17th World Conference on Earthquake Engineering, 17WCEE

Sendai, Japan - September 13th to 18th 2020

COLLAPSE MECHANISM AND STORY DRIFT OF LOW AND MIDDLE-RISE STEEL MOMENT FRAMES UNDER BI-DIRECTIONAL EXCITATION

K. Hiroishi[(1)], T. Ishida[(2)] , R. Maseki[(3)] , H. Narihara[(4)] and S. Yamada[(5)]

[(1)] *Technology Center, Taisei Corp., M.Eng., hrikuz01@pub.taisei.co.jp*
[(2)] *Assist. Prof., Laboratory for Future Interdisciplinary Reserch of Science and Technology, Tokyo Institute of Technology, Dr.Eng., ishida.t.ae@m.titech.ac.jp*
[(3)] *Technology Center, Taisei Corp., Dr.Eng., ryota.maseki@sakura.taisei.co.jp*
[(4)] *Technology Center, Taisei Corp., Dr.Eng., hiroyuki.narihara@sakura.taisei.co.jp*
[(5)] *Prof., Laboratory for Future Interdisciplinary Reserch of Science and Technology, Tokyo Institute of Technology, Dr.Eng., yamada.s.ad@m.titech.ac.jp*

Abstract

In Japanese structural design, low and middle-rise steel moment frames are generally verified for seismic safety by horizontal load capacity calculation with push-over analysis assuming one-directional force. However, during an actual earthquake, the dynamic behavior and the horizontal bi-directional excitation may cause the building behavior different from the design assumption. Therefore, to evaluate the seismic performance of a steel moment frame, it is important to grasp the collapse mechanism and generated story drift under bi-directional dynamic excitation.

Under bi-directional excitation, in a moment frame, bi-axial bending moment and additional axial force by overturning moment act on the column, which decrease the column strength and causes to form the weak column mechanism as compared with one-directional excitation. While overall sway mechanism in which the beams or panels yield prior leads to a stable plastic deformation behavior where member yielding is dispersed throughout the building, the week column mechanism leads to damage concentration of the specific story and brittle collapse. Previous studies have shown that not only bi-directional excitation, but also the deterioration of column's restoring force due to local buckling and composite effect of beams by concrete slab are factors in the weak column mechanism.

In this paper, as a starting point of research to grasp response behavior of low and middle-rise steel moment frame under horizontal bi-directional excitation, seismic response analyses were conducted for 4-story 2 x 1 span moment frame. For the analyses, to evaluate the collapse mechanism and the maximum story drift precisely, a three-dimensional frame model was used that can evaluate composite effect of beams and panels by concrete slab and the deterioration of column's restoring force due to local buckling. The level of input wave used in the study assumes the plastic design in Japanese structural design.

From the analytical results, the following knowledge was obtained.

1) The maximum story drift angle of 1st story depends on the column strength ratio γ, and can be more than 1/40rad in the story with γ of about 1.0 under horizontal bi-directional excitation. It is considered that the smaller γ of the story, the more bi-directional excitation affects, and the story with sufficiently large γ is hardly affected because yielding of the beam and panel precedes.

2) Even in a frame designed to form overall sway mechanism, there is a high possibility that the story collapse will proceed beyond the design assumption due to horizontal bi-directional excitation and composite effect by concrete slab. In a story with γ of about 1.0, all the ends of the columns can yield and all the both ends of the columns can deteriorate.

Keywords: Steel moment frame, 3D frame model, Bi-directional excitation, Weak column mechanism, Local buckling

1. Introduction

In the structural design of low and middle-rise steel moment frames in Japan, it is common to verify the seismic safety by calculating the collapse mechanism and strength of the building by push-over analysis assuming a uni-directional (1D) load. However, under seismic excitation, the behavior of the building may differ from the design assumption due to the dynamic characteristics of the input wave and the horizontal bi-directional (2D) input.

In the moment resisting frame (MRF) under 2D load, the biaxial bending moment and the additional axial force due to the overturning moment act on the column, and the column strength in the structure plane is lower than under 1D load. Therefore, it becomes easy to form a weak column mechanism. When an overall sway mechanism in which beams or panels yield first is formed, plastic deformation of members is dispersed throughout the building, resulting in stable plastic deformation. On the other hand, if the weak column mechanism is formed, there is a risk that damage will concentrate on the column of the specific story, causing brittle collapse. Therefore, as a condition for forming the overall sway mechanism even when the biaxial bending moment of the column is applied, a limit strength ratio of column to beam or panel is prescribed. According to Japanese design manual of Cold forming square steel tube[1], the strength of column should be greater than or equal to 1.5 times that of the beam or 1.3 times that of the panel.

However, seismic response analyses by Mukaide et al. [2] under 1D excitation with a multi-story MRF model that can evaluate deterioration due to local buckling of columns and beams showed that even if Column-overdesign-factor (COF) which is strength ratio of column to beam is 1.5 or more, a weak column mechanism may be formed under the condition that width-thickness ratio of the columns are classified as FC, which is not guaranteed sufficient plastic deformation capacity in Building Standard Law of Japan. Furthermore, a similar analytical study by Nishino et al. [3] that evaluates the increase in beam strength due to the composite effect of slabs showed that a weak column mechanism may be formed even if COF is 1.5 or more and width-thickness ratio of the columns are classified as FA, which is guaranteed sufficient plastic deformation capacity in Building Standard Law of Japan. On the other hand, Chen et al. [4] conducted response analyses of a three-dimensional frame model considering the biaxial bending of columns under 2D excitation, and proposed that limit of COF to avoid weak column mechanism should be 1.55 and 1.80 for FB (intermediate of FA and FC) and FC respectively, which are higher than the values shown in [1].

Considering these, when evaluating the effects of 2D load, it is desirable to analyze in consideration of the effects of column strength variation such as biaxial bending moment, variable axial force and deterioration due to local buckling, and the effect of relative column strength decrease such as the composite effect of slabs. In [4]-[8], a seismic response considering 2D input is analyzed under a wide range of conditions. However, there are few that have comprehensively considered the above effects, and it's not clear how they correspond to actual behavior. On the other hand, in the simulation analysis by Horimoto et al. [9] of a collapse test of a full-scale four-story steel building [10], using an analytical model that can evaluate the composite effect of slabs on beams and column strength deterioration, the experimental result that the frame collapses while shifting from the overall sway mechanism to the weak column mechanism was simulated with high accuracy.

From the above background, in this paper, as a starting point of research to grasp response behavior of a low and middle-rise steel moment frame under 2D input, seismic response is analyzed on moment frames of 4-story 2 x 1 span, and the effects of 2D input on the collapse mechanism and the maximum response story drift are shown. The analytical model used is a three-dimensional frame model that can evaluate the effect of biaxial bending of columns, the strength deterioration of columns due to local buckling, and the increase in beam and panel strength due to the composite effect of slabs. In addition, in order to ensure the correspondence with the actual behavior, the accuracy is verified by simulation analysis of past experiments.

2. Analytical method

2.1 Analytical model

"Model for design" (shown in Fig. 1) that is close to the current Japanese design assumptions is used in push-over analysis for design of target frame. "Model for dynamic analysis" (shown in Fig. 2) that can evaluate the biaxial bending of columns, the strength deterioration due to local buckling, and the composite effect of slabs is used in seismic response analyses. Details of each are shown below.

(1) Common to both models

- The beam and column members are replaced by beam elements considered for deformations in the axis, shear, bending and torsion directions.
- Rigid base and rigid floor are assumed.
- The beam-column joint panel is modeled with rigid elements and elasto-plastic truss elements. The hysteresis rule of the truss element is normal bilinear with a stiffness decreasing rate after yielding of 0.01 (that of Model for design is 0.0), and the elastic axial stiffness and yield axial strength are equivalent to the elastic shear stiffness and shear strength of the panel.

(2) Model for design

- The beam and column are modeled with beam elements, which the hysteresis model is elasto-perfectly plastic. The bending yield strength of the column is based on the MN interaction. The elastic bending stiffness of the beam is evaluated by adding a stiffness increase rate to that of a steel beam as composite effect of slabs. The rate is 1.5 for one side slab and 2.0 for both sides slab.

(3) Model for dynamic analysis

- The column is modeled with reference to [11] and [12] in order to consider biaxial bending and deterioration due to local buckling. The column ends are modeled by MS (Multi Spring), sixteen spring elements are arranged in the section, and the constant curvature distribution length is set to 0.05 L (L: member length). The spring element are considered for axial and shear deformations. The relationship between force and axial deformations of spring element is defined by associating the cumulative plastic strain with the skeleton curve considered for the deterioration due to local buckling. At the time of unloading, the initial stiffness is used until it returns to the skeleton curve again, and different from [11] and [12], the Bauschinger effect is not considered. The spring for shear deformations is elastic.
- In order to evaluate the increase in beam and panel stiffness and strength due to the composite effect of slabs, the area 400 mm from the panel face is composed of an elasto-plastic beam element as a steel beam, a rigid beam element as a stud, and an elasto-plastic compression spring element as a slab. The compression side hysteresis rule of the spring element as a slab is slip model, and the elastic compression stiffness and yield strength are calculated based on the width of the contacting column face as the effective width. On the tension side, the stiffness and strength are zero. The hysteresis rule of bending of the beam element as a steel beam is normal bilinear and the stiffness decreasing rate after yielding is 0.01.
- The central part of the column and beam is modeled by an elastic beam element, and the elastic bending stiffness of the beam is given stiffness increase rate of 1.5 on one side slab and 2.0 on both sides slab as in the model for design.
- The P-δ effect is evaluated by adding geometric stiffness according to the variable axial force acting on the element.
- The damping is Rayleigh type with the damping ratio 2% at the natural frequencies of the primary mode in the X direction and the secondary mode in the Y direction.

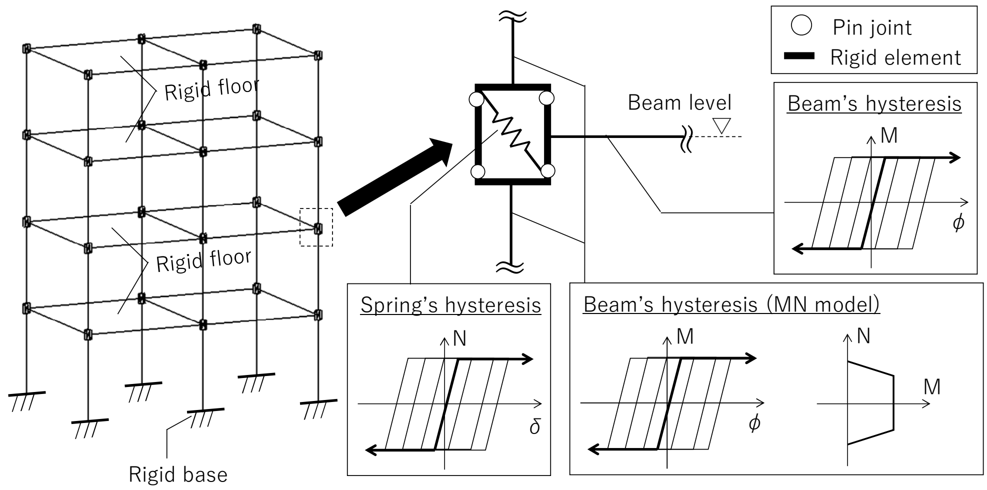


Fig. 1 Model for design

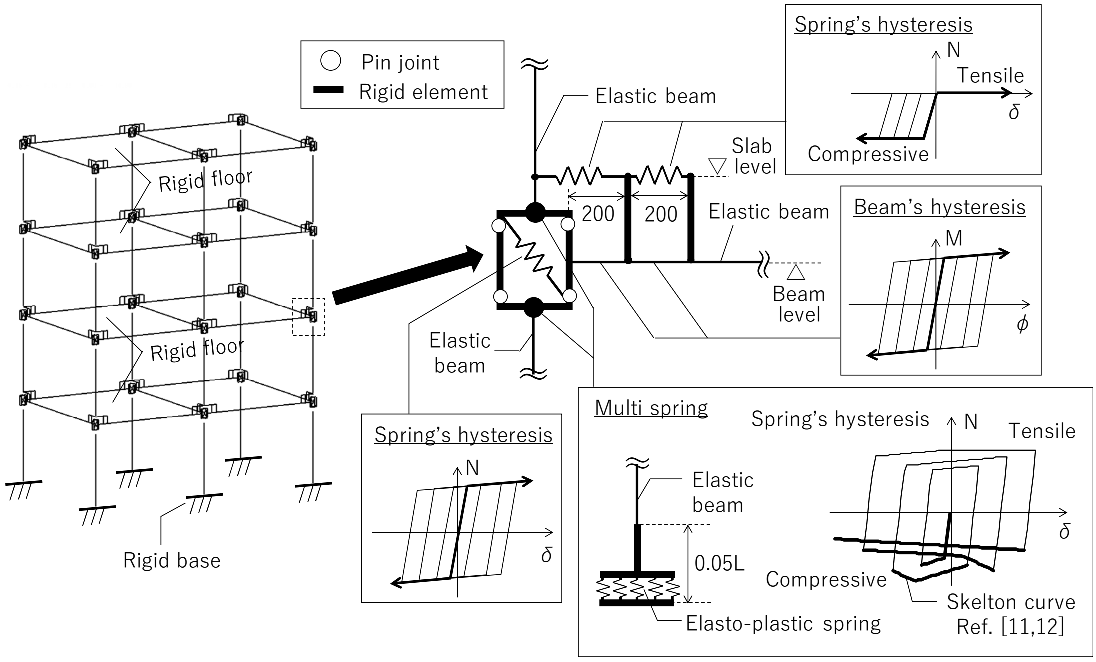


Fig. 2 Model for dynamic analysis

2.2 Verification of analytical model

Model for dynamic analysis shown in 2.1 are verified using the results of previous tests [13],[14], and [10].

2.2.1 Simulation analysis of the loading test of Cold forming square steel tube column [13]

The test specimen is modeled using a column model composed of MS and elastic beam elements in the same way as Model for dynamic analysis shown in 2.1. Axial force and cyclic imposed displacement are applied under the same conditions as in the test for the 0 ° increasing amplitude loading case and 45 ° increasing amplitude loading case. A comparison between analysis and experimental results is shown in Fig. 3. Although the analysis tends to evaluate the strength a little larger, it can be confirmed that the cyclic deterioration behavior due to local buckling can be simulated generally well.

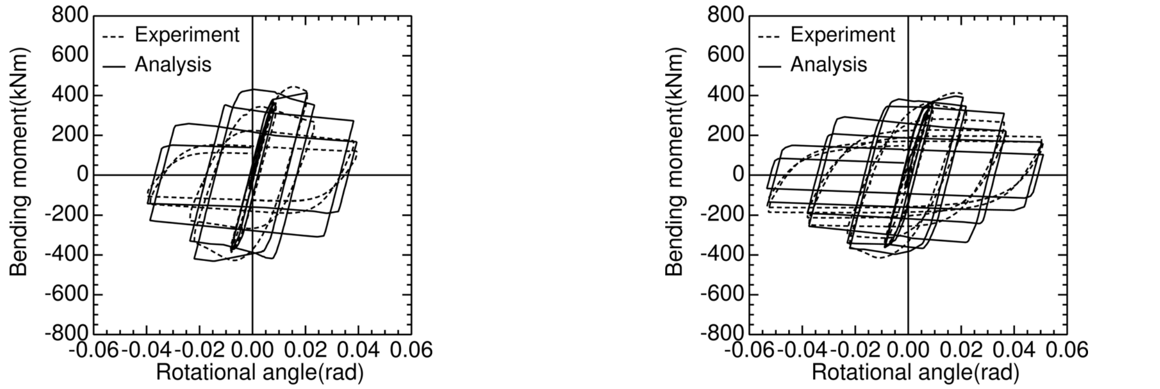
2.2.2 Simulation analysis of loading test of beam to column with concrete slab [14]

The test specimen is modeled in the same way as Model for dynamic analysis shown in 2.1. The loading condition is different from the test, and the panels and beams are separately subjected to imposed deformation. That is, when analyzing the beam, the rotation angle history of the beam observed in the experiment is input with keeping column and panel rigid. Similarly, when analyzing the panel, the panel rotation angle history observed in the experiment is input with keeping beam rigid. A comparison between analysis and experimental results is shown in Fig. 4. Although deterioration of strength due to local buckling of the beam during large deformation could not be simulated, increase of strength due to the composite effect of slabs can be simulated generally well.

2.2.3 Simulation analysis of collapse test of full-scale 4-story steel building at E-Defense [10]

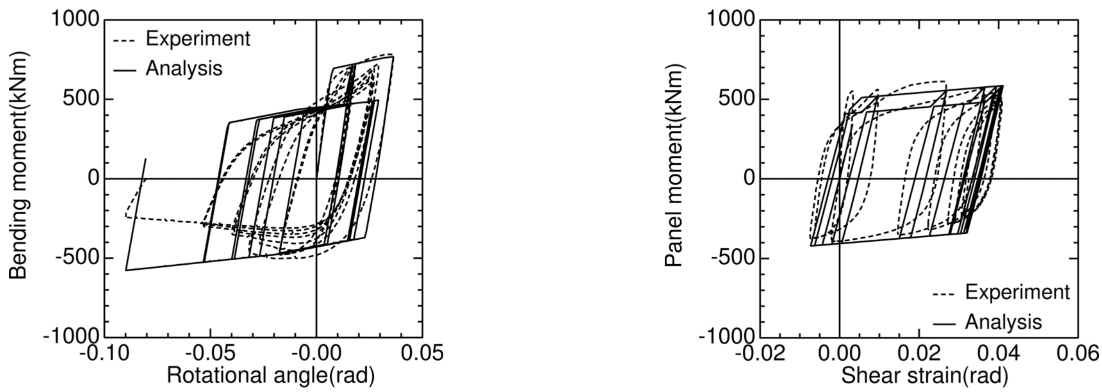
The test specimen is modeled in the same way as the Model for dynamic analysis shown in 2.1. The primary natural period by eigenvalue analysis is 0.87 seconds in the X direction and 0.82 seconds in the Y direction. One reason that the natural period estimated by the 20% excitation experiment (X direction: 0.82 seconds, Y direction: 0.78 seconds) is slightly longer is that the stiffness of non-structural members is not evaluated. The case of 60% and 100% input cases of JR Takatori wave observed in Kobe Earthquake on January 17, 1995 is targeted. The damping is Rayleigh type, and the damping ratio is set to be 2% at the primary mode in the X direction (0.87 second) and the secondary mode in the Y direction (0.27 second).

A comparison between analysis and experimental results is shown in Fig. 5 and Fig. 6. In both cases, the experimental results and the analytical results are generally consistent, confirming the validity of the analytical model.



(a) M-θ of column's bottom (0°direction load) (b) M-θ of column's bottom (45°direction load)

Fig. 3 Comparison of analysis and experiment (Loading test of column)



(a) M-θ of beam (b) M-θ of panel

Fig. 4 Comparison of analysis and experiment (Loading test of composite beam to column connection)

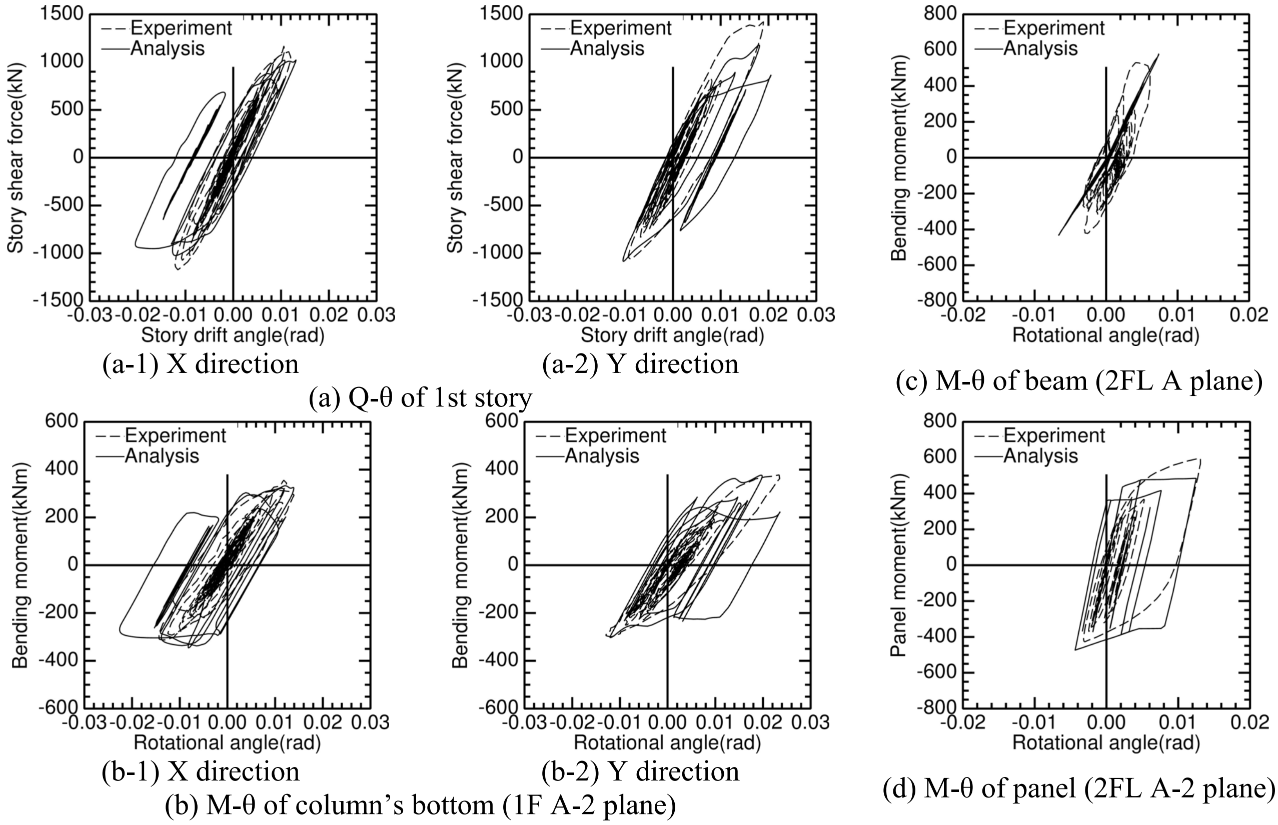


Fig. 5 Comparison of analysis and experiment (60% input cases)

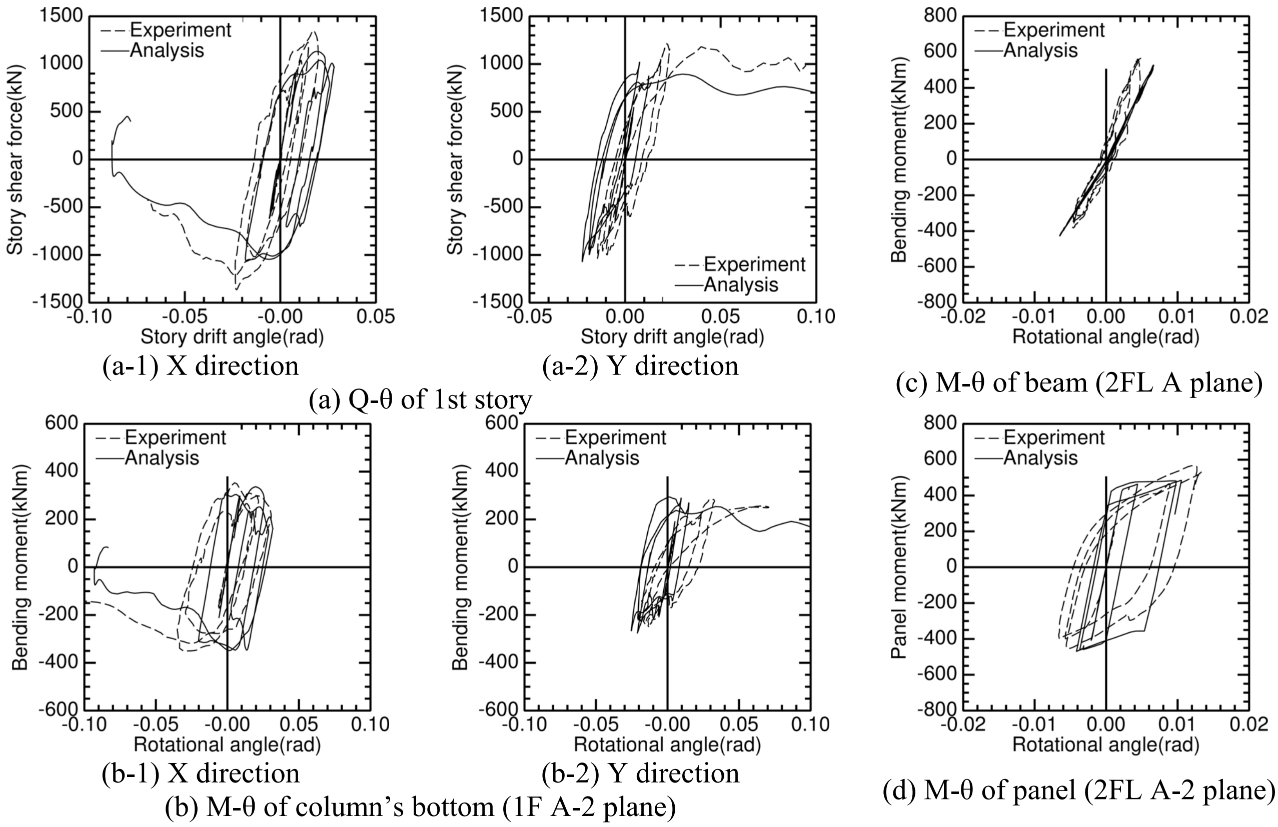


Fig. 6 Comparison of analysis and experiment (100% input cases)

3. Design of target frame

The shape of the target frames is 4-story moment frame with a long side of 5m x 2 spans, a short side of 6m x 1 span, and a floor height of 3.5m. The framing elevation and typical floor plan of the target frame are shown in Fig. 7. The columns are square steel tubes and the beams are H-shaped steel. The width-thickness ratio of the columns is classified as FC, and that of beams is classified as FA or FB (4FL, RFL only) in order to make the column easily deteriorate and give the beam high deformation capacity. The skeleton curve of a single column is shown in Fig. 8. The reference strength of the column is 295N/mm^2 and that of the beam is 235N/mm^2. The yield strength of the analytical model is 1.05 times the reference strength. The 2FL-4FL floor is assumed to be a deck composite slab in which thickness of RC slab is 100mm, and the RFL floor is assumed to be a 150mm thick RC slab. The compressive strength of the slab concrete is 30N/mm^2. The floor load of each story is about 8.0kN/m^2 for 2FL-4FL and about 10.5kN/m^2 for RFL. The load is distributed to each node as node weight.

The design criteria are that the story drift angle to 1/200 or less under the seismic force for elastic design (C_0=0.2) and $Q_u/Q_{un} \geqq 1.0$ in the plastic design. Q_u is defined as the story shear force when the maximum story drift angle reaches 1/50 by push-over analysis of Model for design. Q_{un} is the required horizontal strength of the frame calculated from the following equation.

$$Q_{uni}=Ds_i \times \sum W_i \quad (1)$$

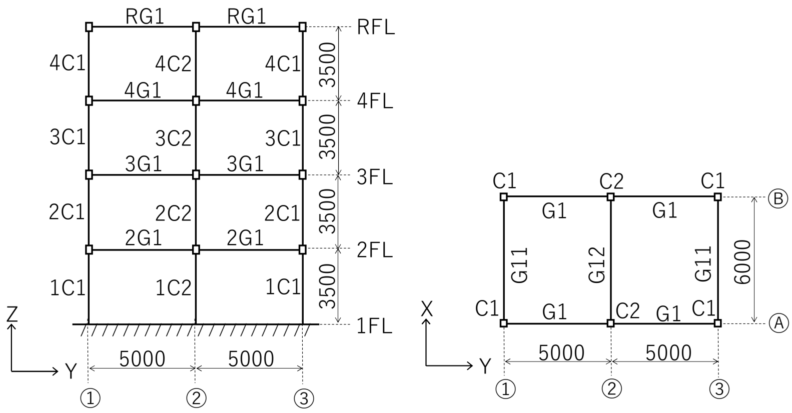
Where Ds is the structural characteristic coefficient (=0.35), $\sum W$ is the weight above the story. The subscript i indicates the story number.

Based on the above conditions, two frame models are designed with different strength ratios of columns, panels and beams. Table 1 shows the frame model data, and Table 2 shows the sections of the main columns and beams. The column strength ratio γ calculated by Eq. (2) is 1.5 and 1.0 in model No.1 and No.2 respectively.

$$\gamma = \Sigma M_{pci} / \Sigma \{\min(1.5 M_{pbi}, 1.3 M_{ppi})\} \quad (2)$$

Where M_{pc} is the sum of the full plastic moments of the upper and lower columns, M_{pb} is the sum of the full plastic moments of the left and right beams, and M_{pp} is the sum of the full moments of the beam-column joint panel. The subscript i indicates the number of beam-column joints.

To satisfy the above design conditions, the depth of columns and beams is adjusted as a parameter. The beam width is determined to be the smaller of 200 mm and half the beam depth, the column thickness is determined as the maximum value classified as FC in 0.5mm units , and the beam thickness is determined to be classified as FA or FB.



(a) Framing elevation (b) Typical floor plan

Fig. 7 Target frame shape

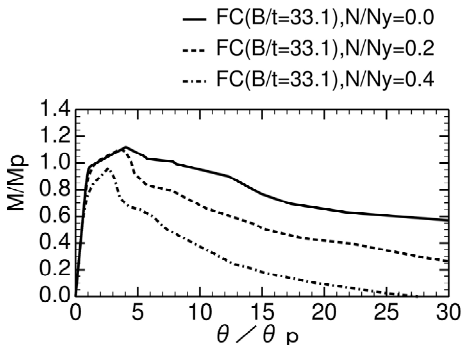


Fig. 8 Skelton curve of column

Table 1 Frame model data

Model No.	Strength ratio of 2FL & 3FL (Column : Panel : Beam)		γ of 2FL & 3FL		Referenced strength[N/mm²]		Q_u/Q_{un}		Primary natural period[s]	
	X	Y	X	Y	Column	Beam	X	Y	X	Y
1	1：0.51：0.44	1：0.51：0.52	1.51	1.50	295	235	1.10	1.14	0.87	0.83
2	1：0.77：0.89	1：0.77：1.15	1.00	1.00			1.21	1.22	0.90	0.87

Table 2 The sections of beams and columns

Model No.	C1	C2	2G1, 3G1	2G11, 3G11	2G12, 3G12
1	□-300x300x9.0 (FC)	□-300x300x9.0 (FC)	H-400x200x6.5x11.5 (FA)	H-400x200x7.5x12.5 (FA)	H-400x200x8.5x13.5 (FA)
2	□-265x265x8.0 (FC)	□-265x265x8.0 (FC)	H-525x200x8.5x11.5 (FA)	H-525x200x8.5x11.5 (FA)	H-525x200x9.5x12.5 (FA)

4. Analytical result

The following shows the time history response analytical results for the designed frames. The input wave is an artificial seismic wave targeting the spectrum defined by Building Standards Law of Japan as the maximum earthquake motions to be considered (Lv.2 earthquake motions). The analytical results of the Y direction structural plane are shown representatively. The maximum story drift angle of each story is shown in Fig. 9, and the relationship between story shear force and story drift angle of the 1st story is shown in Fig. 10. In both figures, the results of push-over analysis ("P.O.") with the Model for design are shown for reference. "P.O. 1st" and "P.O. 2nd" in Fig. 9 are the results of elastic design (C_0=0.2) and plastic design (maximum story drift angle=1/50) respectively. The damage figure in A plane of Model No.1 and 2 are shown in Fig. 11 and Fig. 12 respectively. In the figure, "●" indicates yielding of column and beam, "■" indicates yielding of panel, and "×" indicates deterioration of column. Numerical values in the figure indicate the maximum story drift angles.

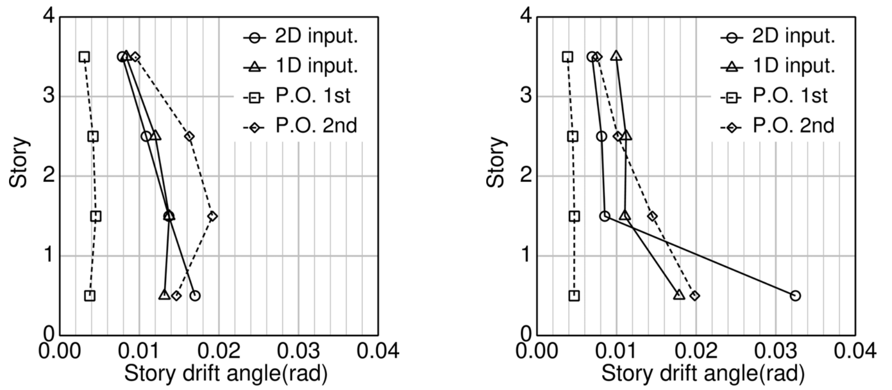
In the result of Model No.1 ($\gamma_{2,\,3FL}$ = 1.5), the maximum story drift angle of 2D input case is increased by about 30% in the 1st story compared to 1D input case as shown in Fig. 9 (a). On the other hand, they are almost similar in the 2nd-4th stories. From the relationship between the story shear force and the story drift angle shown in Fig. 10 (a), it can be confirmed that the effect of 2D input is not significant because the hysteresis of both cases do not greatly differ. From the damage distribution shown in Fig. 11, it can be confirmed that in push-over analysis and 1D input case, the overall sway mechanism in which yielding of the beam or panel is dominant is formed. On the other hand, in 2D input case, it can be confirmed that the weak column mechanism such as column deterioration and plastic hinges at both end of column appears. It is probable that the reason why the maximum story drift angle shown in Fig. 9 is not significantly affected is that seismic energy is not concentrated on the columns due to yielding of beams and panels, and the structure did not completely shift to the weak column mechanism. From the above, although the effect of 2D input is slightly observed, the response result of Model No. 1 is almost as assumed in the design.

On the other hand, in the result of Model No. 2 ($\gamma_{2,\,3FL}$ = 1.0), the effect of 2D input is remarkably observed. As shown in Fig. 9 (b), the maximum story drift angle of the 1st story is significantly increased in 2D input case compared to 1D input case, and the maximum story drift angle in the 2nd-4th stories is decreased accordingly. As shown in the damage figure shown in Fig. 12, yielding of the beam and panel is observed in push-over analysis, but yielding of the beam is hardly observed in 1D input case, and the weak column mechanism is formed. In 2D input case, yielding of the beam or panel appears at all, resulting in a serious damage in which all the column bases at the 1st story deteriorate.

Here, the reason why the collapse mechanism is different between push-over analysis, 1D input case and 2D input case is regarded below. Fig. 13 shows the M-θ relationship of the 2FL A1-2 beam and the 2FL A2 panel. In the results of static analysis (P.O.) using Model for design that does not consider the composite effect of slabs, beams and panels are yielded greatly. On the other hand, in the dynamic analytical case, they are not yielded as much as static analysis because the composite effects of slabs are considered with Model for dynamic analysis. It is probable that the strength of the columns is relatively decreased due to the increase in the strength of the beams and panels, and a weak column mechanism is formed even in 1D input case where there is no effect of 2D input.

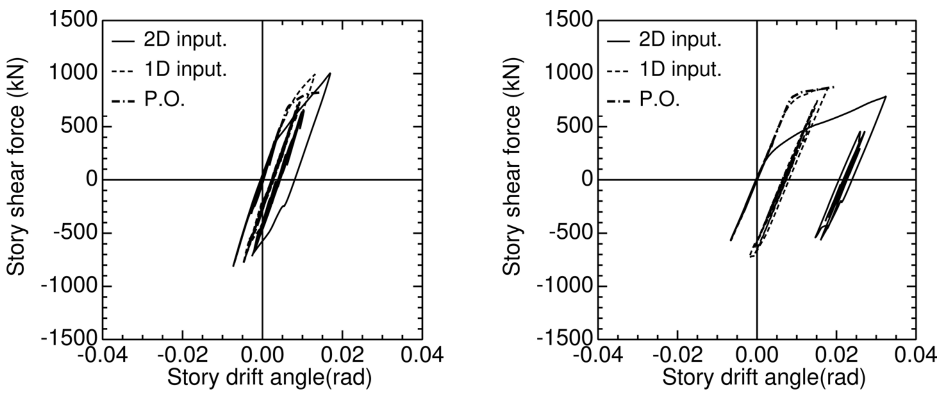
Fig. 14 (a) shows the bending moment history of the 1st story A3 column base in 2D input case, and Fig. 14 (b) shows the time history waveform of the yield axial force ratio. In Fig. 14 (a), the Mx-My interaction curve calculated using the yield axial force ratio under dead load (0.11) and the maximum yield axial force ratio observed in dynamic analysis (0.37) is superimposed. The bold line in both figures shows the half cycle (13.5s to 13.9s) at which the compression variable axial force is maximum. In this half cycle, the bending moment in the 45 ° direction becomes dominant, and the additional axial force due to the overturning moment is maximized. As a result, it is considered that the full plastic moment is greatly decreased, and the column is seriously damaged.

As described above, it can be confirmed that, even if the strength assumed at the time of design is almost the same, the response under 2D input greatly differs in the frames having different strength ratios γ of the columns. In addition, it can be confirmed that even under 1D input, the columns can be damaged more than the design assumption due to the effect of the slab composite effect.



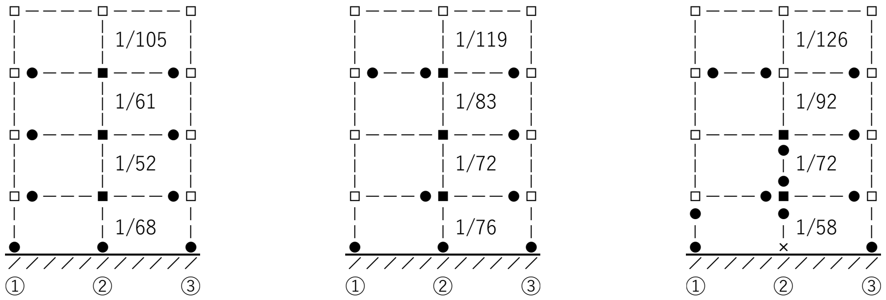
(a) Model No.1 (b) Model No.2

Fig. 9 Maximum story drift angle



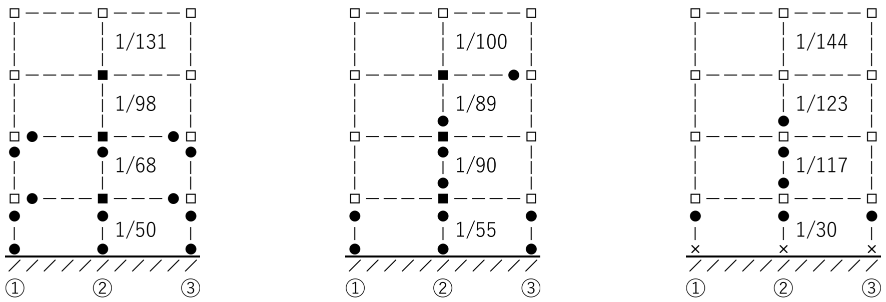
(a) Model No.1 (b) Model No.2

Fig. 10 Q-θ of 1st story



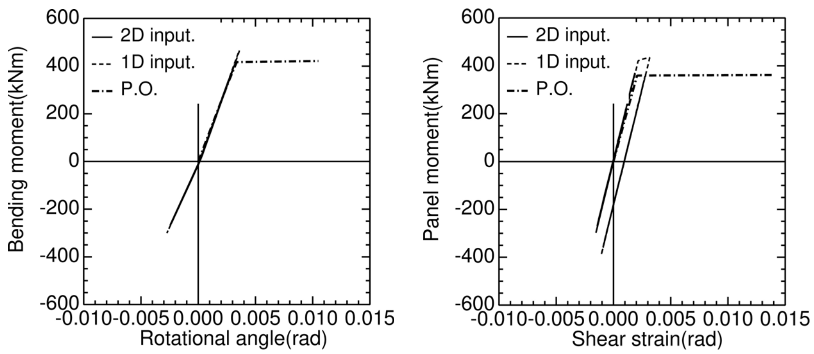
(a-1) Push-over analysis with model for design
(a-2) Dynamic analysis in 1D input case
(a-3) Dynamic analysis in 2D input case

Fig. 11 Damage distribution in A plane of Model No.1



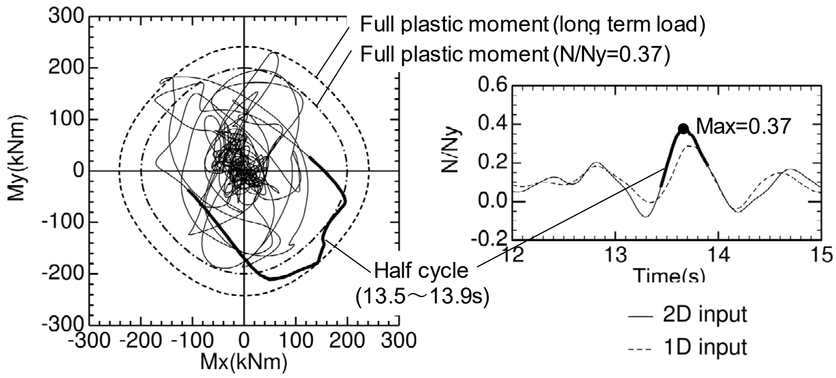
(a-1) Push-over analysis with model for design
(a-2) Dynamic analysis in 1D input case
(a-3) Dynamic analysis in 2D input case

Fig. 12 Damage distribution in A plane of Model No.2



(a)Beam (b)Panel

Fig. 13 Beam and panel M-θ of Model No.2



(a) Moment (b) Axial force ratio

Fig. 14 Column moment and axial force ratio of Model No.2

5. Conclusion

In this paper, seismic response analyses under the horizontal bi-directional input were conducted on moment frames, and the effects of the horizontal bi-directional input on the collapse mechanism and the maximum response story drift were shown. The analytical model was a three-dimensional frame model that can evaluate the effect of biaxial bending of columns, the strength deterioration of columns due to local buckling, and the increase in beam and panel strength due to the composite effect of slabs.

From the analytical results, the following knowledge was obtained.

1) The maximum story drift angle of 1st story depends on the column strength ratio γ, and can be more than 1/40rad in the story with γ of about 1.0 under horizontal bi-directional excitation. It is considered that the smaller γ of the story, the more bi-directional excitation affects, and the story with sufficiently large γ is hardly affected because yielding of the beam and panel precedes.

2) Even in a frame designed to form overall sway mechanism, there is a high possibility that the story collapse will proceed beyond the design assumption due to horizontal bi-directional excitation and composite effect by concrete slab. In a story with γ of about 1.0, all the ends of the columns can yield and all the both ends of the columns can deteriorate.

6. References

[1] Editorial Committee of the 2018 Design and Execution Manual of Cold Forming Square Steel Tube : The 2018 Design and Execution Manual of Cold Forming Square Steel Tube, 2018.2 (in Japanese)

[2] Mukaide, S., Motoki, K., Kitagawa, T. and Tada, M. : Numerical Analysis of Collapsing Behavior for Multi-Story Steel Moment Frames Considering Strength Degradation by Local Buckling, Journal of Structures and Constructional Engineering (Transactions of AIJ) , Vol.78, No.685, pp.579-588, 2013.3 (in Japanese)

[3] Nishino, Y., Mukaide, S., Ori, K. and Tada, M. : Collapse Analysis of Steel Moment Frames with Composite Beams -Part1 Analysis Model-, Summary of Technical Papers of Annual Meeting Architectural Institute of Japan, Structure III, pp.651-652, 2016.8

[4] Chan, I., Koetaka, Y. and Suita, K. : Effects of Input Direction of Ground Motion and Column Overdesign Factor on Seismic Response of 3D Steel Moment Frames with Square Tube Columns, Journal of Structures and Constructional Engineering (Transactions of AIJ) , Vol.80, No.717, pp.1773-1783, 2015.11 (in Japanese)

[5] Kimura, R. and Ogawa, K. : Effect of Column-to-Beam Strength Ratio on Maximum Story Drift Angle Response of Steel Frames under Horizontal Bi-directional Ground Motions, Steel Construction Engineering, Vol.14, No.55, pp.111-121, 2007.9 (in Japanese)

[6] Sakai, Y. and Ogawa, K. : Pertinent Column-to-Beam Strength Ratio of Steel Frames Subjected to Horizontal Bidirectional Ground Motions, Steel Construction Engineering, Vol.17, No.67, pp.53-64, 2010.9 (in Japanese)

[7] Koetaka, Y. and Kanada, M. : Effects of Input Earthquake Direction and Hysteresis Characteristics on Seismic Response of Steel Structures, Steel Construction Engineering, Vol.18, No.69, pp.17-32, 2011.3 (in Japanese)

[8] Yamada, S. and Shimada, Y. : Collapse Behavior and Ultimate Earthquake Resistance of Weak Column Type Multi Story Steel Frame under Bi-Axial Ground Motion, Journal of Structures and Constructional Engineering (Transactions of AIJ) , Vol.76, No.662, pp.837-844, 2011.4 (in Japanese)

[9] Horimoto, A., Tada, M., Tamai, H., Ohgami, K., Kuwahara, S. and Mitani, A. : Three Dimensional Analytical Simulation of Collapse by Collaborative Structural Analysis for Full-Scale 4-Story Steel Building, Proceeding of Architectural Research Meetings at Kinki Chapter, AIJ, pp.277-283,2009,3 (in Japanese)

[10] Suita, K., Matsuoka, Y., Yamada, S., Shimada, Y., Tada, M., and Kasai, K. : Experimental Procedure and Elastic Response Characteristics of Shaking Table Test -Complete Collapse Test if Full-Scale 4-story Steel Building Part 1-, Journal of Structures and Constructional Engineering (Transactions of AIJ) , Vol.74, No.635, pp.157-166, 2009.1 (in Japanese)

[11] Ishida, T., Yamada, S. and Shimada, Y. : Analytical Model of RHS Columns Under Random Bi-Directional Horizontal Forces, Journal of Structures and Constructional Engineering (Transactions of AIJ) , Vol.78, No.691, pp.1631-1640, 2013.9 (in Japanese)

[12] Yamada, S., Ishida, T. and Shimada, Y. : Hysteresis Model of RHS Columns in the Deteriorating Range Governed by Local Buckling, Journal of Structures and Constructional Engineering (Transactions of AIJ) , Vol.77, No.674, pp.627-636, 2012.4 (in Japanese)

[13] Yamada, S. and Shimada, Y. : Cyclic Loading Test of RHS-Columns under Bi-Axial Bending, the 13th Japan Earthquake Engineering Symposium, pp.2868-2873, 2010.11 (in Japanese)

[14] Yamada, S., Satsukawa, K., Kishiki, S., Shimada, Y., Matsuoka, Y. and Suita, K. : Elastic-Plastic Behavior of Panel Zone in Beam to External Column Connection with Concrete Slab, Journal of Structures and Constructional Engineering (Transactions of AIJ) , Vol.74, No.644, pp.1841-1849, 2009.10 (in Japanese)

