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Analytical Study of High-rise Steel Buildings in Case of Loss of Columns

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Abstract

This paper presents an analytical investigation into the effect on the redundancy of steel frame structures exerted by the loss of vertical structural members destroyed by aircraft crash and explosions. This examination seeks to estimate the extent of a building's structural redundancy through an elasto-plastic analysis of three-dimensional frames based on the assumption that certain columns of the model building are lost.

A typical high-rise steel-frame office building with a height of over 60 m was used as the model for analysis. Investigations were carried out on member loss at 4 separate locations. As a result, it was found that steel structural frames designed using joints with load-carrying capacity will remain standing even when multiple vertical load carrying members are lost because the vertical loads can be redistributed to the remaining vertical structural members. It was also found that corner columns perform better as "key elements" than other members.

Keywords: structural redundancy; Loss of members; non linear analysis; axial load utilization ratio; rearranged vertical loads

1. Introduction

As one of its major research themes, the Japan Iron and Steel Federation and the Japanese Society of Steel Construction have promoted studies by the Committee to Study the Redundancy of High-rise Steel Buildings to identify the redundancy of high-rise steel buildings in Japan and to propose a frame structure with high redundancy[1]. Specifically, in order to identify differences in the redundancy of high-rise steel buildings, assumptions were made regarding the loss of structural members due to hazards such as explosions and other accidents. Conditions for preventing progressive collapse attributable to these hazards were then examined by means of numerical analysis, setting as parameters the axial force ratio of columns in case when stationary loads act on columns and the frame system (moment resistant frame—MRF, MRF with hat-bracing, MRF with hat-and-core-bracing, super frame). As a result, the following conclusions were obtained.

1) When the loss of structural members is caused by explosions or other accidents, super frame structures that use hysteretic-type dampers to improve seismic resistance possess greater overall frame stability against local collapse than MRF structures.
2) The axial force ratio of columns during stationary loading may prove to be an effective parameter for examining the effect of efforts to suppress progressive collapse. Within the contextual range of the current research, a limit of approximately 0.25 is the criterion for the axial force ratio under stationary loading in order to prevent progressive collapse.

In the paper, an outline of the research program thus conducted is introduced along with the above research. The redundancy required to compensate for column loss was quantitatively identified by means of numerical analysis that took as an example a high-rise steel building that was actually designed in conformity with the seismic code of the Building Standard Law of Japan.

2. Analysis of the Behavior of Actual Japanese High-rise Steel Buildings against Unexpected External Force (Loss of Members)

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

Specifically, as regard the model in which the columns and beams were replaced with wire, nonlinearity is given to the model; then the columns are removed one by one and it is confirmed that to how much degree of column loss the model after the loss of columns can support the vertical load. During this confirmation, geometrical nonlinear effect in addition to nonlinear material is taken into account as

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nonlinearity. Fig. 1 shows the concept of local column collapse.

Employing a three-dimensional model in this examination, three-dimensional confirmation is made regarding the condition of the vertical load that was previously borne by lost columns and that is now redistributed to and borne by the remaining structurally sound members of the frame.
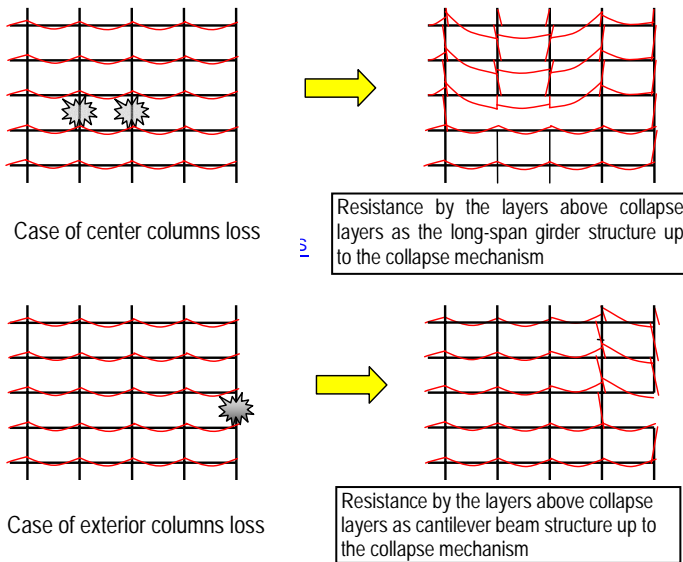

Fig. 1. Resistance by Remaining Frames during Column Collapse

2.1 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. 2 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 extends 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. Table 1 shows the section of frame members.

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. 3. 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.
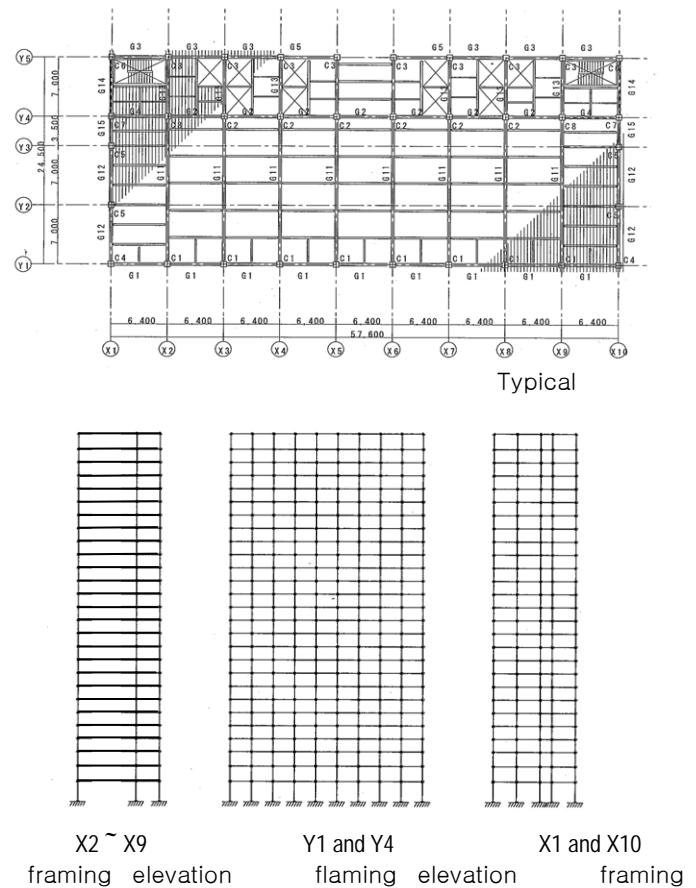

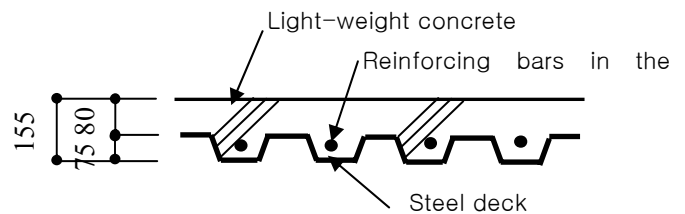**Fig. 2.** Floor Plan and Elevation of Target Building

Fig. 3. Section Plan of Floor Slab of Target Building

Table 1. Section of Frames Members

Member	Section	Standard	n
Column	☐-650×650 or ☐-750×750 Thickness : 25 ~ 45 mm	JIS G 3136 SN490C	0.020 ~ 0.365
Beam	H-600×350 ~ H-1150×300 Thickness of flange : 25 ~ 36 mm Thickness of web : 16 ~ 22 mm	JIS G 3136 SN490B	—

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.

2.2 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. 4. In this case, it was decided that the yield point would be 1.1 times the specification material strength F.
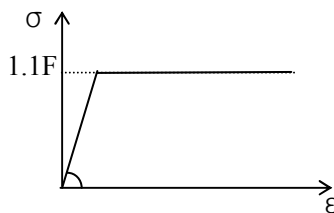
Fig. 4. Stress-Strain Relationship of Material

As shown in Fig. 5, the working load is to be set as the stationary vertical load integrated to the column positions on each floor, and static incremental analysis is conducted in which the vertical load is increased gradually by applying the load 1/50 the stationary vertical load as one step.
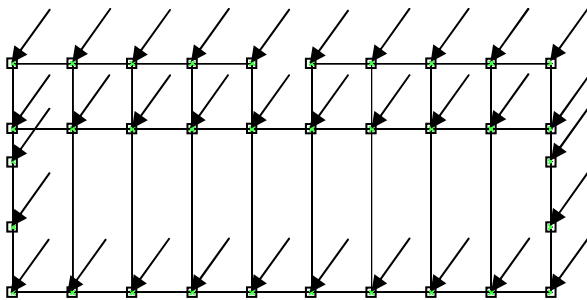
Fig. 5. Arrangement of working load in analysis

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, as shown in Fig.6.

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.
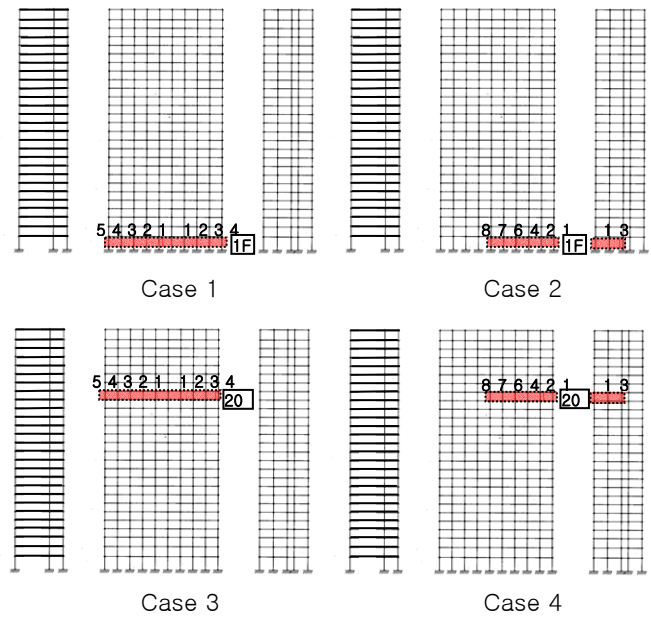

Fig. 6. Location of Member Loss

2.3 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. Distortion of frame at collapse in Case 1 is shown in Fig. 7 (a).

(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. Distortion of frame at collapse in Case 2 is shown in Fig. 7 (b).

(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. Distortion of frame at collapse in Case 3 is shown in Fig. 7 (c).

(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. Distortion of frame at collapse in Case 4 is shown in Fig. 7 (d).
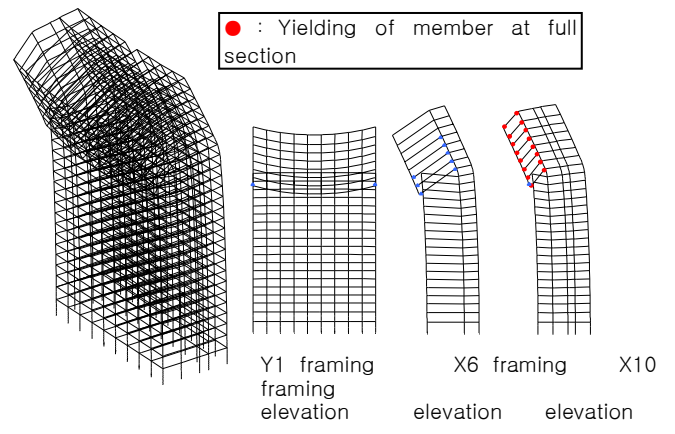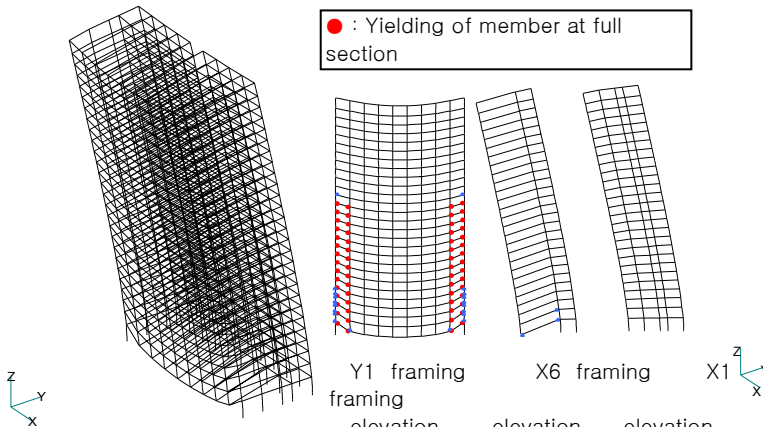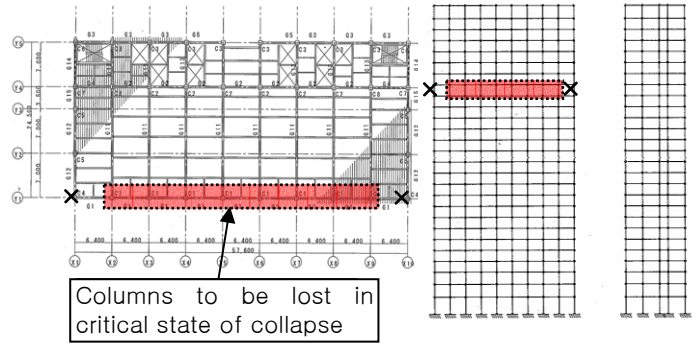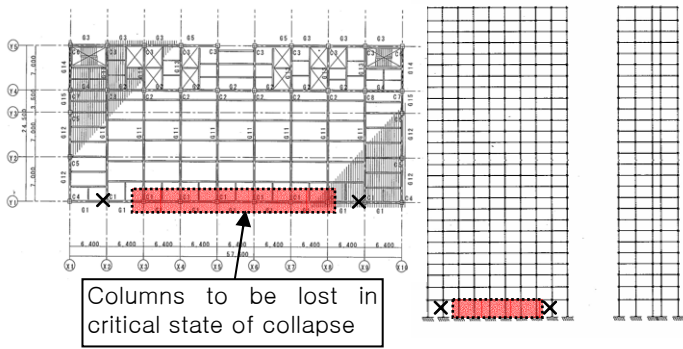
Fig. 7(a). Analysis results: Distortion of Frame (Case 1, loss of interior 8 columns on 1st floor = Collapse)

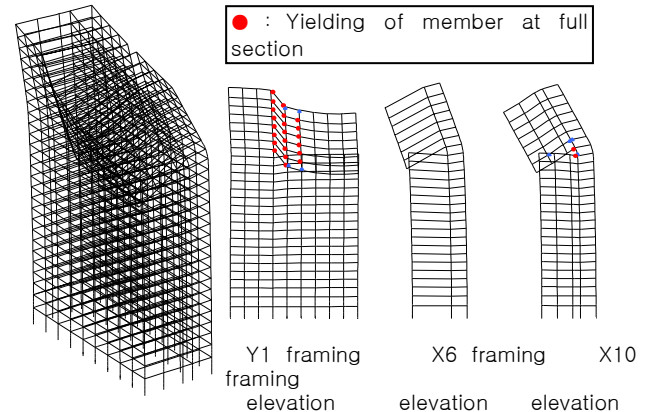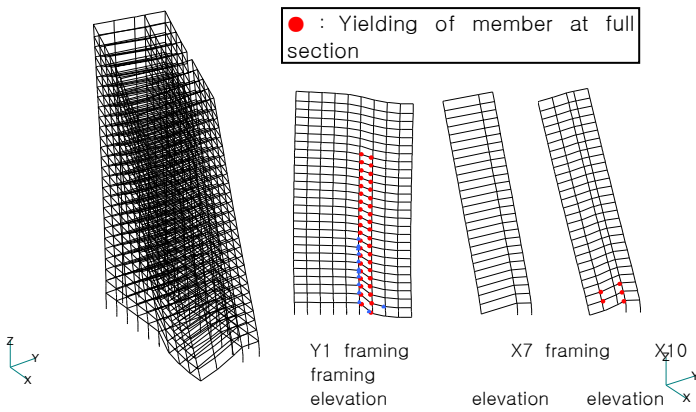**Fig.7(c).** Analysis results: Distortion of Frame (Case 3, loss of interior 10 columns on 20th floor=Collapse)
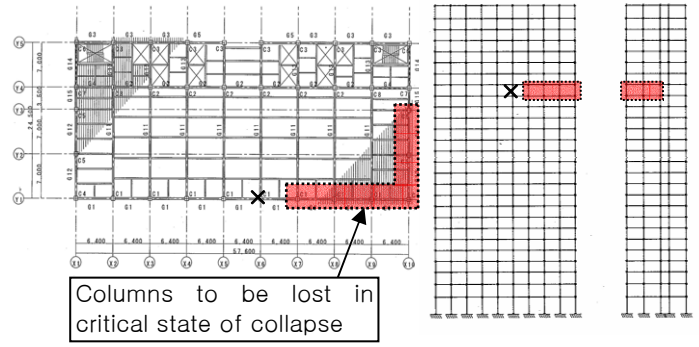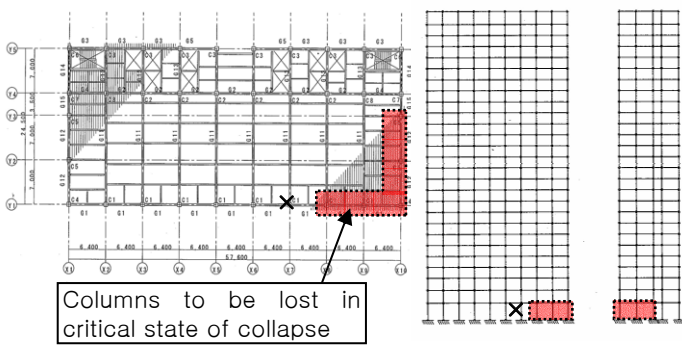
Fig. 7(b). Analysis Results: Distortion of Frame (Case 2, loss of exterior 6 columns on 1st floor = Collapse)

Fig.7(d). Analysis Results: Distortion of Frame (Case 1, loss of interior 7 columns on 20th floor = Collapse)

In addition, the member stress figure in a critical state of collapse in Case 1 is shown in Fig. 9 (a)~(d). Further, redistribution of ordinary loads induced by lost columns, which was estimated based on the axial force exerted on the first floor under the collapse critical state in Case 1, is shown in Fig. 8.
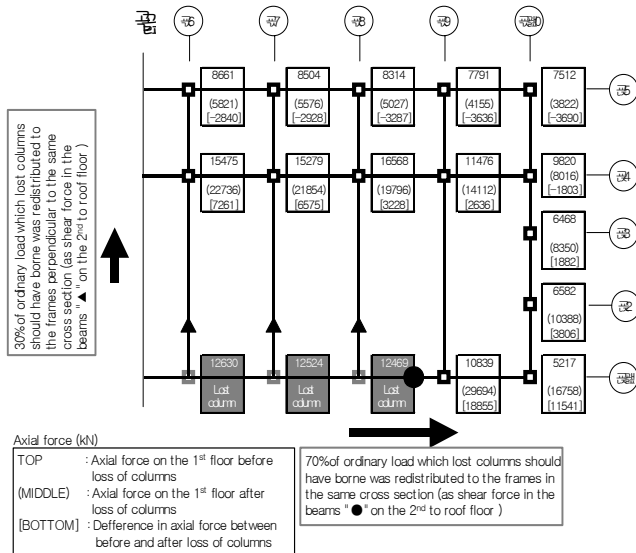

Fig. 8. Redistribution of Ordinary Vertical Load (Case 1, loss of interior 6 columns on 1st floor)
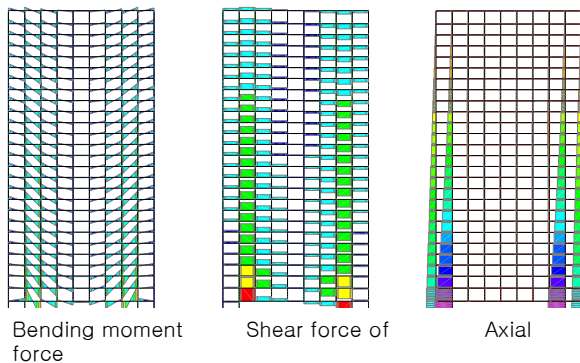

Fig. 9(a). Stress Figure of Y1 Framing (Case 1, loss of interior 6 columns on 1st floor)
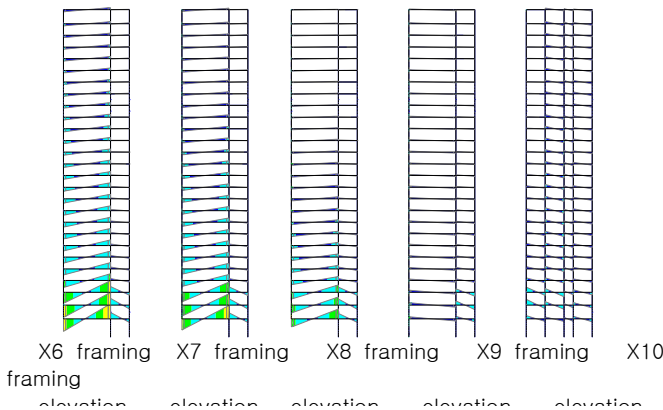

Fig. 9(b). Short-side Direction Bending Moment Figure of Girders (Case 1, loss of 6 interior columns on 1st floor)
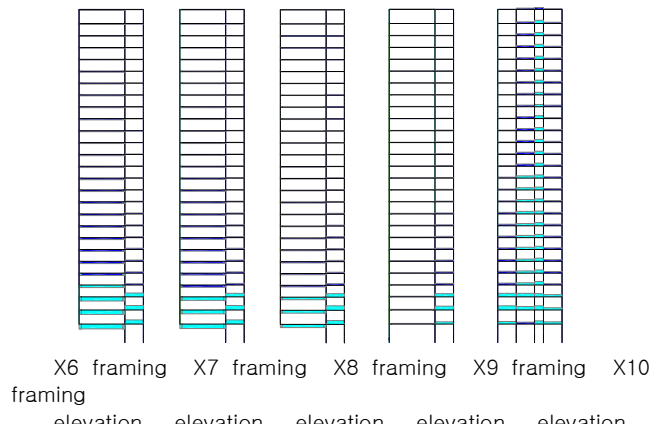

Fig. 9(c). Short-side Direction Shear Force Figure of Girders (Case 1, loss of 6 interior columns on 1st floor)
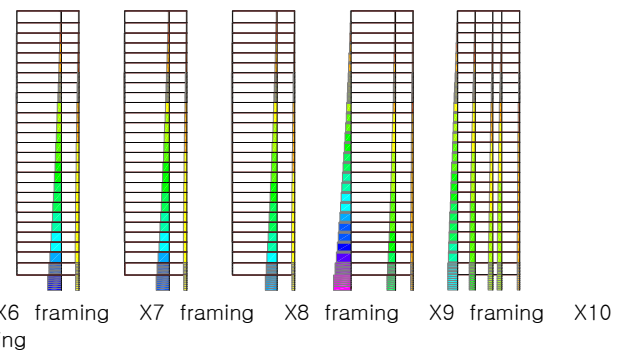

Fig. 9(d). Axial Force Figure of Columns (Case 1, loss of interior 6 columns on 1st floor)

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 that 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 girder. 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 girder can be found to contribute to also redistribution of ordinary vertical load and produce promising three-dimensional effects during great deformation.

2.4 Discussion

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 girders, the building was confirmed to be able to support ordinary vertical loads following loss of 15 to 20% of all columns. Next, we consider analysis results in the following 1), 2) and 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 girder 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) Structural 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 to 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 building with structurally high redundancy.

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. Trial Calculation of Critical Load Supporting Capacity of Long-span Girders

In case of general MRF-structure steel buildings, static stationary vertical loads are transferred in the order of slab→beam→girder→column→foundation structure. From the aspect of entire collapse of building, as the load transfer order drops behind the others, the structural importance increases, and the collapse of girders leads to the collapse of entire floor structure, thereby offering high possibility of the occurrence of entire collapse of buildings. Due to the growing use of long-span girders in recent years, the risk of the occurrence of such collapse is growing.

To meet the situation, non-linear analyses were conducted on the plane frame of the identical building targeted in the previous section to examine 1) to what extent the frame can resist in case when the excess load, which is greater than the stationary vertical load assumed in the design, is applied to the long-span

girder and 2) whether or not there is possibility of the occurrence of entire collapse; and then trial calculation was made on the collapse mechanism.

3.1 Examination Targets and Analytical Cases

Fig. 10 shows the target building subjected to analysis. Fig. 11 shows the model used for analysis. As shown in the figure, the model is the two-dimensional partial plane model incorporating 17.5 m-long long-span girders on the 8th floor. The section from the 6th-floor column head to the 10th-floor column base was modeled under the situation in which long-span girders at the 9th floor is lost and the distance between supporting ports of exterior columns becomes two times the floor height. Table 2 shows the section of frame members.

The working load used in the analysis is the static steadily-increasing load, in which the load 1/50 the uniform load working in a stationary mode on the 8th-floor long-span girder is applied as one step after application of both the stationary axial force working on the column of the upper section of the model and the stationary uniform load working on the girders other than 8th-floor long-span girders. (Refer to Fig. 12.)

Analyses were conducted by taking into account the nonlinear material and geometrical nonlinearity under the analytical conditions similar to those used in the previous section.

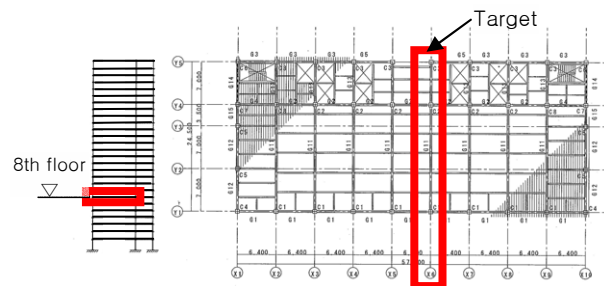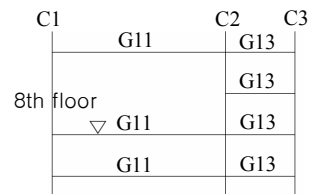**Fig. 10.** Target Building

Fig. 11. Analysis model

Table 2. Section of Frame Members

Column : SN490C	Beam : SN490B
C1 : □-650×650×32	G11 : H-850×300×16×32
C2 : □-650×650×40	G13 : H-850×300×16×28
C3 : □-650×650×28	

Axial force applied from the upper

Stationary uniform load on girder (FIX)

Stationary uniform load on girder (increased)
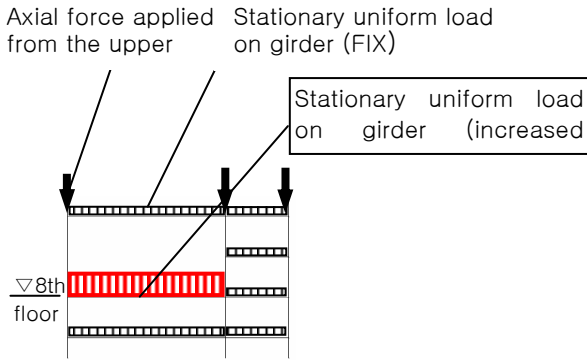
▽8th floor

Fig. 12. Working Load

3.2 Analytical Results

The ratio of the working load to the design vertical load of long-span girders (design floor load: 31.5 kN/m^2) is defined as the load coefficient. That is, the load coefficient indicates how many times greater the uniform vertical load working on long-span girders is to the design load.

Fig. 13 shows the analytical results—vertical displacement at the center of the girders followed by the increase in load (load coefficient), the girder-end bending moment, the horizontal displacement at nodal points on the left-side columns and the girder axis. The figure also shows the deformation figure in terms of loading steps at the occurrence of plastic hinges of the columns.

The following situations were observed from the analytical results.

1) Along with the increase in load, plastic hinges occur at the ends and centers of the girders, thus forming the collapse mechanism of the girders. At the point when the girder collapse mechanism occurs, the load coefficient is 5.82, the girder-end rotation angle is about 1/60 the original angle and the strain is about 3% (A~C in Fig. 13).

2) After formation of the collapse mechanism, tensile axial force occurs in the girders due to the increase in vertical load at their centers, followed by the expansion of plastic rotation in the plastic hinges. The vertical load that can be borne by the girder increases only by the range of the vertical component of force of the girder axial force that is geometrically occurring along with girder's vertical deformation. Further, the horizontal component of force works as an external force (or thrusting force), and as a result, horizontal displacement occurs at the nodal points of girders and columns (C~D in Fig. 13).

3) Plastic hinges occur by the horizontal forces occurring at the nodal points of exterior girders and columns. In this case, the load coefficient is 7.32, and the girder-end rotation angle reaches about 1/15 the original angle (D in Fig. 13).

4) After the first plastic hinge occurs at the column, 3 more hinges occur at the column and the exterior columns cannot keep their axial force. As a result, along with the divergence of deformation, entire frame collapse is caused (D~E in Fig. 13).

3.3 Considerations

The frame subjected to analysis was designed so that the occurrence of girder hinges would precede that of column hinges, and the ratio of the bending strength of long-span girders to that of exterior columns would amount to about two times. Accordingly, in the current frame, because the columns have a sufficiently larger
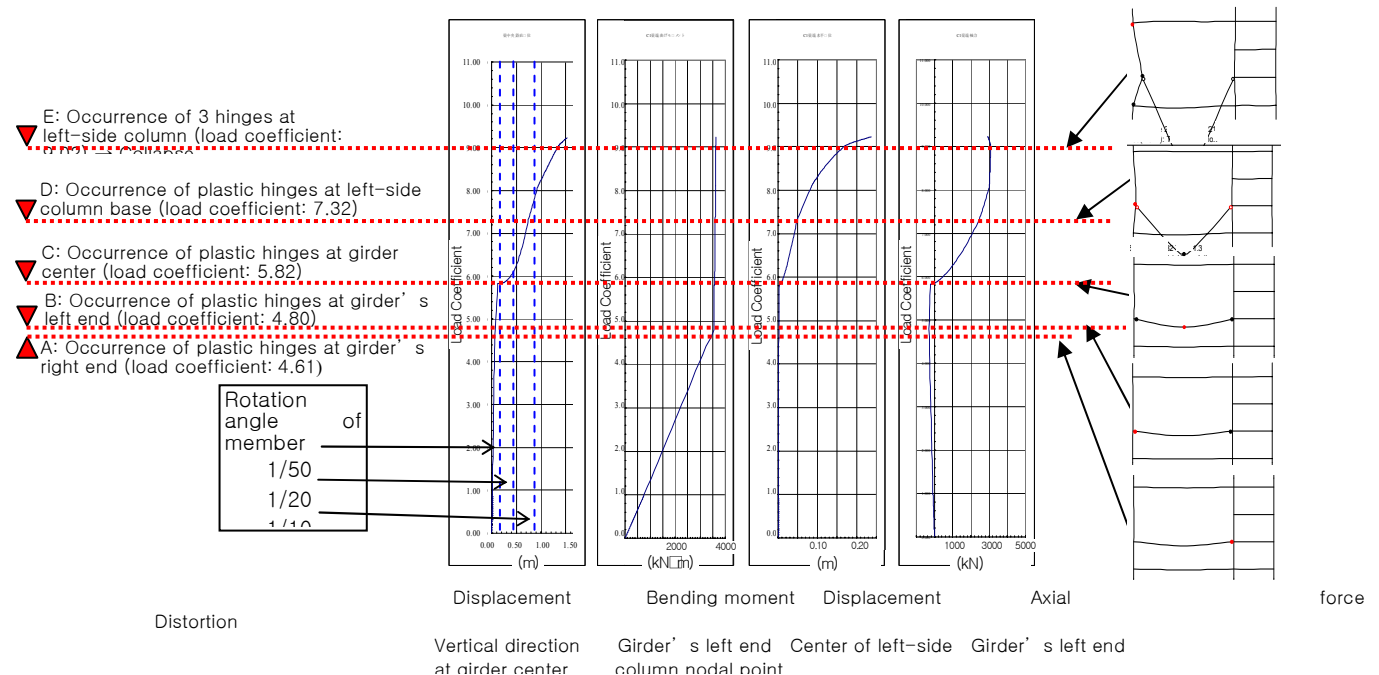

E: Occurrence of 3 hinges at left-side column (load coefficient: ...)

D: Occurrence of plastic hinges at left-side column base (load coefficient: 7.32)

C: Occurrence of plastic hinges at girder center (load coefficient: 5.82)

B: Occurrence of plastic hinges at girder's left end (load coefficient: 4.80)

A: Occurrence of plastic hinges at girder's right end (load coefficient: 4.61)

Rotation angle of member
1/50
1/20
1/10

Distortion

Displacement — Vertical direction at girder center

Bending moment — Girder's left end

Displacement — Center of left-side column nodal point

Axial force — Girder's left end

Fig. 13. Relation between Load Coefficient/Displacement at Nodal Points and Member Stress

strength than the girders, it is considered that girder strength governs the entire strength of the frame.

After occurrence of the girder collapse mechanism, axial forces begin occurring in the girders, and the load supporting capacity is improved due to this effect. However, as girder deformation increases, the girders enter into a range where flange fracture or local buckling of girders occurs, when examining the rotation angle of the girders.

In the current analysis, member fractures that occur on the tension side and local buckling that occurs on the compression side, followed by plastic deformation, are ignored. Further, the sliding of girder-end high-strength bolts, the shear yielding and fracture of gusset plates and the effect of diaphragms and other panel zones are not taken into account. If these factors were taken into account, it is understood that the collapse of single girders would precede the entire collapse of the current frame. To this end, it is considered valid that the most critical condition for long-span girders is the time that plastic hinges occur at the girder end and center (or the occurrence of the girder collapse mechanism) in design. Even in such cases, it is understood that the adoption of materials and weld-joint details that do not cause brittle fracture and the setting of as many web bolts as possible beyond provisional standards are important factors in securing the redundancy of buildings.

4. Conclusion

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) that is not assumed in the design, and identified its characteristics.

The following results were obtained from the examination.
1) 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.
2) If the region in which columns are lost is closer to the exterior of a building, the structural redundancy of the building will be reduced.
3) It is considered valid that the most critical condition for long-span girders is the time that plastic hinges occur at girder end and center (or the occurrence of the girder collapse mechanism) in design.

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