumed that the frame will be only locally collapsed due to the fact that its collapse temperature is 750°C as seen in Case 5. On the other hand, there is high possibility that the frame will totally collapse when its collapse temperature is 711°C as seen in Case 6.

2) Effects of column axial load utilization ratio to frame collapse temperature

Fig. 23 summarizes the relation between the obtained frame collapse temperature and the axial load utilization ratio of a fire-heated center column. The axial load utilization ratio is expressed at room temperature before heating of the column. The bold line in the figure shows the ratio of stress at 1 % of distortion and yield strength at room temperature in a σ-ε relation for each temperature of the materials used for analysis. The distance between this line and the collapse temperature for each case represents the structural redundancy. According to the analysis results, collapse temperature decreases with increasing fire blocks and becomes asymptotic to a curve of material characteristics. In Case 8 that shows the effect of fire on an overall layer, it was found that the collapse temperature of a frame will be equal to that of a single column. According to the results, however, the frame collapse temperature almost coincides with the above curve (heavy line), so the adequacy of the analysis is demonstrated.

3) Relation between frame collapse temperature and beam deflection

Fig. 24 shows the relation between maximum deflection and member temperature of a heated beam. A filled circle and a void circle show the analysis results for Case 5 and 6, respectively. For reference, a heating curve and]criteria of beam deflection (l 2=800d：1：span, d: beam height) that are often used in current fire resistance design are shown by the bold line. From Figs. 22 and 24, it can be seen that in Case 5 where a frame may be only locally collapsed, excessive beam deflection can lead to beam collapse. In addition, cases where the frame may be only locally collapsed are all dominated by beam deflection, so the relation “local collapse”=”beam collapse” can be established.

For more information, Case 5 shows that beam deflection is well over normal fire resistance design criteria. This fire response analysis implies an example of behavior tracking when so-called unexpected external force is applied to the building structures, such as when fire resistant covering materials are lost due to some external factors and when a fire breaks out that exceeds usually assumed fire load.

In the model used for this analysis, as discussed in 3.3, a beam end has a rigid joint framed structure. When a beam end has a pin joint framed structure, it is extremely difficult to redistribute stress to the sound beams above a fire floor. In order to provide structural redundancy, stress redistribution frames such as hat truss and belt truss types are required.

4. CONCLUSIONS

This paper outlines the joint research program of the Japan Iron and Steel Federation and the Japanese Society of Steel Construction, and the committee charged to undertake a "Study on Structural Redundancy of High-Rise Steel Buildings". The study broke down the progressive collapse phenomenon into two modes, and conducted a study on the prevention of progressive collapses for both modes. The study was performed by conducting a series of numerical analyses of the effects of the reduction of strength of members as a result of fire and the loss of members by such accidents as an explosion on the structural redundancy of steel buildings, for different types of frame structure, with the axial load utilization ratio of column under an ordinary load used as a parameter. In summary, the study has led to the following conclusions.

(1) Steel frame structures employing load-carrying capacity joints can withstand large-scale fires and loss of vertical load resistant members if the axial load utilization ratio of columns remains low.
(2) Super-frame structures reinforced to create earthquake resistance by using hysteretic dampers have higher overall structural stability against local collapse than the moment resistant frame, when the strength of members is reduced by fires or structural members are lost as a result of accidents such as an explosion.
(3) The axial load utilization ratio of a column under an ordinary load can be an effective parameter in studies to prevent progressive collapse. As far as the results of this study are concerned, it was found that the limit of the axial load utilization ratio of a column under an ordinary load is about 0.25, in order to prevent a progressive collapse.

Furthermore, taking an example of an actual high-rise steel office building that was designed in conformity with the seismic code of the Building Standard Law in Japan, we estimated numerical redundancy against an external force (column loss from explosion, member heating induced by fire beyond a fire partition) that is not assumed in the design, and identified its characteristics. The following results were obtained from the examination.

(4) When remaining frame members have a rigid joint structure after loss of columns, it is possible to redistribute the loads that failed columns bore and to inhibit progressive collapse even if some columns are lost.
(5) If the region in which a fire breaks out or columns are lost is closer to the exterior of a building, the structural redundancy of the building will be reduced.

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

The Corresponding Committee was established as part of CTBUH (Council of Tall Buildings and Urban Habitat) and the results of the study conducted by this committee will be disseminated worldwide through the Corresponding Committee.

The committee concludes this presentation by expressing their deepest condolences to the families of the more than 3,000 people who were killed by the terrorist attack on the WTC towers, including 479 men and women from the emergency services who sacrificed their lives in the execution of their noble mission, and the 157 aircraft crew who also lost their lives. The committee also hopes that this research will contribute to the upgrading of safety of steel structures.

Table1 Section of frames and load conditions

Section of frames ・ 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 conditions ・ Normalized loading on beam ・ Column force ratio(The lowest story)	: $\tilde{q} = \dfrac{qi^2}{16M_p} = 0.15$: $\bar{p} = N / A\,\sigma_y = 0.225, 0.3, 0.35, 0.45$

Table2 Assumption for formulating yield strength and stress-strain relationship of structural steel SN400

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

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

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

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

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

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

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

$$E_{pt} = Lineconnecting(0°C, 50.0t/cm^2), (400°C, 50.0t/cm^2), (600°C, 5.0t/cm^2), and (850°C, 0.0t/cm^2)$$

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

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

Et : Young's modulus at T℃, σyt : Yield strength at T℃

Ept : Plastic Modulus at T℃, σot : Reference plastic stress at T℃

nt : Shape parameter at T℃

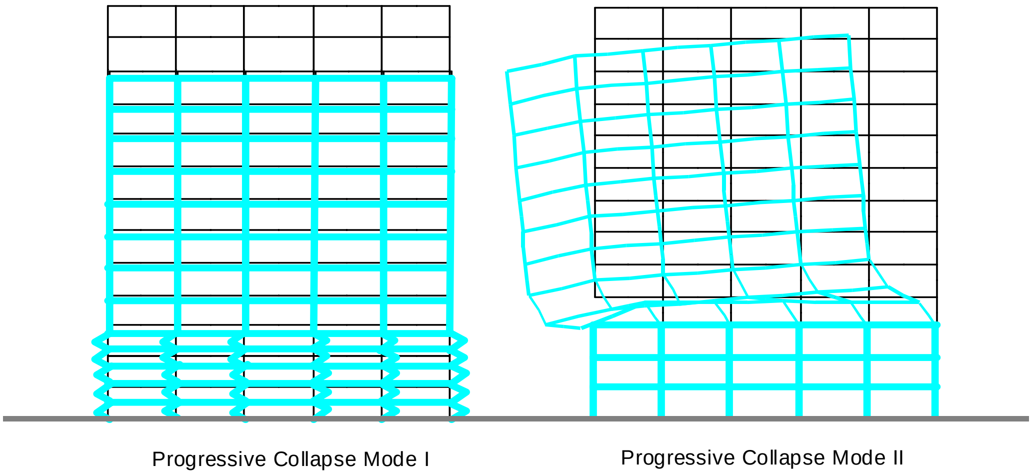
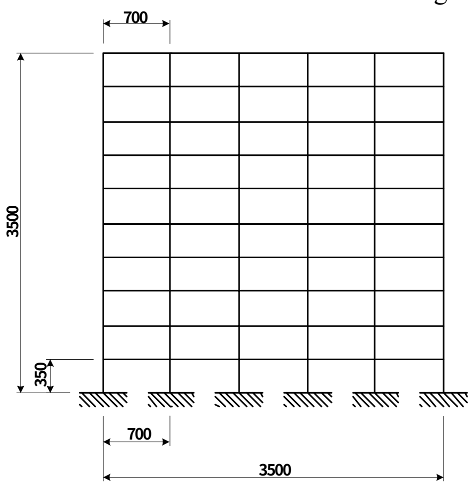
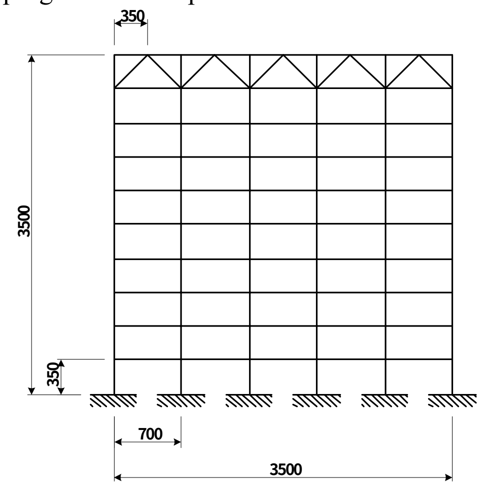


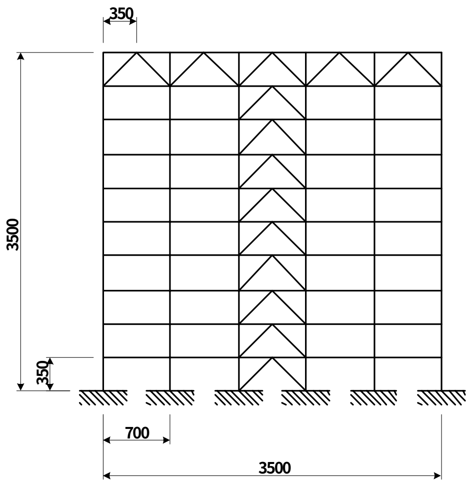
Fig.1 Modes of progressive collapse



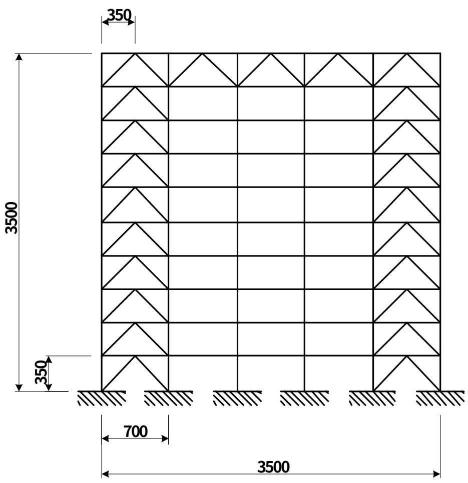
Moment resistant frame (Model 1)



Moment resistant frame with hat bracing(Model 2)



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



Super-frame(Model 4)

Fig.2 Analytical Models

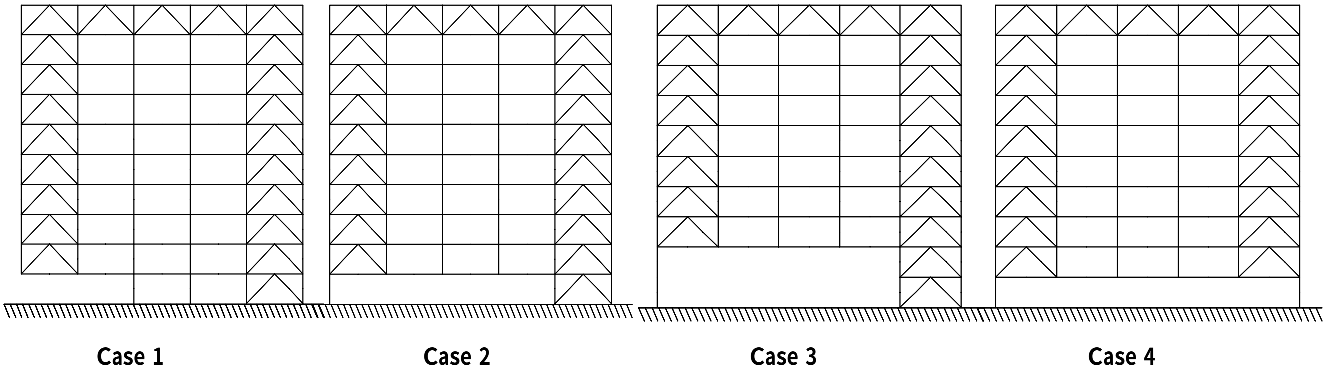


Fig.3 Analytical models assuming four patterns of main member loss

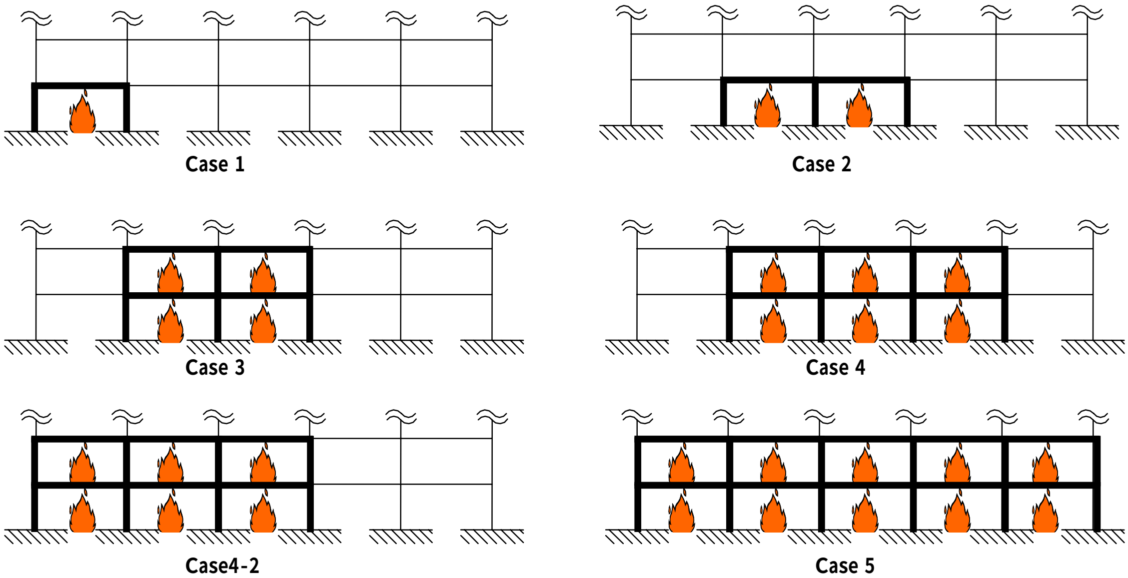


Fig.4 Location of fires in analytical models

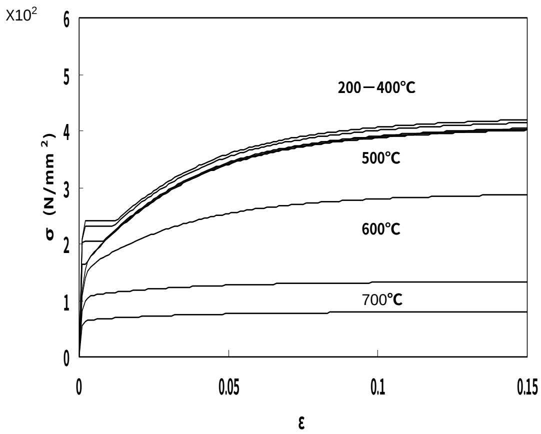


Fig.5 Stress-strain relationships of SN400 at different temperatures

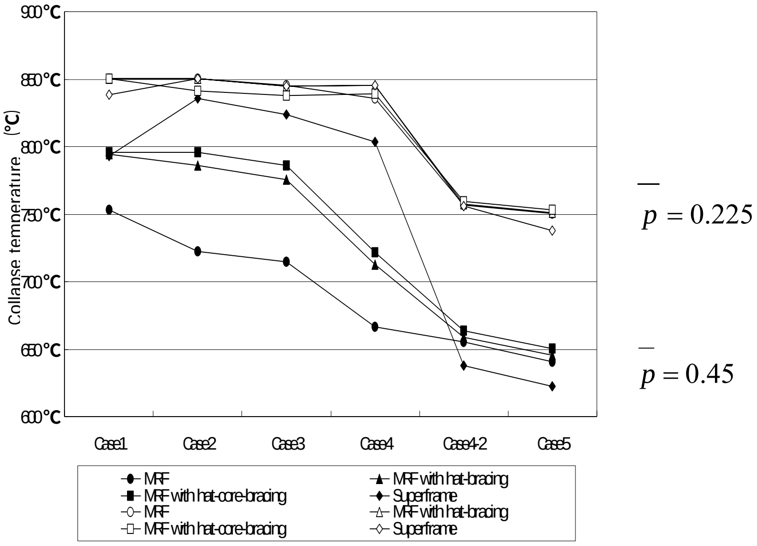


Fig.6 Collapse temperatures produced by each analytical model versus scale of fire

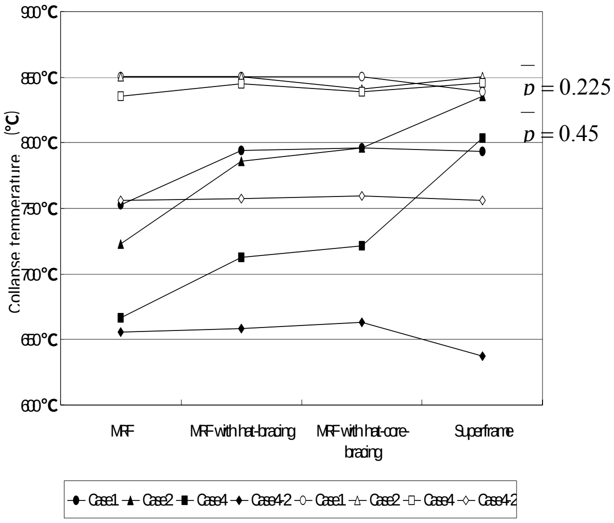


Fig.7 Fire collapse temperature for each case versus the frame structure

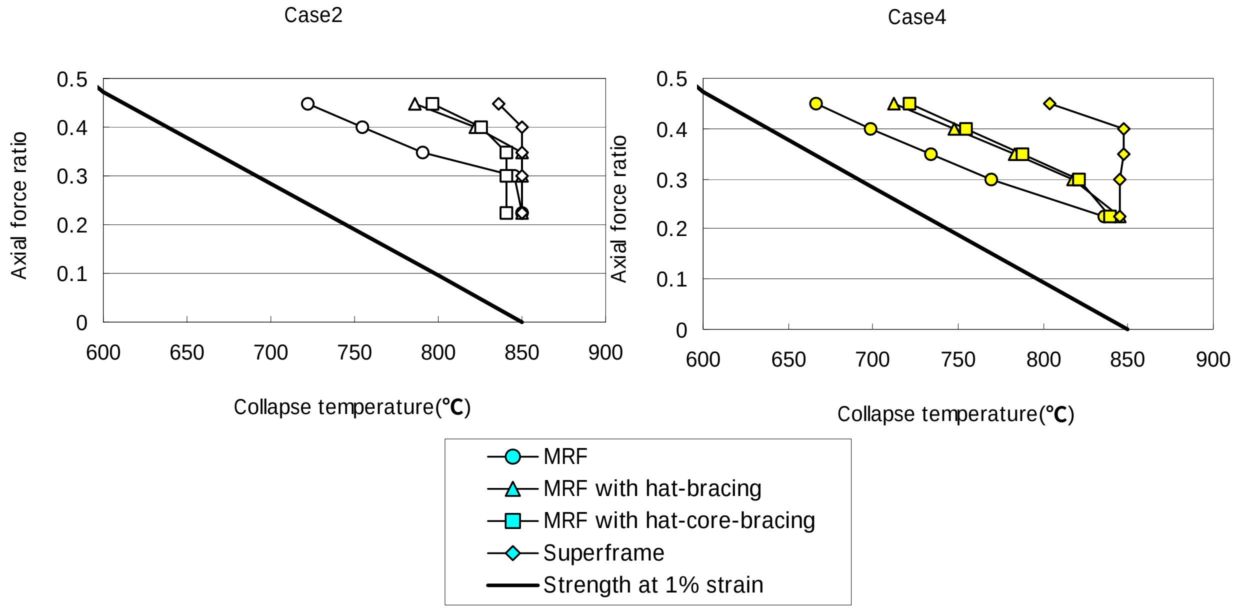


Fig.8 Relationships between axial load utilization ratio of interior column and collapse temperature

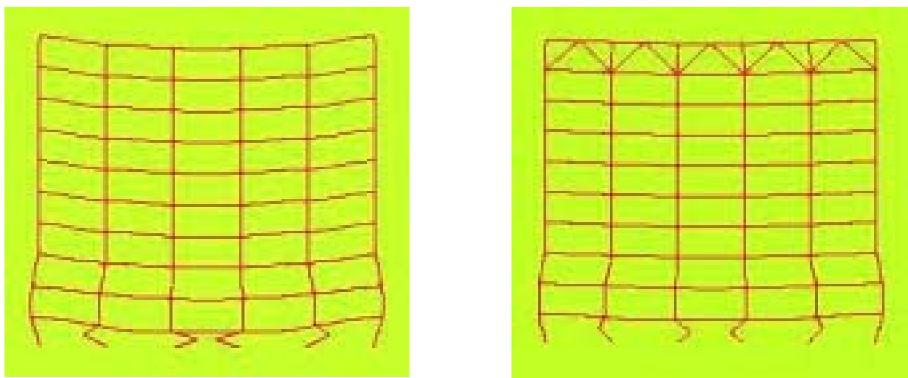
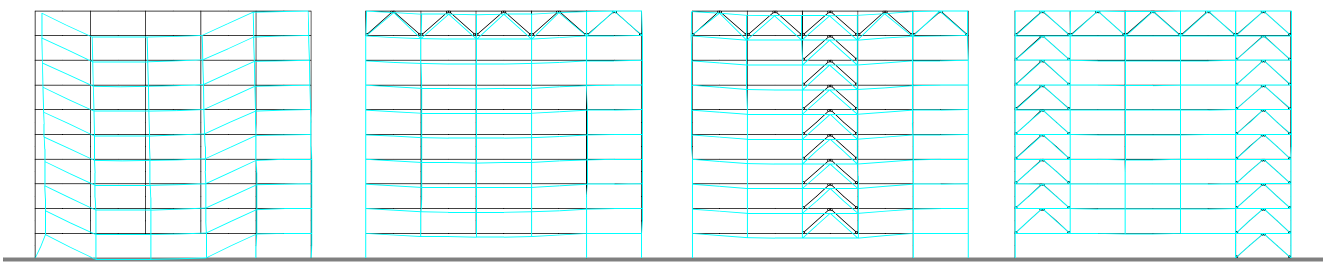


Fig.9 Distorted structures (fire case4: $\bar{p} = 0.45$)



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

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

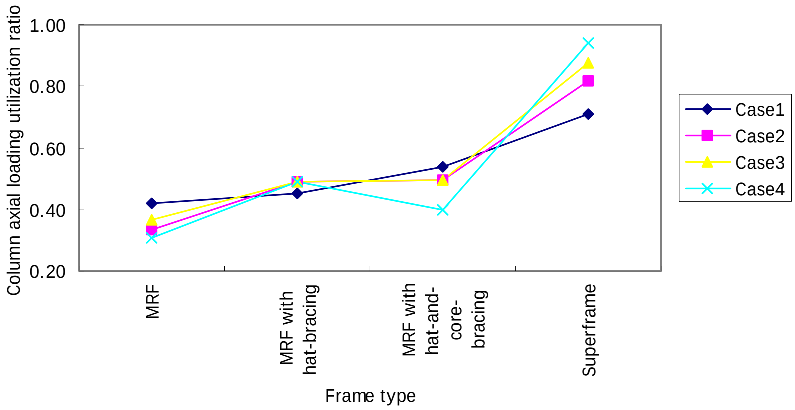


Fig.11 Collapse axial load utilization ratio versus frame structure type

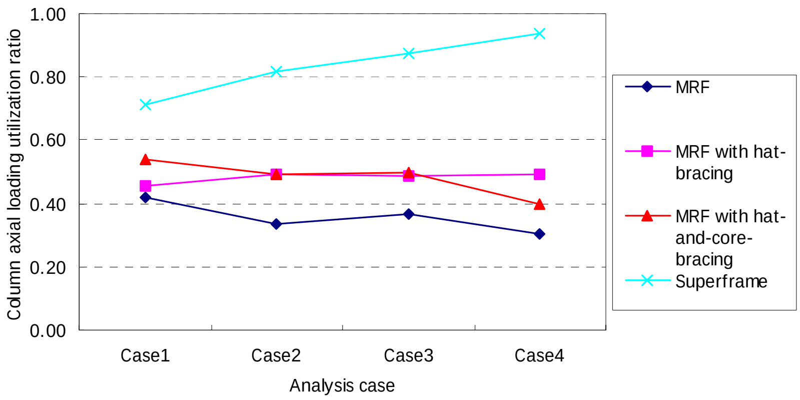


Fig.12 Collapse axial load utilization ratio versus cases for main member loss

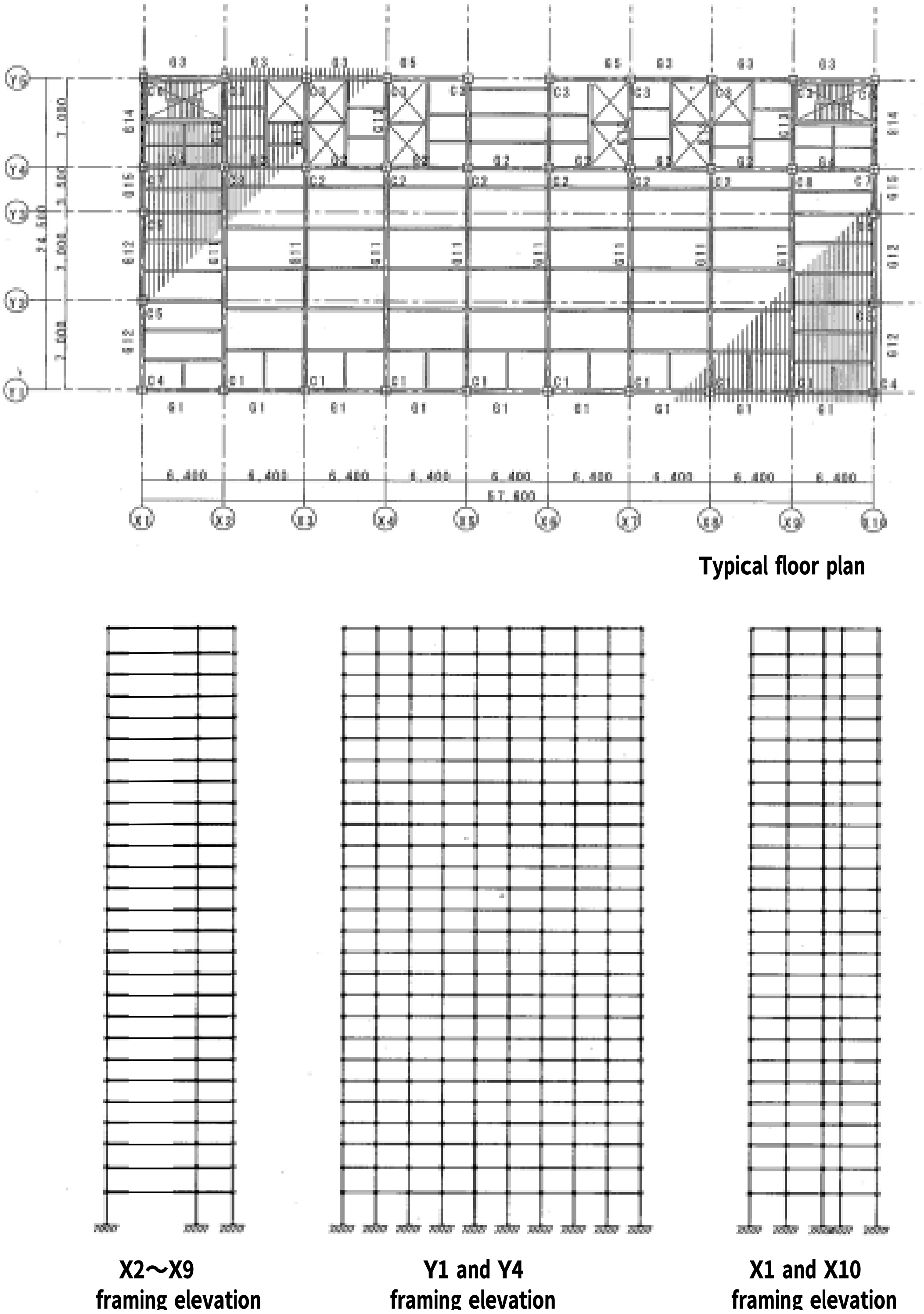


Fig.13 Floor plan and elevation of target building

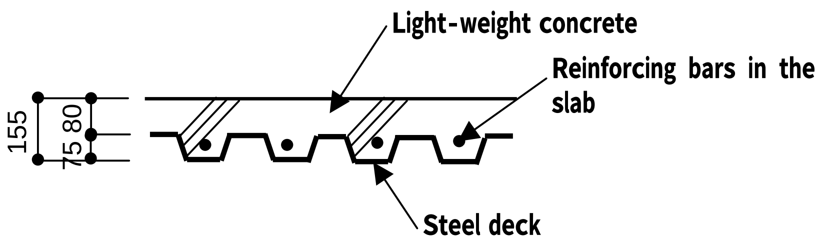


Fig.14 Section plan of floor slab of target building

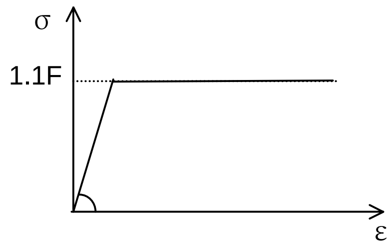


Fig.15　Stress-strain relationship of material

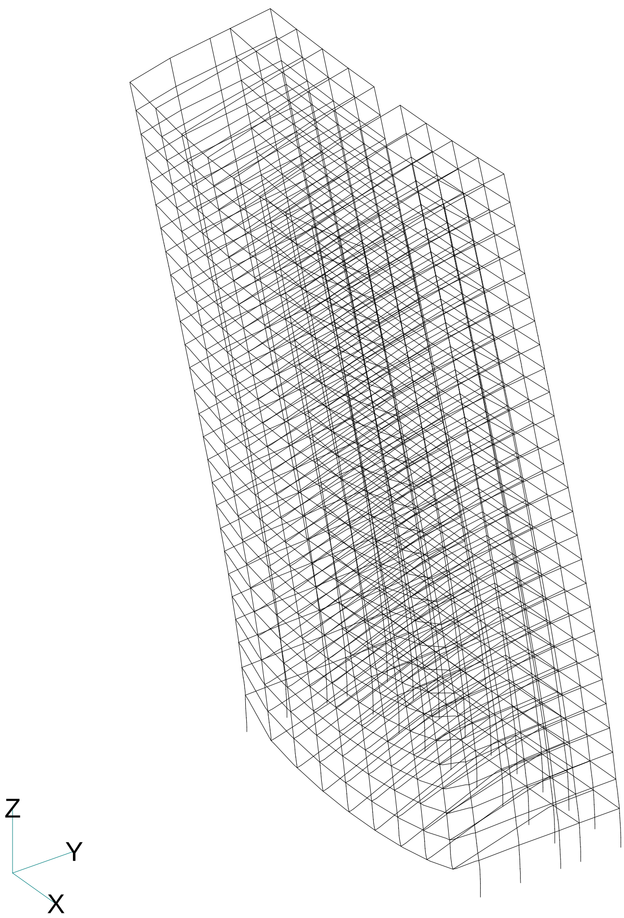


Fig.16　Analysis results：Distortion of frame（case1, interior 8 columns lost on the 1st floor＝Collapse）

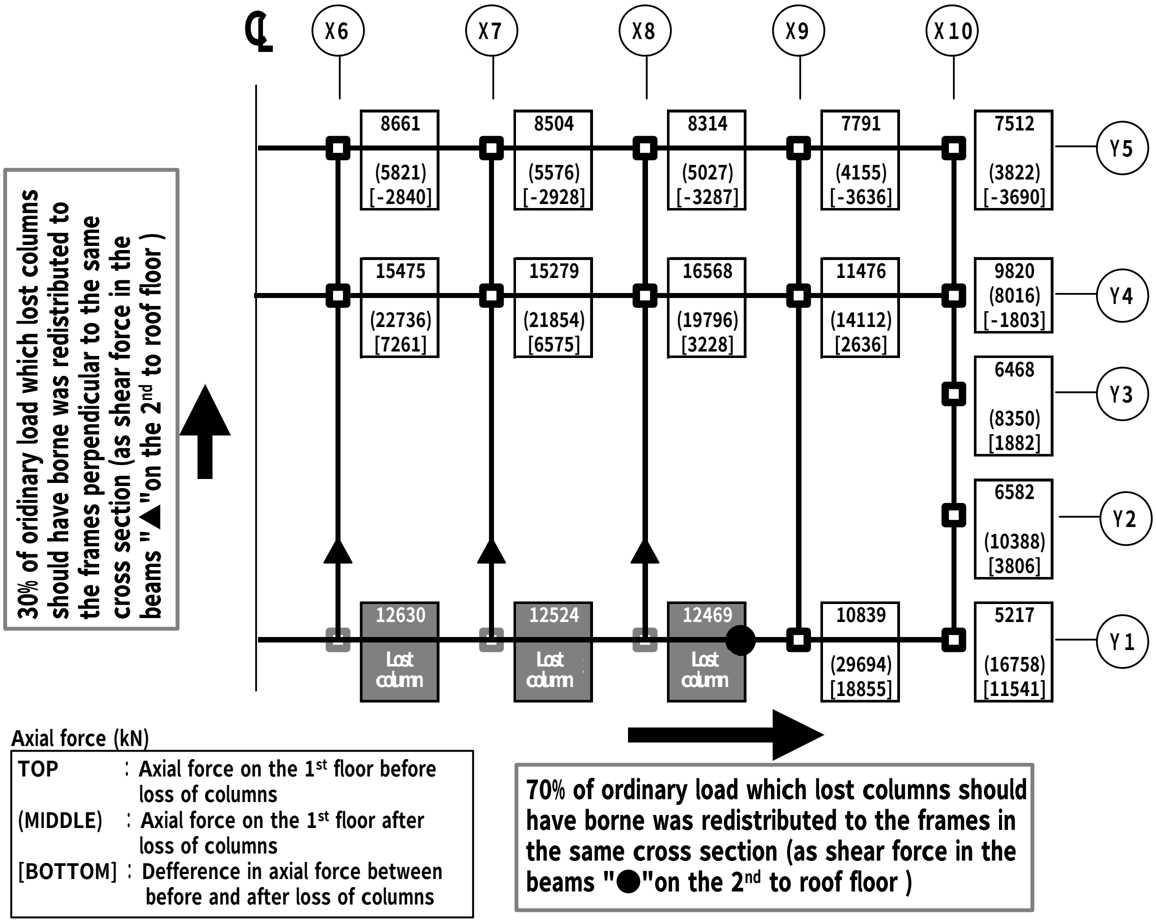
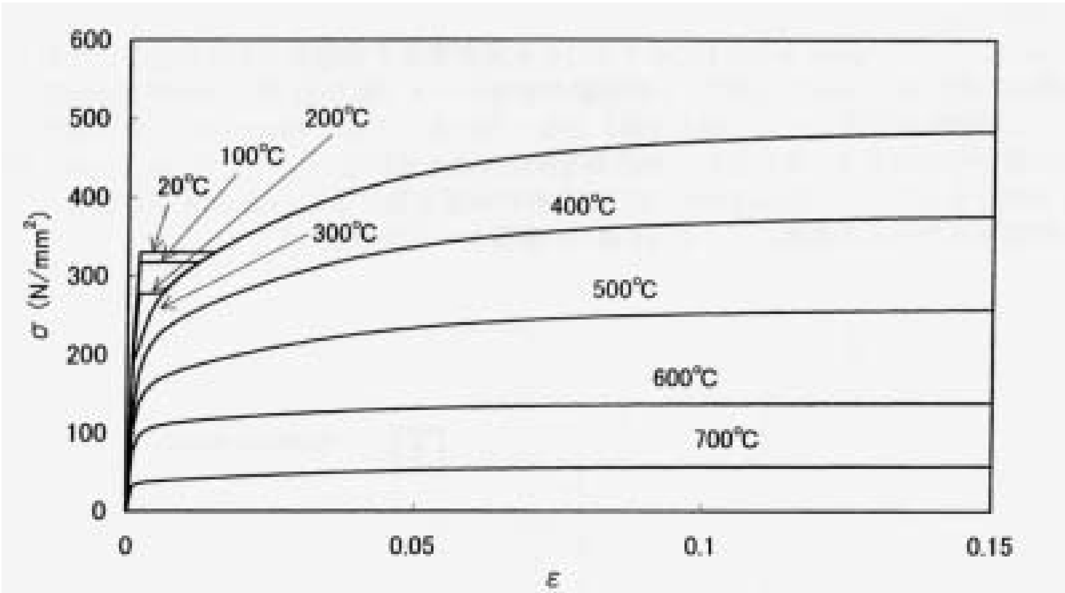


Fig.17 Redistribution of ordinary vertical load（case1, interior 6 columns lost on the 1st floor）



$$\sigma(\varepsilon) = \max\{\sigma_1(\varepsilon), \sigma_2(\varepsilon)\}$$

$$\sigma_1(\varepsilon) = \min\{E \cdot \varepsilon, \sigma_{yt}\}$$

$$\sigma_2(\varepsilon) = f_1(\varepsilon) + f_2(\varepsilon)$$

$$f_1(\varepsilon) = \frac{E_0 \cdot \varepsilon}{\left\{1 + \left(\frac{\varepsilon}{\varepsilon_o}\right)^n\right\}^{\frac{1}{n}}} \quad , \quad f_2(\varepsilon) = \frac{E_1 \cdot \varepsilon}{\sqrt{1 + \left(\frac{\varepsilon}{\varepsilon_2}\right)^2}}$$

$$E_0 = E - E_t$$

$$\varepsilon_2 = \frac{\sigma_0}{E_0}$$

$$\varepsilon_2 = 0.05$$

$$E = (1.0 - 10^{-6} \cdot T^2) \cdot 210000 \quad [MPa]$$

$$E_t = Lineconnecting(0°C, 4500MPa), (300°C, 4500MPa), (600°C, 600MPa), and (1000°C, 0.0MPa)$$

$$\sigma_{yt} = \begin{cases} where \quad T \le 500°C \\ (1.0 - 4.0 \cdot 10^{-6} \cdot T^2) \cdot 330 \quad [MPa] \\ where \quad T > 500°C \\ 0 \quad [MPa] \end{cases}$$

$$\sigma_0 = Lineconnecting(0°C, 270MPa), (320°C, 270MPa), (600°C, 110MPa), and (750°C, 0.0MPa)$$

$$n_t = 1.5$$

Fig.18 Stress-strain relationship of steel

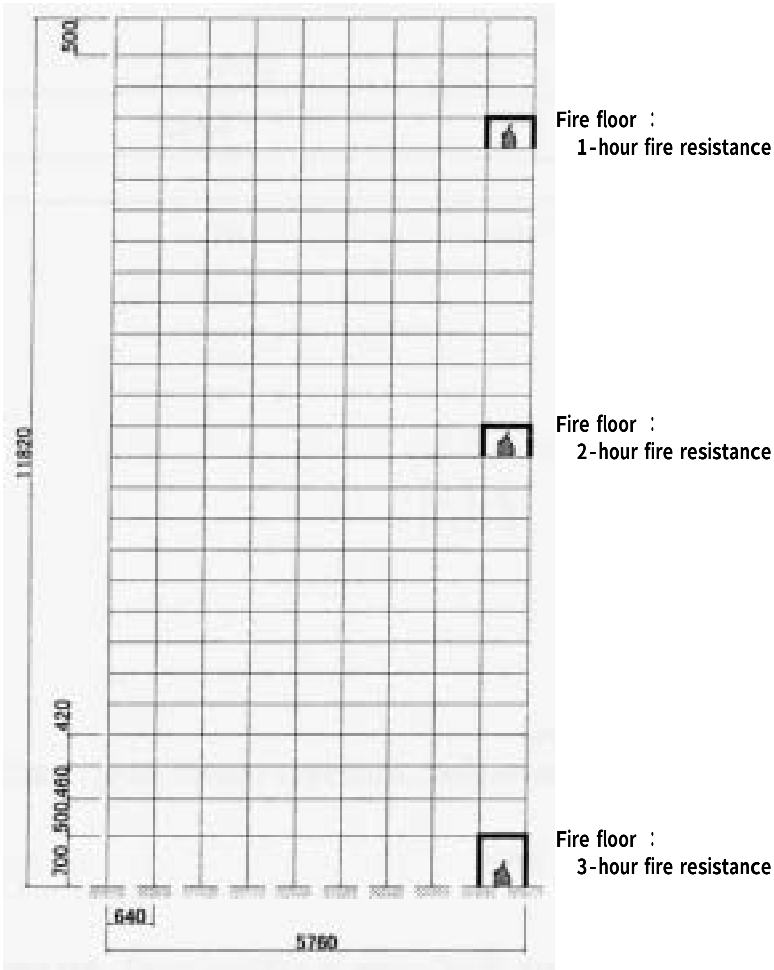


Fig.19　Analytical model（2-D model, showing location of fires）

Fig.20　Fire case（Layer direction）

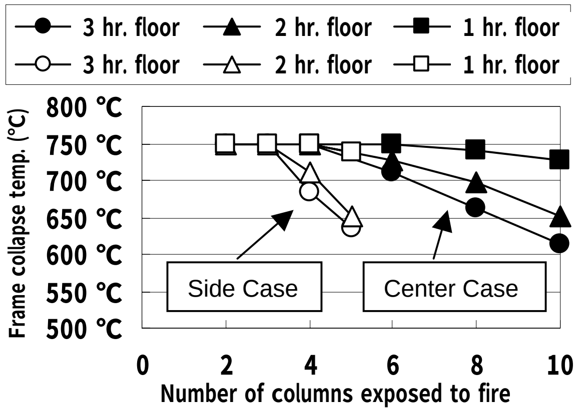


Fig.21 Relationship between frame collapse temperatures and scale of fire

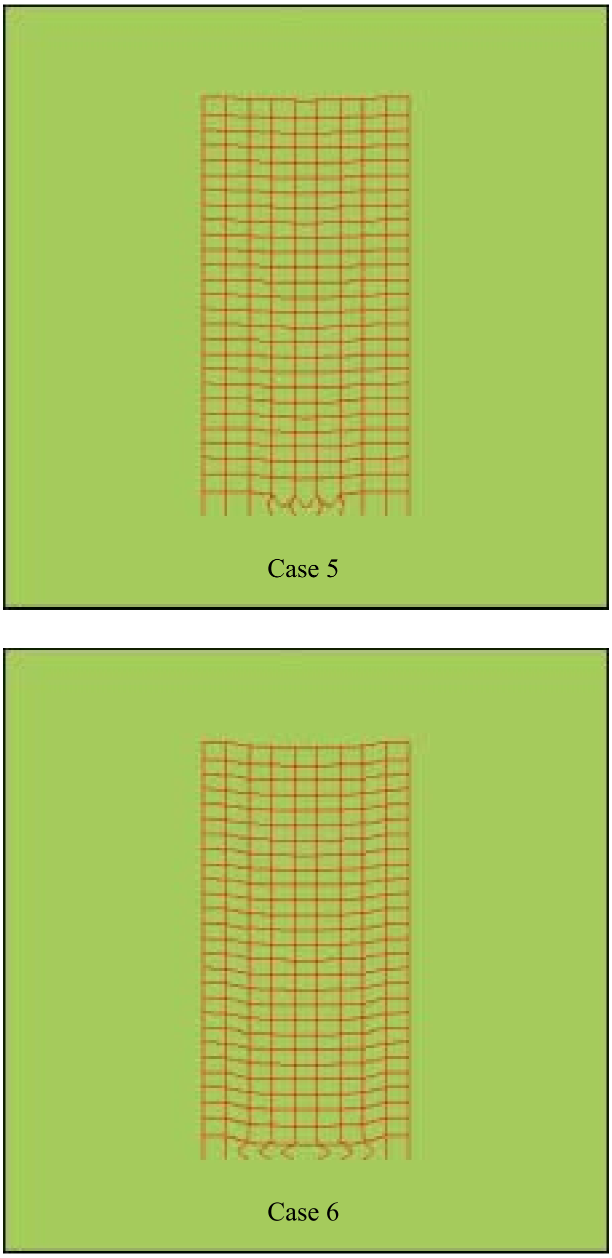
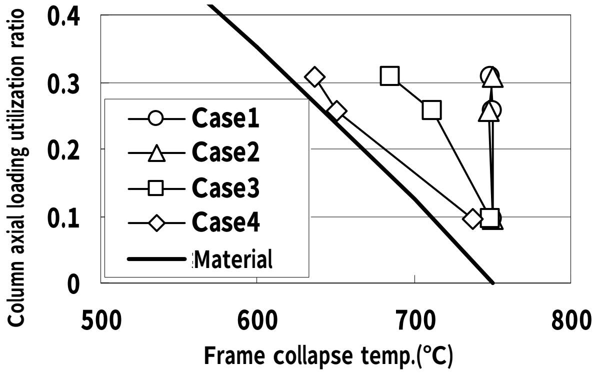


Fig.22　Distortion of frames（case5 and case6）



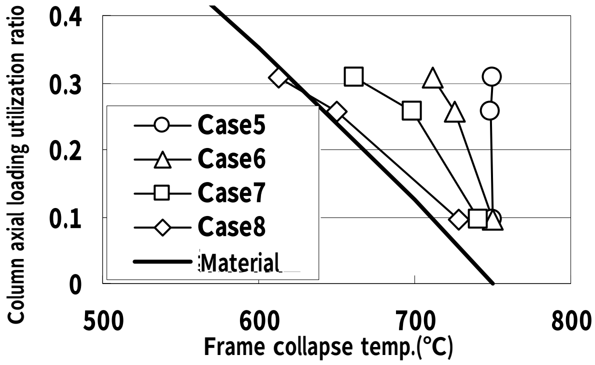


Fig.23 Relationship between column axial load utilization ratios and frame collapse temperatures

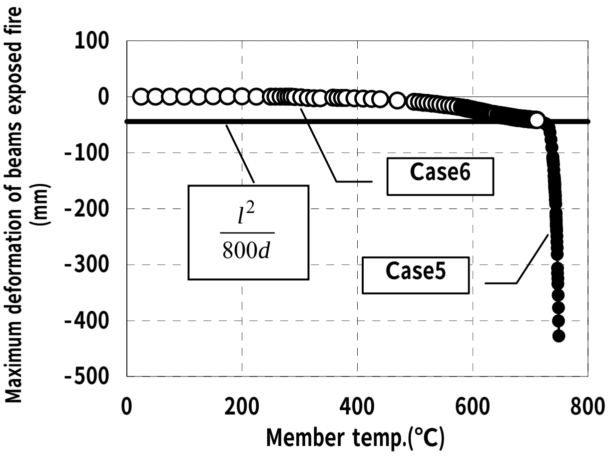


Fig.24 Maximum deflection of beams exposed to fire

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