DESIGN PARAMETER OF SEMI-RIGID CONNECTIONS THROUGH STATIC STEEL FRAME ANALYSIS

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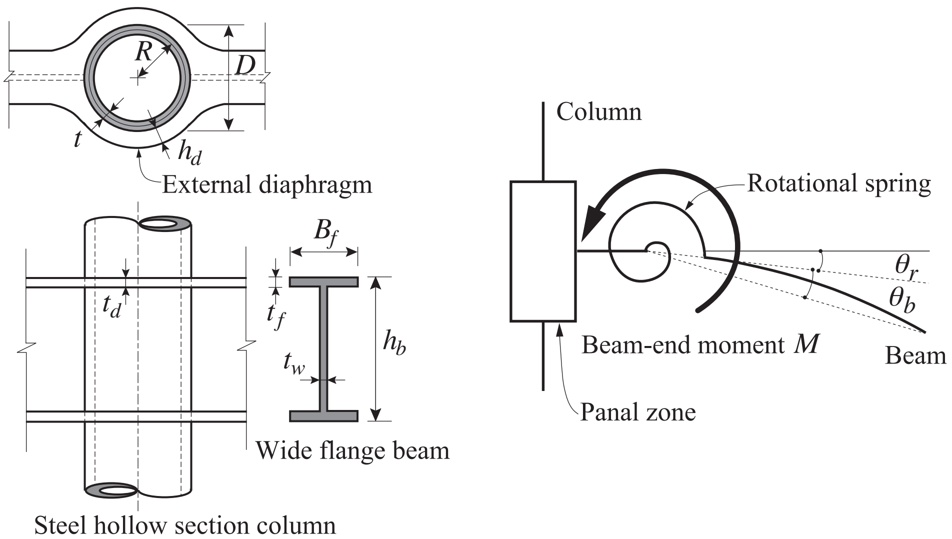
ABSTRACT:

In this paper, in order to obtain a rational design parameter involving the connection factor that can be used in Japan, a fundamental study about the characteristic of multi-storied steel frames with changes in the strength of the beam-to-column connections was performed via a static analysis. The beam-to-column connection is an external diaphragm. The strength of the connection was varied with the height of the external diaphragm. In order to formulate a design condition involving the connections, four parameters were defined. The main parameter is the connection factor, adopted when the small one changed to a large one, and a push-over static analysis with an earthquake load was conducted. From the analysis results, information about the availability of the use of semi-rigid frames in Japan was obtained. The optimum value of the parameter, when both the connections and beams yield, can also be obtained.

KEYWORDS: multi-storied steel frame, beam-to-column connection, semi-rigid connection, external diaphragm, static analysis

1. INTRODUCTION

Nowadays, semi-rigid connection frames are used in Europe and US. In Japan, considering the effect of earthquakes, a rigid connection design of the beam-to-column connections on the steel frame is required. Usually, when a hollow section column is connected to wide-flange beams, a through connection is required. This kind of connection must cut the columns at the location where it connects with the beams. Two slices of steel plates were welded to the columns; finally, the columns were welded together again. They exhibited sufficient stiffness; this is considered to be a rigid connection. It can be seen that the weld work has been done, and the column is cut at the place having the largest moment. When the Kobe earthquake occurred, the welds on these columns broke. This damage is considered to be very dangerous since a soft-story failure mode can develop easily. Therefore, this method is inefficient and unsafe.



(a) External diaphragm model (b) Rotational spring model

Figure 1 Beam-to-column connection models

In this study, a kind of connection with an external diaphragm is introduced. The sketch of the external diaphragm is shown in Figure 1 (b). The advantage of this type of connection is that without cutting column at the top and bottom of the floor where maximum moment is happened. For the local deformation at the column near to the beam flange, an external diaphragm connection should be considered as a semi-rigid connection.

The stiffness of the overall frame is related to the stiffness of the connection. When the frame uses a semi-rigid connection, the total stiffness decreases with a decrease in the stiffness of the semi-rigid connection. The preliminary design under a seismic load in the design code of Japan demands that the maximum inter-story drift angle cannot exceed a special value of 1/200 radians. Consequently, the maximum inter-story drift angle did not lead to the damage of any nonstructural component: the maximum limit can be selected as 1/120.

2. DESIGN PARAMETER OF BEAM-TO-COLUMN CONNECTIONS

During the analysis of the frame having a semi-rigid connection using the frame analysis program *club.f*, the external diaphragm is modeled as a rotational spring with proper stress and stiffness, as shown in Figure 1 (b). In this paper, four parameters were defined, which indicate the relationship between the stiffness and strength of the beam-to-column connection with the beam connected to it. The stress and stiffness of the beam-to-column connection can be calculated from the presumption expressions, which are deduced from the analysis result when employing subassemblage steel frames using the finite element analysis (FEA) program. The stiffness and strength of the connections obtained from the presumption expressions are related to the size of the columns and beams connected to them.

2.1. Connection Factor

In the aseismic design code of Japan for steel frames, there is a requirement for the connection factor α. In this paper, the α is expressed by parameter r. It is the ratio of the final moment capacity of the beam-to-column connection to the full plastic moment of the beam connected to it. When the level of steel material is 400 N, this parameter must be larger than 1.3. In this paper, the connection factor r is used to obtain the design of the maximum strength of the connection; r can be expressed as

$$r = M_{r_{\max}} / M_{Pb} \tag{2.1}$$

where $M_{r_{\max}}$ is the ultimate moment capacity of the beam-to-column connection and M_{Pb} is the full plastic moment of the beam connected to it.

2.2. Yield Stiffness Ratio of Beam-to-Column Connection

The moment rotational relationship of the beam-to-column connection can be expressed by a smooth curve; the yield stress of the beam-to-column connection is considered to be the moment when the tangential stiffness of the moment rotational curve is one-third the initial stiffness of the connection. Eqn. 2.2 describes the design of the yield stiffness of the connection.

$$r_y = M_{r_y} / M_{Pb} \tag{2.2}$$

Here, M_{r_y} is the yield stiffness of the connection and M_{Pb} is the full plastic moment of the beam connected to it.

2.3. Initial Stiffness Ratio of Beam-to-Column Connection

Initial stiffness of the connection influences the total stiffness of the frame; therefore, the maximum inter-story drift angle is related to the stiffness of the connection. To have an evaluation of the initial stiffness of the beam-to-column connection, a parameter k_0 is defined. It can be expressed using Eqn. 2.3.

$$k_0 = K_{r0} / K_b \tag{2.3}$$

Here, K_{r_0} is the initial rotational stiffness of the beam-to-column connection. K_b is the rotational stiffness of the beam connected to it. It can be expressed as $K_b = 6EI/L$. Here, E denotes the Young's modulus of steel. I is the moment of inertia of the beam. L is the actual length of the beam.

2.4. Hardening Factor of Beam-to-Column Connection

After the beam-to-column connection yields, it is expected that the connection have enough secondary stiffness so that the beams can reach their yield stress without the fracture at the connection. The parameter e_t denotes the hardening modulus of the beam-to-column connection.

$$e_t = K_{r_2} / K_{r_0} \tag{2.4}$$

Here, K_{r_2} denotes the secondary stiffness of the connection. K_{r_0} denotes the initial stiffness of the connection.

3. ANALYSIS PROGRAM

3.1. Examination of the Frame Analysis Program club.f

In this paper, the external diaphragms are modeled as rotational springs. They are situated at the end of the beams and have a finite stiffness and strength. It reveals the characteristics of the external diaphragm. The analysis of a two-story two-bay analysis is performed using the *club.f* program. This program simplifies the analysis, because actual frames can be modeled as wire frames with rotational springs and joint panels. Earlier, a similar analysis was performed using the FEA program with *Marc*. Figure 2 shows a comparison of the analytical results obtained when using the *club.f* program and *Marc*. It shows that the *club.f* finely traces the *Marc* results. It is possible to use such a procedure to investigate the characteristics of the steel frame with changes in the characteristics of the beam-to-column connection with an external diaphragm.

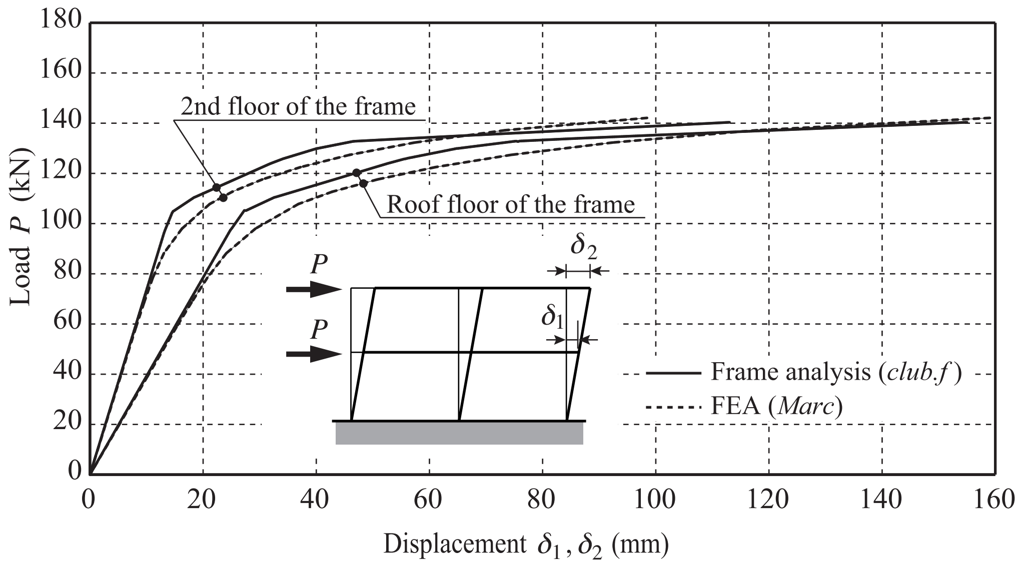


Figure 2 Comparison of the analysis programs

3.2. Earthquake Loads

This study considers a steel frame with an external diaphragm modeled as a rotational spring. The connection factor r is changed from 0.7 to 1.6 in steps of 0.1. The stiffness is also altered when changing the connection factor. In the stiffness design of the steel structures, the designer seeks to make the structure sufficiently stiff so that deformations under the most adverse working conditions will not hamper the strength or serviceability of the steel structure. An examination of the deflections of the frame when subject to a lateral seismic load must be undertaken in the preliminary design, adhering to the design code of Japan .

3.3. Pushover Static Analysis

A monotonic static analysis of the frame was done with a lateral force P_i as the pushover force. This force is used according to the aseismic design code of Japan. The value of r is changed from 0.9 to 1.6; the analysis result of the frame with a rigid connection is also included. It can be seen that with a decrease in r, the maximum inter-story drift angle increases.

4. ANALYSIS FRAMES

This study is performed using actually designed 12 frames. The frames have wide-flange beams and circular hollow section columns. Figure 3 shows the frame shape and section sizes (Table 1).

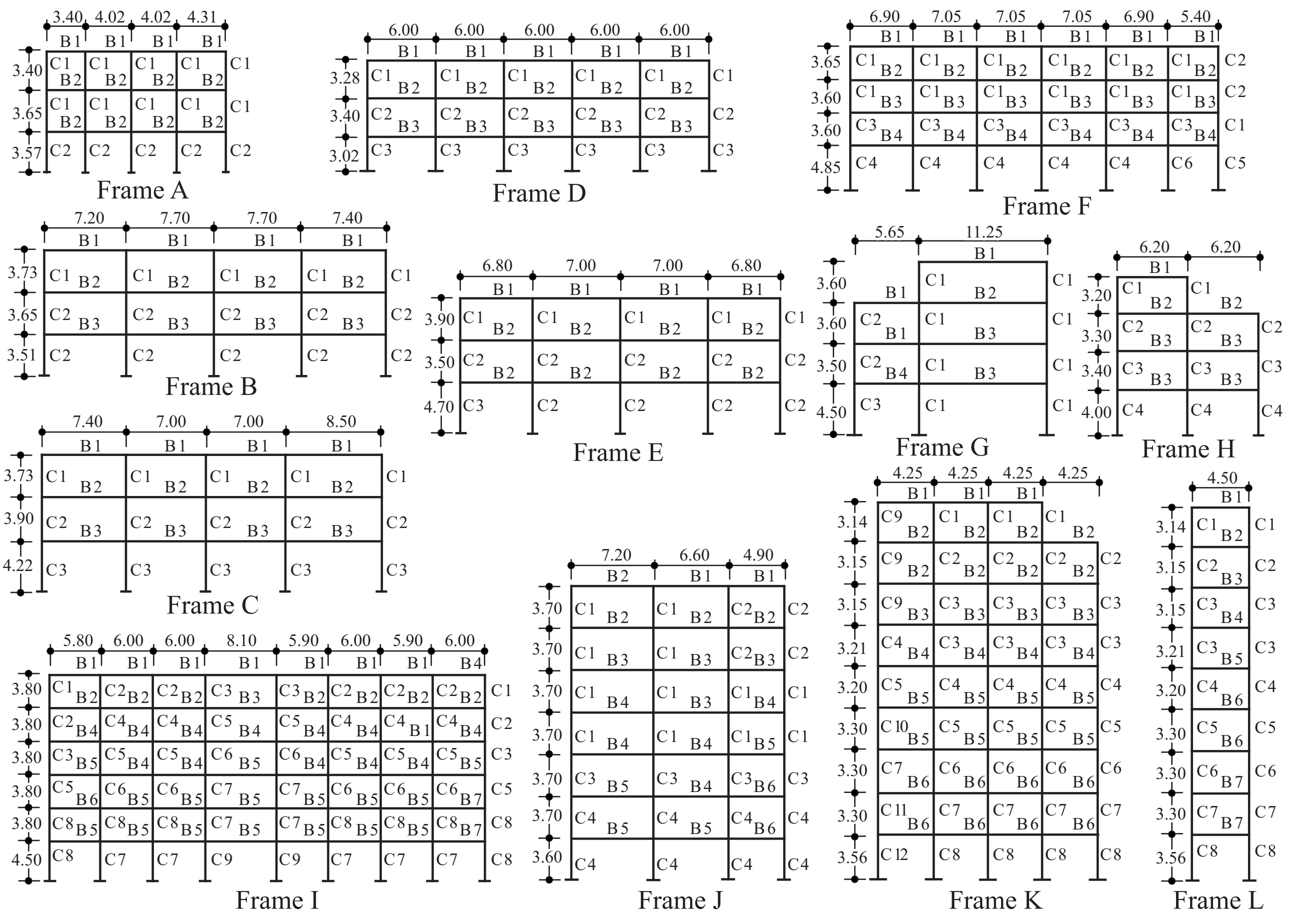


Figure 3 Shape of the analysis frames

5. ANALYTICAL RESULT

5.1. Maximum Inter-story Drift Angle

In Japan, in order to ensure the serviceability of the structure, the limit value of the maximum inter-story drift angle of the frame cannot exceed 1/200. By performing the analysis result of the entire frame with different connection factors, it can be shown that most of the results of a semi-rigid connection exceed 1/200. However, when it is required that the flexibility of the structures does not lead to the damage of any nonstructural component, the designer can choose the limit deformation value to 1/120, then most of the results of a semi-rigid connection according to the design code in Japan, as shown in Figure 4. Therefore, it is possible to use semi-rigid connections in steel frames in Japan if the stiffness is not excessively small.

Table 1 Columns and beams section

	Column	Section (mm)	Beam	Section (mm)
A	C1	318.5 φ x 7.9	B1	H- 248 x 124 x 5 x 8
	C2	318.5 φ x 10.3	B2	H- 298 x 149 x 5.5 x 8
B	C1	267.4 φ x 7	B1	H- 248 x 124 x 5 x 8
	C2	355.6 φ x 12.7	B2	H- 446 x 199 x 8 x 12
			B3	H- 496 x 199 x 9 x 14
C	C1	508 φ x 7.9	B1	H- 244 x 175 x 7 x 11
	C2	609.6 φ x 9.5	B2	H- 496 x 199 x 9 x 14
	C3	609.6 φ x 12	B3	H- 596 x 199 x 10 x 15
D	C1	355.6 φ x 12	B1	H- 446 x 199 x 8 x 12
	C2	400 φ x 12	B2	H- 496 x 199 x 9 x 14
	C3	406.4 φ x 12.7	B3	H- 500 x 200 x 10 x 16
E	C1	355.6 φ x 9.5	B1	H- 300 x 150 x 6.5 x 9
	C2	457.2 φ x 12.7	B2	H- 500 x 200 x 10 x 16
	C3	508 φ x 12		
F	C1	400 φ x 19	B1	H- 294 x 302 x 12 x 12
	C2	318.5 φ x 19	B2	H- 340 x 250 x 9 x 14
	C3	457.2 φ x 19	B3	H- 500 x 200 x 10 x 16
	C4	508 φ x 16	B4	H- 482 x 300 x 11 x 15
	C5	406.4 φ x 21.4		
	C6	558.8 x 16		
G	C1	406.4 φ x 21.4	B1	H- 500 x 250 x 9 x 19
	C2	298.5 φ x 22	B2	H- 588 x 300 x 12 x 20
	C3	355.6 φ x 20	B3	H- 582 x 300 x 12 x 17
			B4	H- 482 x 300 x 11 x 15
H	C1	216.3 φ x 8	B1	H- 248 x 124 x 5 x 8
	C2	216.3 φ x 12	B2	H- 298 x 149 x 5.5 x 8
	C3	267.4 φ x 12.7	B3	H- 346 x 174 x 6 x 9
	C4	318.5 φ x 10.3		
I	C1	406.4 φ x 12.7	B1	H- 350 x 250 x 9 x 14
	C2	457.2 φ x 14	B2	H- 500 x 200 x 10 x 16
	C3	457.2 φ x 19	B3	H- 482 x 300 x 11 x 15
	C4	457.2 φ x 16	B4	H- 488 x 300 x 11 x 18

	Column	Section (mm)	Beam	Section (mm)
I	C5	508 φ x 16	B5	H- 582 x 300 x 12 x 17
	C6	508 φ x 19	B6	H- 588 x 300 x 12 x 20
	C7	508 φ x 16	B7	H- 692 x 300 x 13 x 20
	C8	508 φ x 19		
	C9	609.6 φ x 19		
J	C1	457.2 φ x 12.7	B1	H- 298 x 201 x 9 x 14
	C2	406.4 φ x 12	B2	H- 400 x 200 x 8 x 13
	C3	457.2 φ x 19	B3	H- 450 x 250 x 9 x 14
	C4	457.2 φ x 22	B4	H- 434 x 299 x 10 x 15
			B5	H- 488 x 300 x 11 x 18
			B6	H- 588 x 300 x 12 x 20
K	C1	355.6 φ x 12.7	B1	H- 200 x 204 x 12 x 12
	C2	406.4 φ x 12.7	B2	H- 298 x 201 x 9 x 14
	C3	457.2 φ x 12.7	B3	H- 336 x 249 x 8 x 12
	C4	558.8 φ x 12.7	B4	H- 340 x 250 x 9 x 14
	C5	550 φ x 16	B5	H- 386 x 299 x 9 x 14
	C6	558.8 φ x 16	B6	H- 344 x 348 x 10 x 16
	C7	600 φ x 16		
	C8	660.4 φ x 19		
	C9	500 φ x 12		
	C10	558.8 φ x 16		
	C11	600 φ x 19		
	C12	700 φ x 19		
L	C1	400 φ x 12	B1	H- 396 x 199 x 7 x 11
	C2	400 φ x 16	B2	H- 446 x 199 x 8 x 12
	C3	400 φ x 19	B3	H- 440 x 300 x 11 x 18
	C3	400 φ x 19	B4	H- 496 x 199 x 9 x 14
	C4	500 φ x 19	B5	H- 482 x 300 x 11 x 15
	C5	500 φ x 22	B6	H- 596 x 199 x 10 x 15
	C6	550 φ x 19	B6	H- 596 x 199 x 10 x 15
	C7	600 φ x 19	B7	H- 582 x 300 x 12 x 17
	C8	700 φ x 19	B7	H- 582 x 300 x 12 x 17

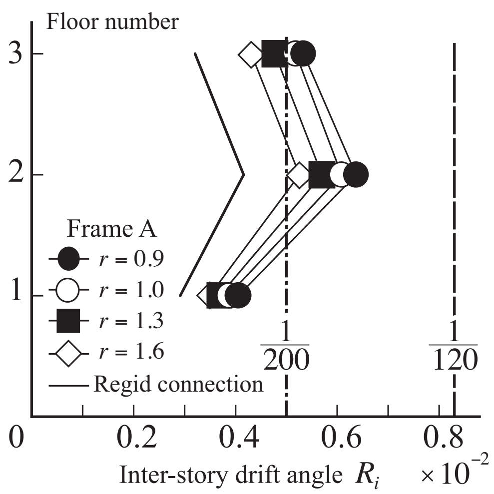


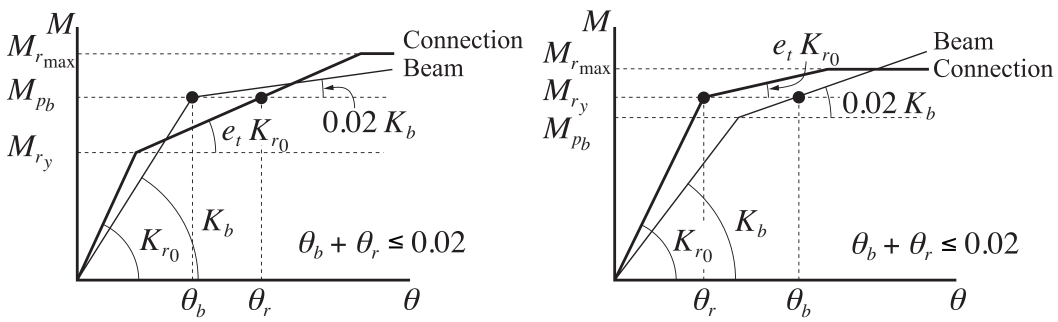
Figure 4 Inter-story drift angle

5.2. Appropriate Design Strategy for Connection Factor

Survival in large earthquakes depends directly on the ability of their framing system to dissipate energy hysteretically while undergoing large inelastic deformations. In a rigid connection frame, the main dissipation of the seismic energy in a beam part is only by the beams. While in a semi-rigid frame, it is composed of beams and beam-to-column connections. In other words, the dissipation of the seismic energy of the beam part is not only in the beams but also in the connections. The yielding of the beams and beam-to-column connections is expected when large inelastic deformations occur. An appropriate design strategy for the connection factor is important.

A monotonic pushover analysis is done using *club.f*. The analysis is done until the drift at the top of the frame is 1/50 of the total height of the frame. This value is considered to be a large inelastic deformation of the frame since it is the maximum value of the inter-story drift angle for many middle-level earthquake records in the dynamic analysis results.

Before the frames undergo a large deformation, the yielding condition for the frame for the beams as well as the beam-to-column connection can be categorized into two cases.



(a) Case that connection yield precedes (b) Case that beam yield precedes

Figure 5 Yield pattern order between connections and beams

5.2.1 Connection yield precedes

The yielding of the beam-to-column connection develops first. After the yielding, the connection maintains the secondary stiffness till the stress reaches the yield moment of the beams. The rotational angle of the beams plus one of the beam-to-column connections cannot exceed 0.02, as shown Figure 5 (a). A proper result of the yield ratio can be expressed using Eqn. 5.1.

$$r_y \ge \frac{e_t}{1-e_t}\left(\frac{1}{e_t} + k_0 - 0.02k_0\frac{K_b}{M_{Pb}}\right) \tag{5.1}$$

This equation expresses the areas and the surface represents the lower boundary. The surface changes with the parameters k_0, e_t, K_b, and M_{Pb}. This is shown in Frame A in Figure 6 (a).

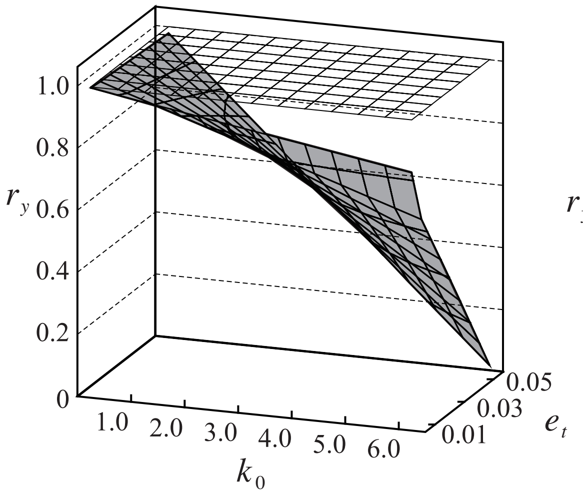
5.2.2 Beam yield precedes

The yield of the beam develops first. After the yielding, the beam maintains the hardening modulus to be 0.02 until the stress reaches the yield stress of the beam-to-column connection, as shown in Figure 5 (b). Similar to that in case 1, the rotational angle of the beam plus the beam-to-column connection cannot exceed 0.02. From Figure 5 (b), a proper result for the yield ratio can be expressed by Eqn. 5.2.

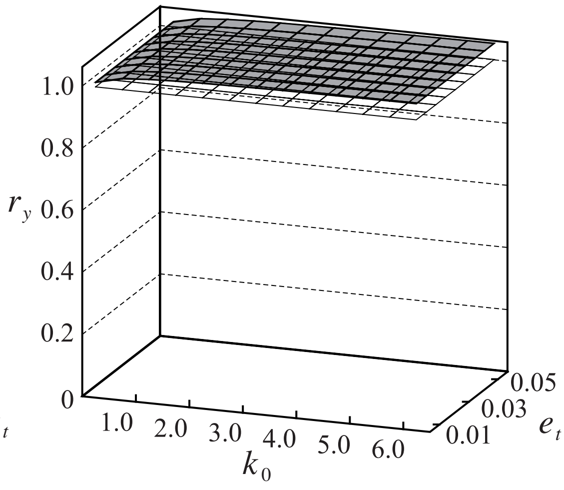
$$r_y \le \frac{0.02\dfrac{K_b}{M_{Pb}}k_0 + \dfrac{1-\mu}{\mu}k_0}{\dfrac{k_0}{\mu}+1} \tag{5.2}$$

This equation expresses the areas and the surface represents the upper boundary. The surface changes with the parameters k_0, K_b, and M_{Pb}. This is shown in Frame A in Figure 6 (b).

All the yield ratios in all the analysis frames were investigated using Eqn. 5.1 and Eqn. 5.2 when r = 1.3. This



(a) The case that connection yield precedes



(b) The case that beam yield precedes

Figure 6 Area in which both connection and beam yield together (Frame A)

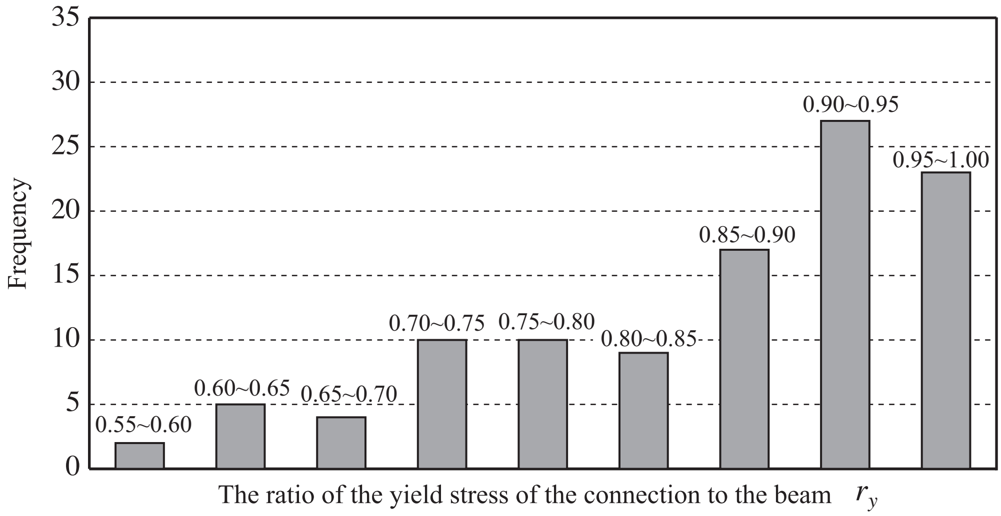


Figure 7 Frequency distribution of yield-limited ratio for the designed connection

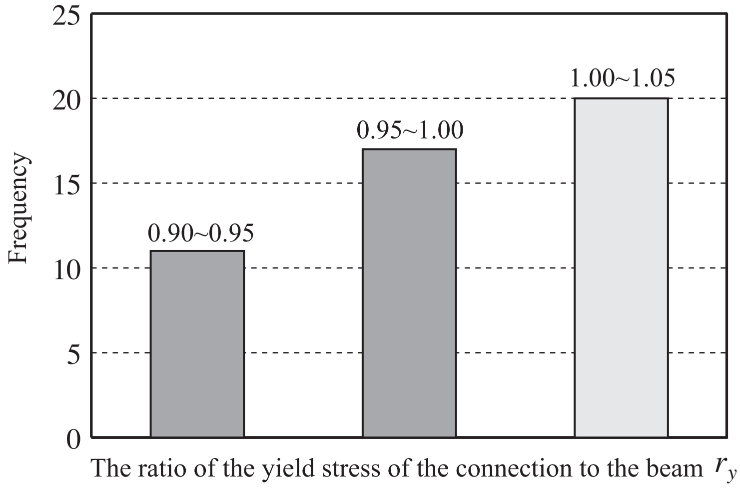


Figure 8 Frequency distribution of a reasonable yield ratio

satisfies the establishment of the possession strength design in Japan. The distribution of the lower boundary of the yield ratio r_y is shown in Figure 7 on the assumption that the sum of the deformation of the beam and beam-to-column connection does not exceed 1/50. It can be shown that most of this result is in the range from 0.85 to 1.00. Moreover, it can be shown that this graph includes prohibitively small results. This is because when the secondary stiffness is sufficiently large, it is possible that the frame reaches the yield stress of the beam even with a small yield ratio. The mean value of r_y is approximately 0.85.

When $r = 1.3$, the yield ratio r_y can be calculated from the presumption expression, and the truth value of r_y must include the areas shown in Figure 6 (a) and (b). The frequency distribution of this truth value is shown in Figure 8. It includes the results from case 1 and case 2, and the mean value of r_y for case 1 is 0.96.

6. CONCLUSIONS

The static analysis of the multi-storied steel frame with a semi-rigid connection was carried out. The strength of the semi-rigid connection is expressed by a parameter r: r is changed from 0.7 to 1.6 in steps of 0.1. From the static analysis result of all the frames, the following conclusions were drawn:

1. When the limit value of the maximum inter-story drift angle of the frame is assumed to be under 1/200 with a seismic load, the deformation in most of the frames cannot satisfy this criterion even when the frame with a rigid connection can satisfy it. However, a relief limit value of 1/120 can be used in the frame, which does not lead to damage to any nonstructural component. Most of the analysis results of the deformation can satisfy this criterion when r is not too small. Therefore, the use of a semi-rigid connection is possible in Japan if the stiffness is not excessively small.
2. An appropriate design parameter is obtained when not only the beams but also the beam-to-column connections yield when the frames have a large deformation. If the yield of the beam-to-column connection occurs first, the lower boundary of the yield ratio r_y can be achieved. The mean value of the yield ratio of the entire analysis frame is 0.85 when $r = 1.3$.
3. When $r = 1.3$, the yield ratio r_y can be obtained by using the presumption expression of the connection yield strength yielded by the FEA result; it is assumed. The mean value of the yield ratio r_y is 0.96.

ACKNOWLEDGMENT

This research was supported by a Grant-in-Aid for engineering research from the study group for steel structures in the Japanese Society of Construction and graduate school action scheme for internationalization of university students of Kumamoto University.

REFERENCES

Chen W.F. and Kishi N.N. (1989). Semi-rigid steel beam-to-column connections database and modeling. *J. Struct. Eng., ASCE* **120:6**
European Committee for Standardization (CEN): Eurocode3: Design of steel structures Part 1.1 General rules and rules for buildings, 1993
Ogawa K. Combined non-linear utility for building frames, *club.f*, version K-3.3
Sui W. and Yamanari M. (2007). Evaluation of the characteristics of external diaphragm connections with steel CHS columns and wide-flange steel beams. *Pacific Structural Steel Conference 2007, Proc*. **2,** 123-129
Kawashima T. and Ogawa K. (2007). Investigation on column-to-beam strength ratio and stiffness ratio. *Steel Construction Engineering* **14:53,** 23-32