기둥손실에 따른 철골프레임 잔존내력의 해석적 평가

Analytical Evaluation of Residual Strength for Steel Frame in case of Column Member Loss

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요 지

구조물의 우발적인 붕괴가 발생할 경우, 기둥 또는 기둥군(群)에 낙하물에 의한 충격이 가해지게 된다. 낙하물의 충격하중은 기둥부재의 하중변형관계에 따라 소성변형에너지로 흡수가 가능하다. 진행성 붕괴를 방지하기 위해서는 기둥부재의 에너지 흡수 능력이 상시지지 하는 연직하중과 낙하물의 충격하중을 합한 연직하중보다 커야 한다. 이를 위해 구조물이 최종 붕괴 상태에 도달되는 전 과정에 대한 기둥부재의 하중변형관계를 명확히 파악할 필요가 있다. 본 논문에서는 1층 4경간 평면철골프레임의 비선형유한요소해석을 실시하여 기둥부재의 우발적 손실에 대한 에너지 흡수 능력을 평가하였다. 또한, 극한해석을 실시하여 연직하중의 저하 정도를 비교·검토하였다.

핵심용어 : H-형강 기둥, 잔존내력, 소성변형에너지, 비선형FEM

Abstract

When impacts by falling objects are applied to the structures, vertical resisting member(column or column group) results in progressive collapse. By knowing clearly load-deformation relationship of a structural frame, to prevent progressive collapse by absorbing potential energy of falling objects though column groups are lost by the impact of falling object accidently. If residual strength in vertical direction exceeds vertical load, which the sum of the weight of falling objects and usual supportive vertical load as the result of absorbing released location energy, it does not result in progressive collapse. On the other hand, in case when weight of falling objects is included in usual supportive vertical load. In this paper, 1-story 4-spans model is analyzed by non-linear FEM and to examine the level of deterioration, limit analysis of 1-story 4-spans plane frame was carried out.

Keywords : *h-shaped steel column, residual strength, plastic deformation energy, non-linear FEM analysis*

1. Introduction

When load carrying members of the structure are exposed to abnormal loads such as shocks caused by explosion, part of or all of the structural members are collapse or disappeared, this phenomenon is refers progressive collapse of the structure(Nair, 2003). Many researchers are interested in this field specially after: Progressive collapse of Ronan Point apartment in the UK in 1968, the collapse of US Federal Office Building in Oklahoma by the car bomb incident in 1998 and progressive collapse of the 2001 World Trade Center by air craft crashes caused by terrorism action(Ohi *et al.*, 2003).

Accidental collapse of a structure has an impact on columns or column groups by means of falling objects. Column members on the lower part are damaged due to shocking caused by collapse at the

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upper part. For this case, many researchers have studied how to prevent progressive collapse although columns on the lower part are damaged(Park, 2011; Ohi *et al.*, 2005).

Ohi *et al.*(2003) presented the condition where the shocks of falling object has not cause the vertical load carrying structural members(Column member) to progressive collapse. Lee *et al.*(2009) presented the behavior of column-removed double-span beams in welded steel moment frames and proposes a simplified nonlinear dynamic analysis method for the preliminary evaluation of progressive collapse potential. Lee(2010), the residual axial resistance of steel compressive members damaged by blast loading was precisely evaluated using the explicit-to-implicit finite element analysis method.

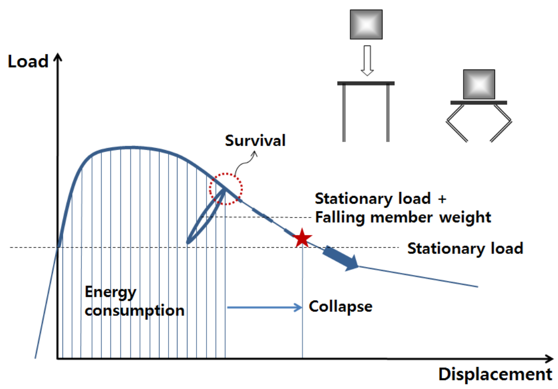
To develop a model for accidental prevention performance assessment reliable for enhancing safety of a structure; there is a great need to assess actual behavior of structural damage due to accidental loads. Therefore, in this study, we analyzed the non-linear finite element of a 1-story 4-spans steel frame to assess energy absorption for accidental loss of columns. And also we did limit analysis to compare and examine the level of decrease of vertical load carrying capacity.

2. Non-linear finite element analysis of post-bucking behavior of steel frame

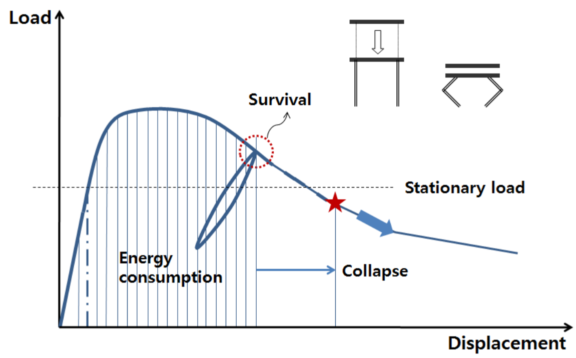
2.1 Condition for preventing progressive collapse

It is necessary to identify the relationship of load-deformation of a column member in the entire process of a structure up to final collapse. Through the process, although a part of the column is accidentally damaged, it is possible to prevent progressive collapse. The condition is described in Fig. 1.

It is possible to absorb the free potential energy of falling objects as plastic deformation energy as shown in Fig. 1(a) according to the relationship of



(a) Case that the weight of falling objects and the stationary load are considered separately



(b) Case that the weight of falling objects is included in the stationary load

Fig.1 Limit that the single storey does not collapse due to the impact of falling objects

load-deformation of a column member. The column is buckled when the strength exceeds the plastic limit. However, as a result of energy absorption; if the residual strength is greater than the vertical load which is the sum of stationary vertical load and the weight of falling objects, progressive collapse is not caused. Meanwhile, the case that the weight of falling object is included in the stationary vertical load as shown in Fig. 1(b). In this case, it was assumed that uniform impact is exerted on all columns.

2.2 Analysis model

Non-linear finite element analysis was performed for plane steel frames to assess accidental loss of a column member by LS-Dyna. The analysis model is shown in Fig. 2 to model the 1-story 4-spans plane frame in which 5 columns of H-100×100×6×8 were

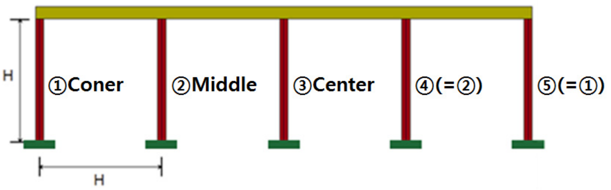


Fig. 2 Analysis model of 1-story 4-spans plane frame

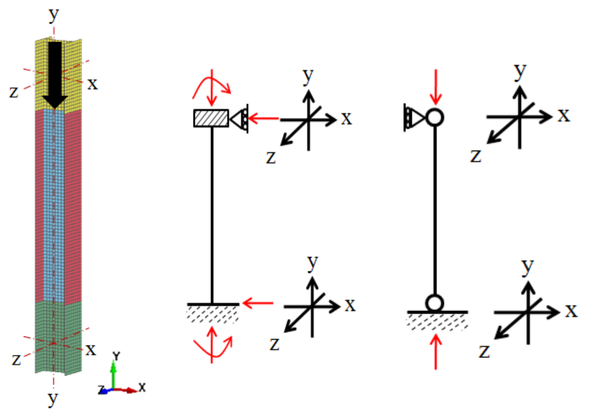
arranged with an assumed rigid body on top, as a shell element.

In the analytical model, the nonlinear inelastic, plastic kinematic characteristics of the materials model was used. In addition, to take into account the strain hardening characteristics of material, a kinematic hardening model was used. The contact between rigid body and specimen is defined as a Tied Shell Edge to Surface contact. After creating extra nodes between the joints of the specimen and rigid bodies and setting array parameters, vertical loads are applied to the upper rigid body fixing the lower rigid. Then setting solution time, the outputs and all other required variables to find the exact solution.

The constraint condition of the upper rigid body was modeled to be the following 2 types as shown in Fig. 3. The first condition was to model the case of constraining rigid beam rotation by allowing displacement only in the vertical direction and constraining displacement and all rotations in the other directions(Fig. 3(a)). The second condition was to model the case of free rotation of rigid beams by allowing only the rotation which enables displacement and column buckling in the vertical direction and constraining displacement and rotation in the other directions(Fig. 3(b)).

Assuming sharp increase of loads in collapse, linear incremental loading was applied at 100mm/sec so that axial loads are given to the upper rigid beam in the vertical direction for applying forced displacement.

The material of columns was modeled with SS400 (structural steel), and the upper and lower rigid bodies were modeled with rigid material steel. Properties of each material are shown in Table 1.



(a) Constrained rotation (b) Free rotation

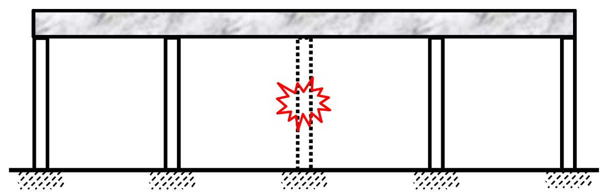
Fig. 3 The constraint condition of column model

Table 1 Properties of material for the analysis models

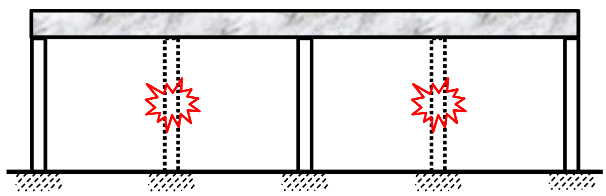
	Column (SS400)	Upper and lower rigid body
Young's modulus(GPa)	205	205
Poisson's ratio	0.29	0.3
Density(kg/mm^3)	7.845×10	7.7×10
Yield stress(GPa)	0.31	-
Tangent modulus(Gpa)	1.0	-
Failure strain	0.23	-

Table 2 Different cases considered for accidental loss of column member

TYPE	I	II	III	IV	V	VI	VII	VIII
Member loss	-	③	④	⑤	③+④	③+⑤	④+⑤	②+④



(a) TYPE II



(b) TYPE VIII

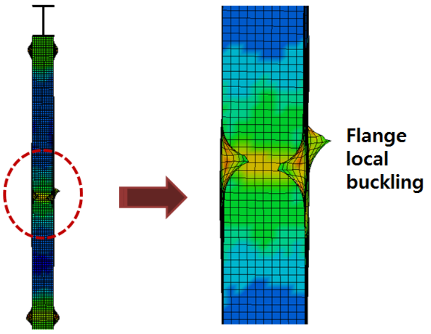
Fig. 4 Different cases considered for accidental loss of column member(TYPE II, TYPE VIII)

Table 2 and Fig. 4 shows the different cases that can be considered for accidentally lost column members.

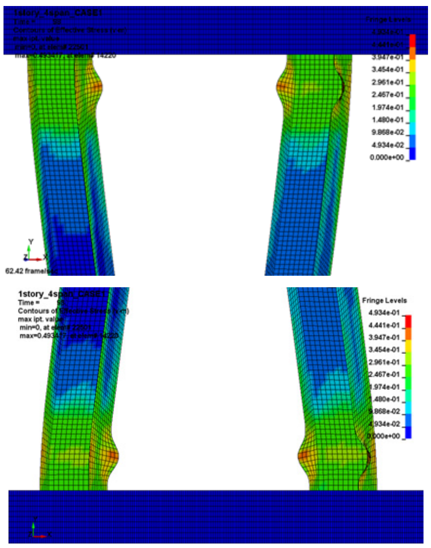
2.3 Result of analysis

2.3.1 Buckling deformation

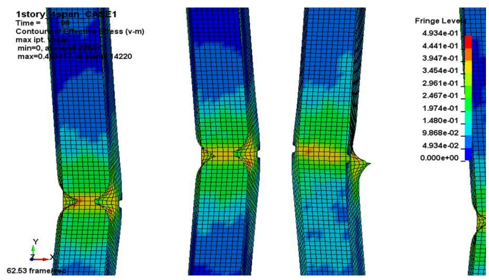
As shown in Fig. 5, buckling occurred in the weak axis although both ends are fixed. Local buckling occurred around both ends in addition to sharp local buckling in the central part of the columns. It is considered that the great slenderness ratio of the columns contributed to plastic hinge and buckling as described below.



(a) Local buckling of the flange of the columns



(b) Buckling in the end of the columns



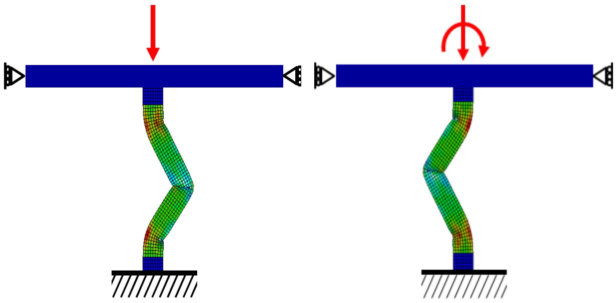
(c) Buckling in the center of the columns

Fig. 5 Buckling deformation of the column

2.3.2 Constrained rotation of rigid body beam

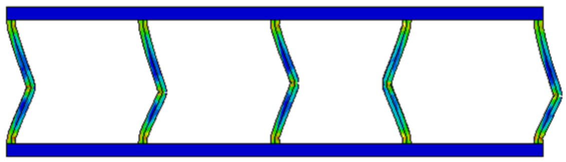
As mentioned in Fig. 3 the two types of the constrained condition of upper rigid beam, free rotation and constrained condition, the idealized constraint condition and buckling mode was shown in Fig. 6 below for both cases.

Fig. 7 and 8 shows the result of analysis of each column loss for the plane steel frame, provided that



(a) Constrained rotation (b) Free rotation

Fig. 6 Constraint condition of upper rigid beam



(a) TYPE Ⅰ

(b) TYPE Ⅱ

Fig. 7 Analysis results of Buckling deformation of the column for constrained beam rotation

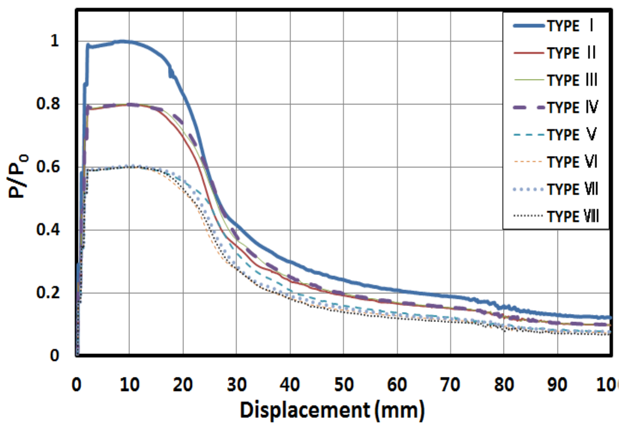


Fig. 8 Relationship of load-displacement provided that beam rotation was constrained

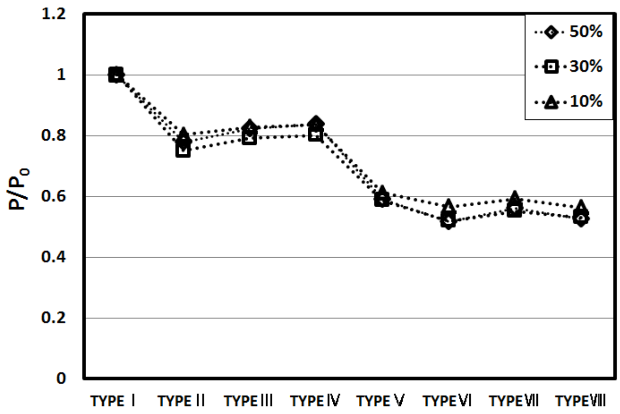


Fig. 9 Energy absorption for each position of member loss(post-buckling 50%, 30%, 10%)

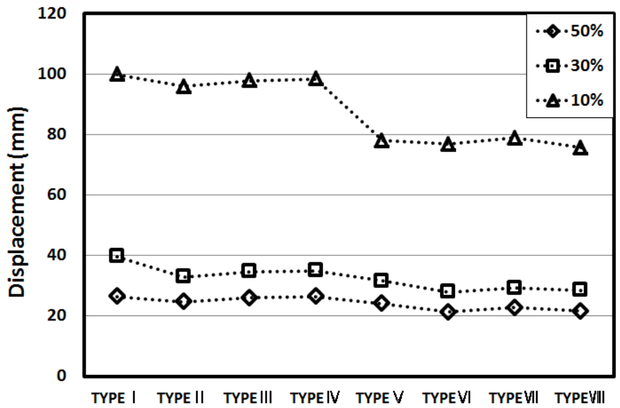


Fig. 10 Vertical displacement at the position of post-buckling strength 50%, 30%, 10% for each position of member loss

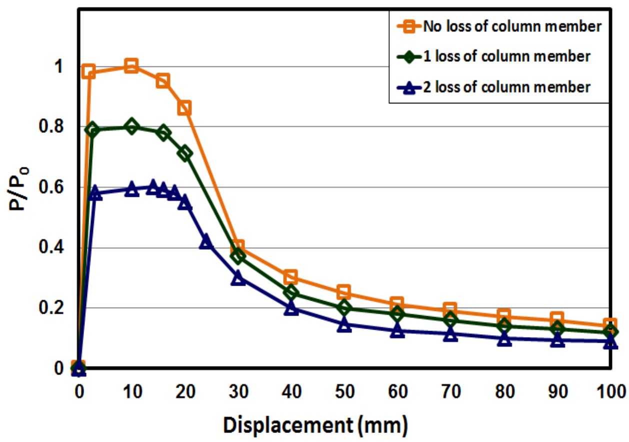


Fig. 11 Decrease in energy absorption depending on the loss of column member

rotation of rigid body beams is constrained.

Fig. 9 and 10 show energy absorption in each position of member loss under post-buckling 30% for each type and vertical displacement under the position of post-buckling strength 30% for each position of member loss.

Displacement changes in the vertical direction were examined depending on column loss while the post-buckling strength is lowered to 30%. Deformation approximately 80-90% occurred in the vertical direction when one column member was lost as compared to the case that the original 5 columns were not damaged. Deformation of approximately 75-80% occurred in the vertical direction when two column members were lost(Fig. 11).

Loss of column members lowered the load transfer capacity to support vertical loads while lowering the deformation capacity. Accordingly, energy absorption capacity sharply decreased on the overall structure.

2.3.3 Free rotation of rigid body beams

Figs. 12~15 show the analysis result of column loss in the 1-story 4-spans plane frame, provided that rotation of rigid body was free.

While the post-buckling strength was lowered to 30%, changes in displacement in the vertical direction caused by column loss were exhibited to be sharply lowered in terms of energy absorption capacity in case of column member loss as compared to the case that the original 5 column members were not damaged as shown in Fig. 14. Unlike the case of constrained rotation of upper rigid body beams, there is a difference depending on positions rather than the number of lost columns. While load transfer capacity for supporting vertical loads for the number of lost column members and each position was lowered, the deformation capacity was lowered to result in decrease in the global energy absorption capacity. However, as shown in Fig. 15, the result of type 8 with 2 column loss is similar to the case of one column loss in terms of vertical displacement. This implies that beam rotation is a variable for energy absorption capacity and displacement.

Energy absorption capacity was assessed on the basis of the relationship between vertical loads and vertical displacement in steel frames for each case of

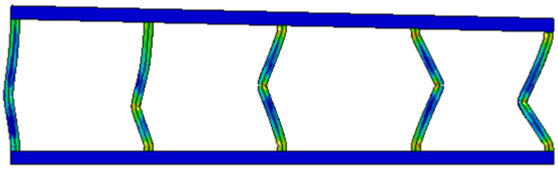


Fig. 12 Analysis results of Buckling deformation of the column for free beam rotation

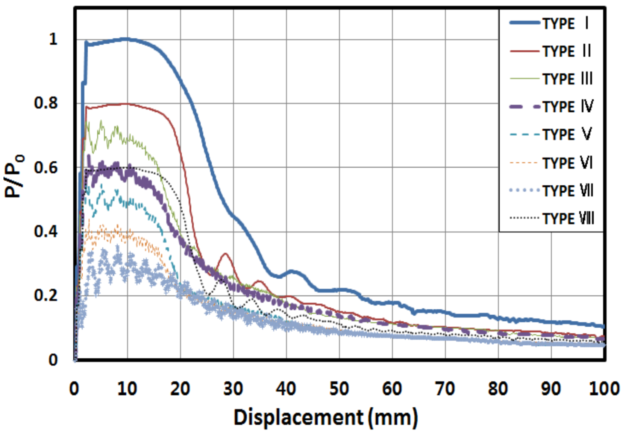


Fig. 13 Relationship of load-displacement for free beam rotation

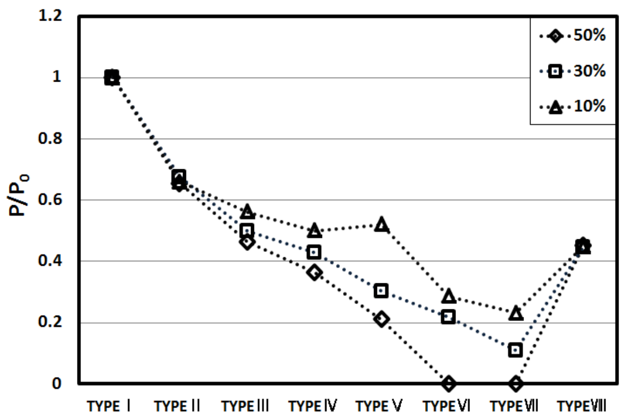


Fig. 14 Energy absorption for each position of lost beams(post-buckling 50%, 30%, 10%)

column loss. The cases of constrained and free rotation of upper rigid body beams were compared. First, constrained beam rotation exhibited decrease in energy absorption depending on the number of lost

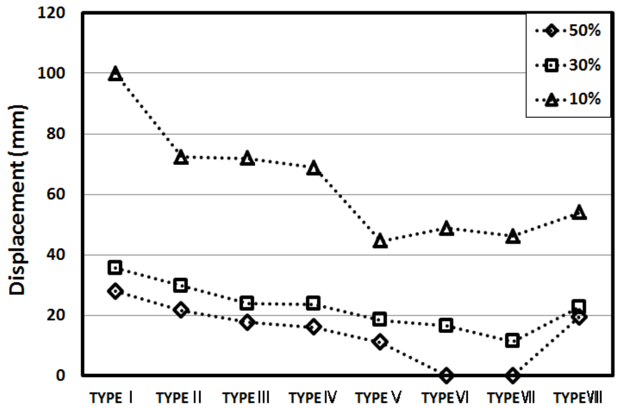


Fig. 15 Vertical displacement at the point of post-buckling strength 50%, 30%, 10% for each position of lost beams

columns regardless of the position of lost columns. A slight difference was exhibited in terms of energy absorption when the same number of columns was lost as well. As known from the type of intermediate and external column loss, it was presumed that the difference resulted from different concentration of arrangement regardless of positional distance between columns. On the other hand, free beam rotation exhibited different changes depending on the number and positions of lost columns. Since beams are a rigid body, they did not experience deflection, but produced results depending on each position by means of the effect of rotation.

3. Assessment of vertical load supporting by means of limit analysis

3.1 Limit analysis

The compact procedure(CP) method proposed by Livesley is a limit analysis that uses a the linear programming based on the lower bound principle (Livesley, 1976).

Based on the static allowable condition in which the equilibrium condition and the plastic condition are satisfied, the load carrying capacity is maximized, and the load value obtained becomes equal to the true collapse load value.

With the CP method, the following problem is solved.

Maximize λ (1)

Subject $\lambda \cdot \{P_0\} = [\text{Con.}] \cdot \{M\}$ (2)

$|M_j| \leq M_{pj}$ (3)

Where is, λ: load factor, $\{P_0\}$: standard value for load vector at nodal point, $[\text{Con.}]$: connection matrix, $\{M\}$: vector of member force(internal force), M_{pj}: member yield strength.

Here, in CP method, all vertical load carrying members are modeled by linear member. It was assumed that plastic deformation is ignored as it is very small compared to elastic deformation, so it is modeled based on perfect plastic behavior of steel. The material property used in limit analysis is the same as property of material used during modeling of nonlinear FEM in chapter 2 that indicated in Table 1.

OHI *et al.*(2003) evaluated the load carrying capacity λ_{damage} in the state in which a certain member disappeared, to load carrying capacity λ_0 in the original state of the framework. They defined its decreasing rate as member sensitivity(Sensitivity Index).

$$\textit{Sensitivity}\ \text{Index (S.I.)} = \frac{\lambda_0 - \lambda_{damage}}{\lambda_0} \tag{4}$$

3.2 Modification of equilibrium equation

In the case that a certain member in the frame disappears, the constraints represented by Equations. (2) and (3) could be modified thusly: the member force that disappears is merely removed from the member force vector. In addition, the corresponding column of the connectivity matrix is also removed. A computer program can accomplish these tasks systematically.

$$\lambda_{damage}\begin{Bmatrix} \\ P_0 \\ \\ \end{Bmatrix} = \begin{bmatrix} C_{11} & \cdots & C_{i1} & \cdots & C_{n1} \\ \vdots & & \vdots & & \vdots \\ C_{m1} & \cdots & C_{im} & \cdots & C_{nm} \end{bmatrix}_{\text{Removed}} \begin{Bmatrix} M_1 \\ \vdots \\ \cancel{M_i}\ \text{Removed} \\ \vdots \\ M_m \end{Bmatrix} \tag{5}$$

3.3 Analysis model

Limit analysis was applied to plane steel frames to examine the level of lowered vertical loads due to accidental loss of each column as shown in Table 2. It is assumed that plane steel frames experience concentrated loads in the center of rigid beams and on top of the columns in each span in the vertical direction. However, it is conditioned that 1/2 load is applied on top of the columns at both ends.

The following is the condition for fixed beams-columns for examining the condition of supporting and joining the analysis models. Firstly, the columns are fixed to a point and steel-joined with a beam. Secondly, if the horizontal force on top of the columns is not considered and vertical loads are applied, each column experiences deflection in a symmetrical buckling mode.

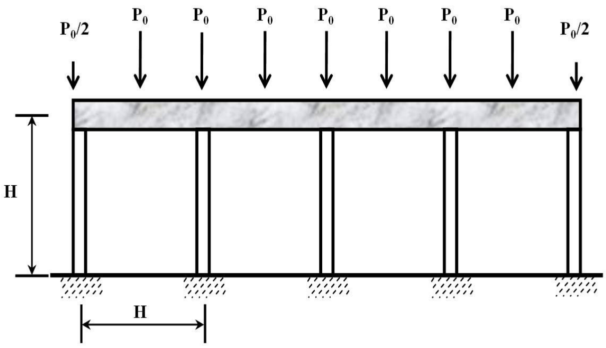


Fig. 16 Analysis model of 1-story 4-spans plane frame

3.4 Result of limit analysis

The result of supporting vertical loads after applying limit analysis for each column loss of the plane steel frame is shown in the following Table 3.

It was observed that the lower capacity of supporting vertical loads was exhibited, the closer to the lost outer columns from the lost central column, with reference to the positions of lost columns on the basis of no lost columns. For the case of each two column loss, it was observed that different supporting capacity was exhibited depending on the position as in the case of one column loss and depending on column continuity. It was observed that loss of the

Table 3 Result of limit analysis

TYPE	Member loss	λ	$\lambda_{damage}/\lambda_0$
I	-	75.625	-
II	③	60.496	0.8
III	④	53.136	0.7
IV	⑤	45.574	0.6
V	③+④	37.961	0.5
VI	③+⑤	30.4	0.4
VII	④+⑤	22.838	0.3
VIII	②+④	45.372	0.6

outer side column was as critical as the loss of 2 columns which were not continuous, in that the supporting capacity of vertical loads by No.8 model was approximately the same as No.4 model.

3.5 Comparison of result of non-linear finite element analysis with limit analysis

Fig. 17 shows comparison of P/P_0 by non-linear finite element analysis with $\lambda_{damage}/\lambda_0$ by limit analysis.

For constrained beam rotation, the analysis result of non-linear finite element exhibited the same load supporting capacity regardless of the position of lost column member for the case of one column loss. However, for free beam rotation, the result of non-linear finite element analysis and the limit analysis exhibited each different value depending on the position of lost column members. This is because it was impossible to carry out exact simulation of coopera-tion of column members remaining after rigid modeling for the upper beam to be uniformly transformed

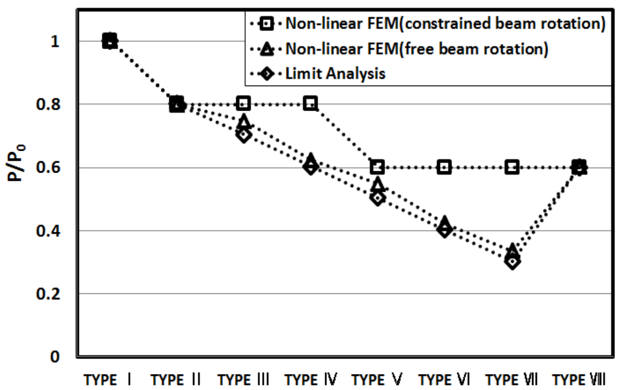


Fig. 17 Comparison of limit analysis with non-linear finite element analysis

(constrained rotation) and lost without rotation of the rigid body with the column collapse type in the non-linear finite element analysis in which the beam was constrained.

However, in the analysis result of free beam rotation, it was possible to assess load supporting capacity more exactly because the rotation effect of rigid body was considered depending on the position of lost columns as in the limit analysis by means of CPM.

4. Conclusions

In this study, we examined changes in lowered energy with respect to column member loss in a 1-story 4-spans strong beam-weak column frame and have the following conclusion.

① Changes in energy absorption after buckling were analyzed with respect to the position and the number of lost column members by means of non-linear finite element simulation of a 1-story 4-spans frame. It was observed that both load transfer capacity and deformation capacity decreases. In addition, the loss of column members and energy absorption capacity of the structure sharply decreased as well.

② As a result of comparing the results of finite element analysis and limit analysis, the rotation effect of the rigid body with respect to the position of lost columns was reflected to enable exact load supporting capacity to be assessed in the non-linear finite element analysis which allows beam rotation and the limit analysis using CPM.

The result of this study for changes in energy by column member loss in this 1-story 4-spans strong-beam weak-column frame will be very useful for future progressive collapse prevention studies on the basis of changes in plastic deformation energy of members.

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