16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N°3645 (Abstract ID)

Registration Code: S-N1463030129

EFFECTS OF CONNECTION DETAIL ON STRUCTURAL BEHAVIOR OF STEEL BRACED FRAMES

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Abstract

Braced frame structures are used for school gymnasiums and factories with large space. It is very important building which will be used as emergency public shelters in time of disaster. In braced frames, generally the columns are placed along their weak axis for wide-flange steel cross sections and the beam web is connected through gusset plates on the column web by using high strength bolts. These beam-to-column connections are assumed as a pin connection at the stage of seismic design, and seismic performance of the braced frames depends on the strength of the tension-only brace. Connection of these components is constructed by a general design practice, although its detail is very complex. In other words, the effect of various connection details on the seismic performance of the braced frame is unclear at the stage of seismic design.

This paper focuses on differences in connection details, and describes their effects on structural behaviors of braced frame structures. The cyclic loading tests of 2 story-1 bay full-scale braced frames with various connections were carried out. The main parameters are: a connection eccentricity of brace, details at beam-end connection and presence of concrete slabs. Connection eccentricity of brace is selected in order to investigate the additional stress on the framing components. Details at beam-end connection are adopted for the purpose of verifying the influence on the seismic performance of braced frames. Concrete slabs on beams are prepared to investigate the effect on strength of a braced frame.

The test results can be summarized as follows: (1) connection eccentricity of brace led to a decrease in the maximum strength and an elastic stiffness of the braced frame; (2) connection detail with narrow pitch of bolts or thin beam web caused local buckling at the beam web; (3) a presence of concrete slabs on beams resulted in increase in lateral strength of the beam-column subassemblies.

Keywords: braced frame; connection eccentricity of brace; maximum strength; elastic stiffness; deformation capacity

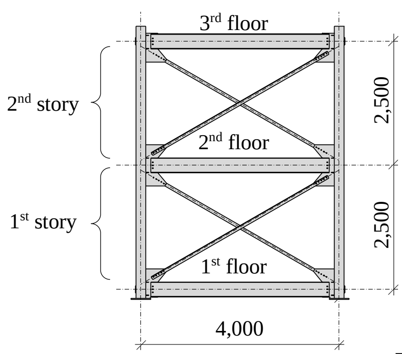
1. Introduction

Braced frame structures are used for school gymnasiums and factories with large space. It is very important building which will be used as emergency public shelters in time of disaster. In exterior frames of the structure, generally the columns are placed along their weak axis for wide-flange steel cross sections and the beam web is connected through gusset plates on the column web by using high strength bolts. These beam-to-column connections are assumed as a pin connection at the stage of seismic design, and seismic performance of the braced frames depends on the strength of tension-only braces. Connection of these components is constructed by a general design practice, although its detail is very complex. In other words, the effect of various connection details on the seismic performance of the braced frame is unclear at the stage of seismic design. This paper focuses on differences in connection details, and describes their effects on structural behaviors of braced frame structures.

2. Test Program

2.1 Outline of specimen

The outline of specimen is illustrated in Fig. 1. All specimens are 2 story-1 bay full-scale braced frames which are consisted of columns, beams and tension-only braces with the span of 4.0m and the story height of 2.5m. The columns and the beams of a specimen are wide-flange steel placed along weak axis and strong axis, respectively. Gusset plates (PL9) for connecting a beam and braces are joined to a column web by fillet welding. The beam-end connections are the pin details which are connected only to the beam web by high strength bolts. The braces are set in X shape, and their connections are designed in order to meet the condition of the joint with the load-carrying capacity.



Column: RH-400x200x8x13 or BH-285x170x6x6

Beam: RH-300x150x6.5x9 or BH-300x150x4.5x6

Brace: L-65x65x6 or FB-80x16

Gusset plate: PL9

Bolt at brace connection: 5-M16(F10T)

Bolt at beam connection: 5-M16(F10T) or 5-M20(F10T)

Fig. 1 – Outline of specimen

2.2 Test setup

The test setup is shown in Fig. 2. The column bases of specimens are fixed to the strong floor via pin-roller jigs which are free in the horizontal direction. In addition, loading beams are placed at the 1st and 3rd floor levels around the specimens, and they are connected to a reaction wall by way of horizontal reaction jigs and 1000kN oil jack. The load from the oil jack transfers to the loading beams, a specimen, the reaction beams, and finally returns to the reaction wall via the horizontal reaction jigs. This loading method realizes the situation that almost the same axial compressive force is applied to the beams in all floors. The repeated cyclic loading controlled by the average story drift (R) of the 2 story braced frame is carried out.

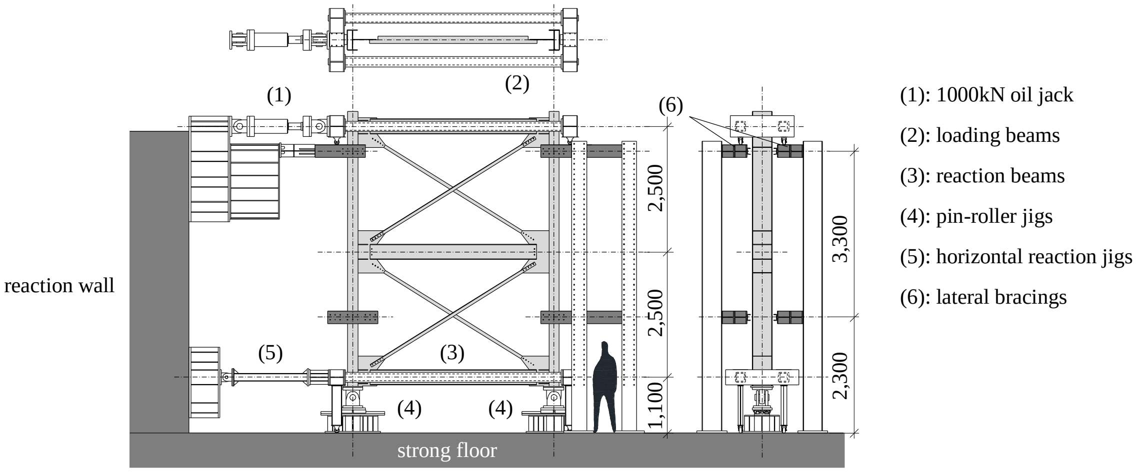


Fig. 2 – Test setup

2.3 Test specimens and parameters

The definition of test parameters and the list of all specimens are represented in Fig. 3 and Table 1. Test parameters are: the pitch of bolts at the beam-end connections (p), the diameter of bolts, the connection eccentricity of braces (e), the cross section of framing components (columns, beams and braces) and the presence of concrete slabs. The standard specimen (Le-0) is the braced frame which has the beam-end connection using M16 bolts arranged by the Japanese standard pitch (60mm), without the connection eccentricity of brace and a concrete slab. Based on this specimen, 13 specimens in total are prepared.

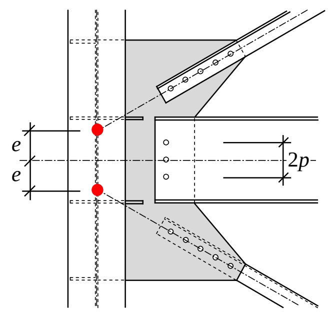
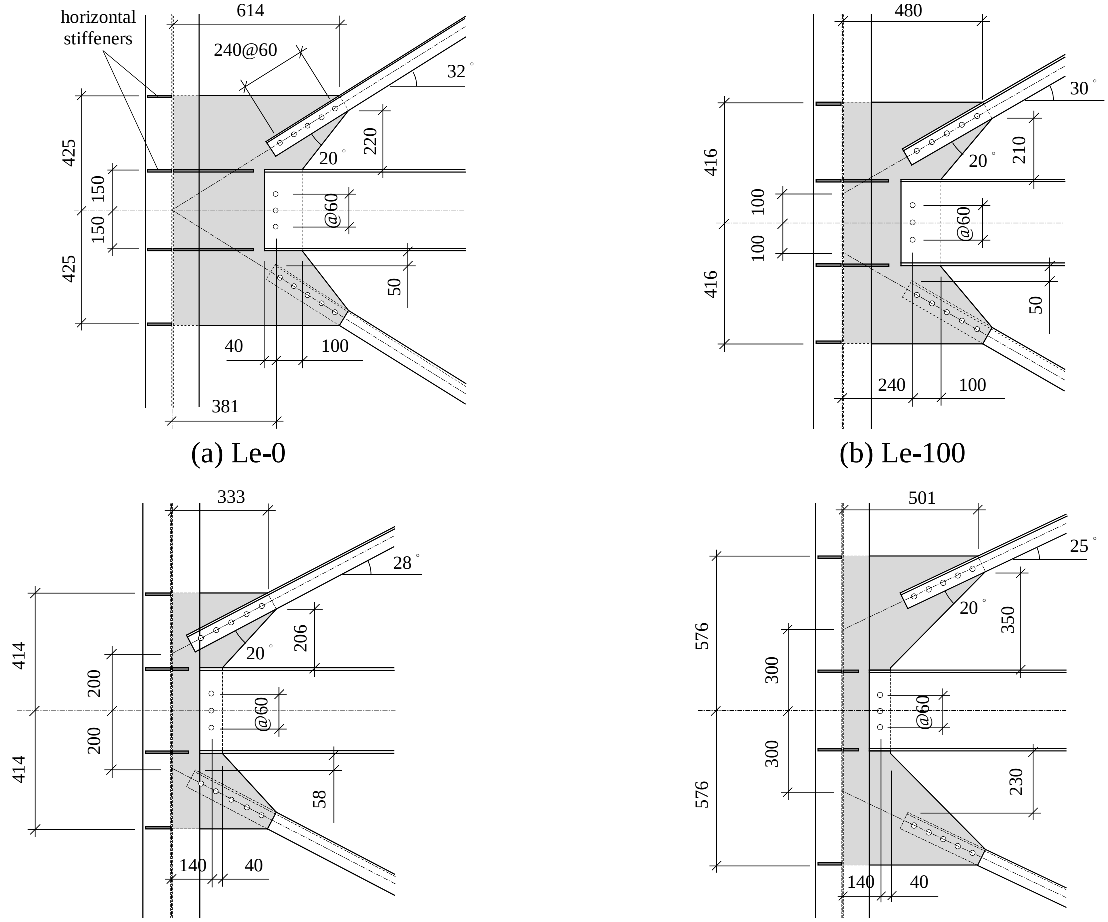


Fig. 3 – Definition of test parameters

Table 1 – List of test specimens

specimens	columns	beams	braces	pitch of bolts at beam-end connections p [mm]	diameter of bolts at beam-end connections	connection eccentricity of braces e [mm]	
Le-0	RH-400x200x8x13	RH-300x150x6.5x9	L-65x65x6	60	M16	0	Column series
Le-100						100	
Le-200						200	
Le-300						300	
Se-0	BH-285x170x6x6		L-65x65x6 (with gusset plate at intersection of braces)	80	M20	0	
Se-100						100	
Se-200						200	
Se-300						300	
Bp-40	RH-400x200x8x13	BH-300x150x4.5x6	L-65x65x6	40	M16	0	Beam series
Bp-80				80			
Lp-40		RH-300x150x6.5x9		40			
Lp-80			FB-80x16 (with gusset plate at intersection of braces)	80			
Slab	RH-400x200x8x13	RH-300x150x6.5x9 (with concrete slab)	L-65x65x6 (with gusset plate at intersection of braces)	60	M16	0	

The connection details in each connection eccentricity of braces are shown in Fig. 4 with examples of Le series. Shapes of all gusset plates are decided for securing effective cross-sectional area. Additionally, horizontal stiffeners are placed in columns at edges of gusset plates and beam flange levels in order to facilitate stress transfers in connections.



(a) Le-0 (b) Le-100

(c) Le-200 (d) Le-300

Fig. 4 – Connection details of each connection eccentricity of brace (Le-0~300)

The horizontal shear strength corresponding to the number of headed studs which are used for specimen with concrete slab is shown in Fig. 5. The horizontal shear strength means the transfer capability of the axial force between a concrete slab and a steel beam. The vertical axis represents the horizontal shear strength, and the horizontal axis stands for the number of headed studs. Moreover, broken lines in Fig. 5 indicate strengths corresponding to failure modes caused by the axial force of the beam with a concrete slab. Further, the horizontal shear strength which is required to meet the condition of an incomplete composite beam in assuming the beam-end connection rigid is indicated by the red line [1]. In these tests, the number of headed studs was determined so that the horizontal shear strength exceeds the horizontal component of yield strength of brace and the condition of an incomplete composite beam.

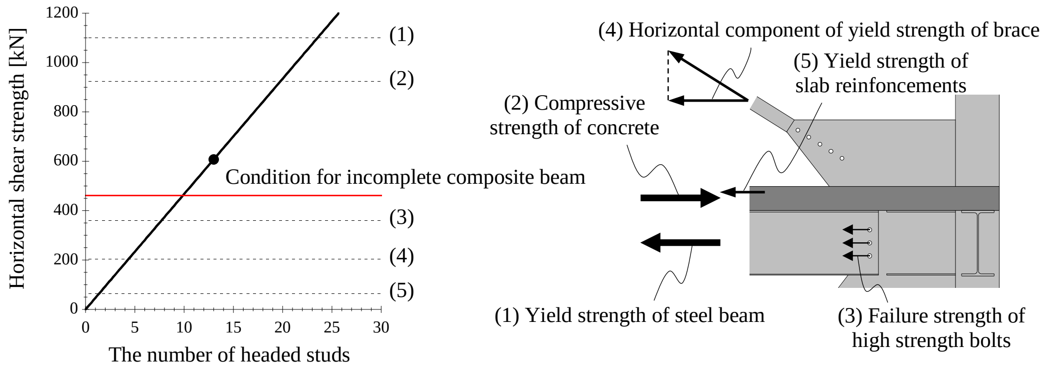


Fig. 5 – Relationship between the horizontal shear strength and the number of headed studs

3. Test Results

3.1 Effects of connection eccentricity of brace

First of all, test results in the column series (specimen Le in Table 1) are discussed, and influences of the connection eccentricity of the brace on the structural behavior of braced frames are considered. Relationships between the average story drift *R* and the story shear *Q* in the specimens are shown in Fig. 6. The horizontal axis represents the average story drift, and the vertical axis stands for the story shear. In addition, broken lines indicate the horizontal component of yield strength of tension-only brace. All specimens are in almost elastic range and demonstrate large stiffness until the average story drift reaches ±0.005 rad. After the maximum strengths in each specimens are caused by the yield of tension-only braces, after that, the maximum strengths are almost constantly maintained. The slip behavior is confirmed under the repeated cyclic loading, and the strengths of the second cycle are lower than ones of the first cycle in the same amplitude. Also, it is found that larger connection eccentricity of brace brings the lower maximum strength and elastic stiffness.

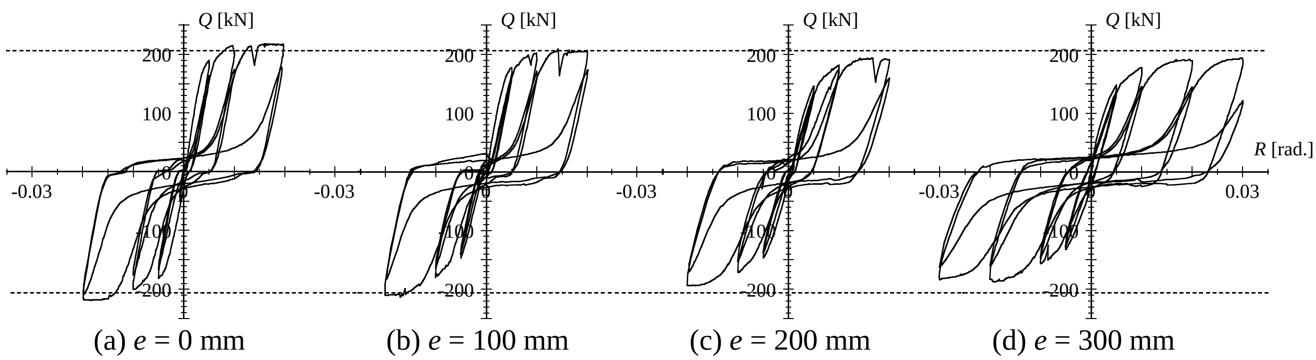
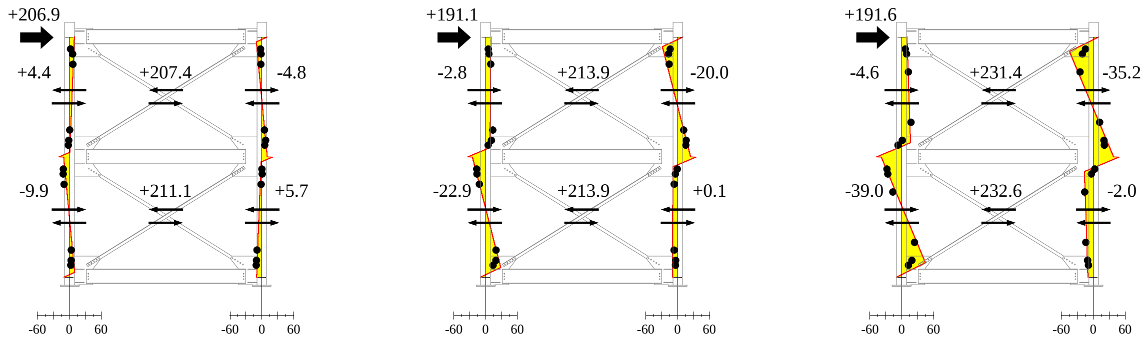


Fig. 6 – Relationships between average story drift and story shear (Le-0~300)

Bending moment distributions of column at the average story drift of +0.02 rad are plotted in Fig. 7. As the connection eccentricity of brace is larger, it is found that the bending moment increases and moment gradients are reversed. Namely, the shear force of columns which have a large connection eccentricity of brace becomes the reversed shear force — which is the shear force acting in the same direction as an external force. Therefore, it is clear that the decreases in the maximum strength and the elastic stiffness are caused by the reversed additional bending moments due to the connection eccentricities of braces.



(a) e = 100 mm (b) e = 200 mm (c) e = 300 mm

Fig. 7 – Bending moment distribution and shear force of column at average story drift of +0.02 rad. (Le-0~300)

Here, the calculation method for the additional bending moment and the reversed shear of column according to the connection eccentricity of brace is investigated. It should be noted that, the horizontal forces of tension-only braces are considered because ones of the compressive braces are negligible in comparison to the horizontal component of yield strength of brace. Assuming that there is no bending moment at the beam-end connections, the one side of columns is modeled as the statically indeterminate structure illustrated in Fig. 8. The additional bending moment in each node is represented as the following equations under the assumption that the horizontal force of tension-only braces in each story equals.

$$ {}_{\mathrm{c}}M_1 = -\frac{(H-e)(4H-3e)}{4H^2} \cdot {}_{\mathrm{b}}Q \cdot e \tag{1} $$

$$ {}_{\mathrm{c}}M_2 = \frac{3(H-e)}{4H} \cdot {}_{\mathrm{b}}Q \cdot e \tag{2} $$

$$ {}_{\mathrm{c}}M_3 = -\frac{(H-e)(H+3e)}{4H^2} \cdot {}_{\mathrm{b}}Q \cdot e \tag{3} $$

Moreover, the shear force of column in each story due to the additional bending moment is calculated as the below equations.

$$ {}_{\mathrm{c}}Q_1 = -\frac{e(7H-3e)}{4H^2} \cdot {}_{\mathrm{b}}Q \tag{4} $$

$$ {}_{\mathrm{c}}Q_2 = -\frac{e(H+3e)}{4H^2} \cdot {}_{\mathrm{b}}Q \tag{5} $$

The shear forces of both columns are calculated as the summation of ${}_{\mathrm{c}}Q_1$ and ${}_{\mathrm{c}}Q_2$ because the bending moment distributions of the both columns are the point-symmetrical.

$$ {}_{\mathrm{c}}Q_{\mathrm{n}} = {}_{\mathrm{c}}Q_1 + {}_{\mathrm{c}}Q_2 = -\frac{2e}{H} \cdot {}_{\mathrm{b}}Q \tag{6} $$

The bending moment distributions obtained from Eq. (1) ~ (3) are compared to test results. It is necessary to consider the bending moments at the beam-end connections for the actual the bending moment distributions of columns. The bending moment distribution of columns based on the slip strength at beam-end connections is shown in Fig. 9 when the bending moments at the beam-end connections on the 2nd floor are equally distributed to both upper and lower columns. ${}_{\mathrm{b}}M_{\mathrm{c}}$ in Fig. 9 is the bending moment at beam-column node caused by the slip strength of high strength bolts at the beam-end connection. The bending moment distributions of columns obtained by superimposing the bending moment due to the connection eccentricities of braces (Fig. 8) and the bending moment because of the slip strength of high strength bolts at the beam-end connection (Fig. 9) are shown in Fig.7 by solid lines and the yellow fills. The calculated values of the bending moment distribution show good agreement with the test results. Therefore, it can be said that the stress distribution obtained by modeling a column as the statically indeterminate structure makes the evaluation of the additional bending moment of column caused by the connection eccentricity of brace possible.

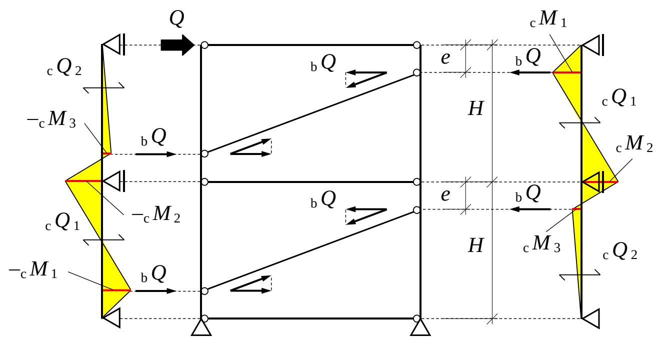


Fig. 8 – Stress distribution of column with connection eccentricity of brace

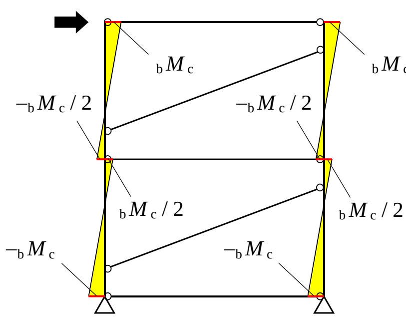


Fig. 9 – Bending moment distribution of column due to the bending moments at the beam-end connections

Finally, the evaluation method of the maximum strength and the elastic stiffness is investigated. The relationships between the connection eccentricity of brace and the maximum strength or the elastic stiffness in column series are plotted in Fig. 10. The vertical axes stand for the maximum strength (Q_{max}) or the elastic stiffness (K) divided by one of specimens which have no connection eccentricity of brace (Q_{0max} or K_0), respectively. The horizontal axes represent the connection eccentricity of brace (e) divided by the story height (H). As previously mentioned, it is indicated that both of the maximum strength and the elastic stiffness decrease in proportion to the connection eccentricity of brace.

At first, the maximum strength is evaluated. Considering the reserved shear obtained by Eq. (6), the maximum strength is calculated by using the horizontal component of the yield strength of brace (${}_bQ_y$) and the below equation.

$$Q_{max} = \left(1 - \frac{2e}{H}\right) \cdot {}_bQ_y \tag{7}$$

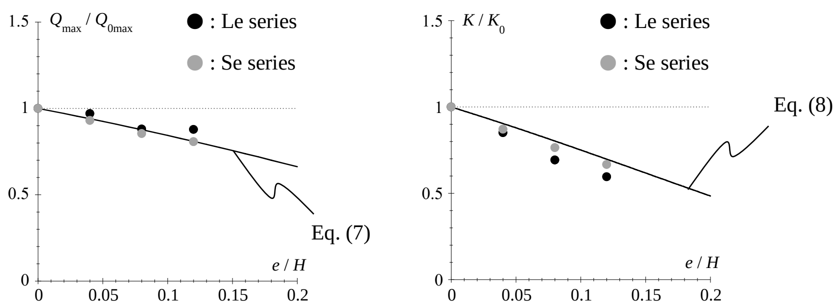
The calculated strength is expressed as the solid line in Fig. 10 (a). The calculated value shows good agreement with the experimental values, and it indicates that Eq. (7) enables to evaluate the maximum strength of braced frames with the connection eccentricity of braces.

On the other hand, the elastic stiffness of braced frames based on the axial stiffness of the tension-only braces (${}_bk$) is represented as the following equation.

$$K = \frac{{}_bk(H-2e)^2\cos^2\theta}{2H^2} \tag{8}$$

θ: a mounting angle of brace

The calculated elastic stiffness is shown as the solid line in Fig. 10 (b). The calculated value shows good agreement with the test results. Therefore, the elastic stiffness of braced frames in these tests can be estimated by Eq. (8).

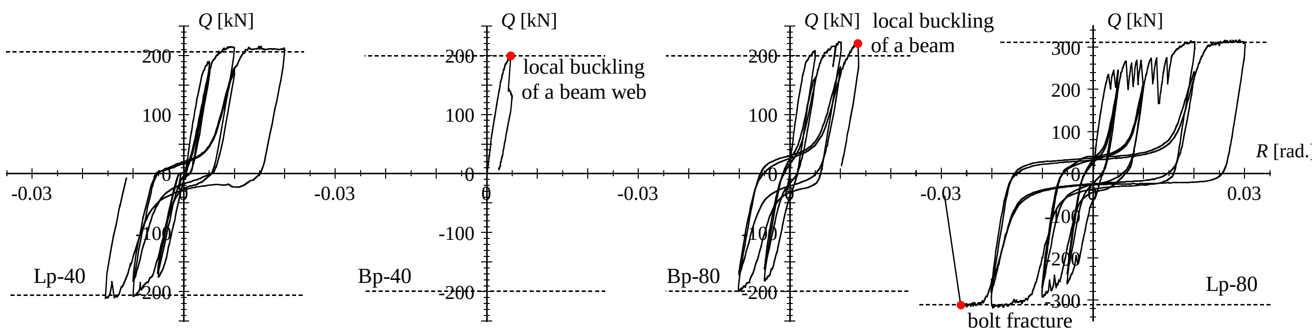


(a) Maximum strength (b) Elastic stiffness

Fig. 10 – Relationships between maximum strength or elastic stiffness and connection eccentricity of brace

3.2 Effects of details at beam-end connection

Test results in the beam series are discussed, and influences of the connection details at the beam-end connection on the deformation capacity of braced frames are investigated. Relationships between the average story drift R and the story shear Q in beam series are shown in Fig. 11. Although (a) specimen Lp-40 has the narrower pitch of bolts at the beam-end connection than the standard specimen, it is observed that the pitch of bolts at the beam-end connection has few influences on structural behavior of the braced frame. On the other hand, in specimen Bp-40 which has thinner beam webs, the local buckling of a beam web at a beam-end connection occurred before the average story drift reached at +0.005 rad, and then its strength dropped rapidly. Although the local buckling of a beam web at a beam-end connection similarly occurred on (c) specimen Bp-80, widening the pitch of bolts at the beam-end connections greatly improved the deformation capacity. In addition, as a result of (d) specimen Lp-80 which has wide pitch of bolts at the beam-end connections and the braces with a larger cross-sectional area, large axial compressive forces from braces caused the fracture of the high strength bolts at a beam-end connection.



(a) p = 40 mm, t_w = 6.5mm (b) p = 40 mm, t_w = 4.5mm (c) p = 80 mm, t_w = 4.5mm (d) p = 80 mm, t_w = 6.5mm

Fig. 11 – Relationships between average story drift and story shear (Lp-40, Bp-40, Bp-80 and Lp-80)

In the previous research [2], cyclic loading tests of pin-detailed beam-end connection under compression were carried out, and then a requirement to achieve the sufficient rotation capacity (±0.03 rad) is proposed. It assumes that the pin detailed beam-end connection has the effective cross section illustrated in Fig. 12, it is clarified that the sufficient rotation capacity is secured unless the effective stress exceeds 150 N/mm^2. In this paper, the applicability of the proposed requirement is verified. The stress at the effective cross section is represented in Fig. 13. The vertical axis (σ) indicates the stress at the effective cross section obtained from dividing the maximum strength by the effective cross section in each specimen. The effective stresses in specimens (Bp-40, Bp-80 and Lp-80), which were collapsed by the local buckling or the bolt fractures, exceeded

150 N/mm^2 of the upper-limit stress established in the previous research [2]. Therefore, it is proved that the requirement for the sufficient deformation capacity is applicable to the braced frames.

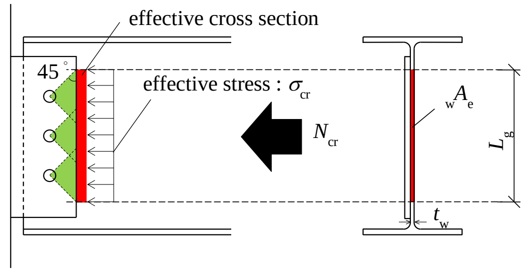


Fig. 12 – Effective cross section of a beam web at a beam-end connection

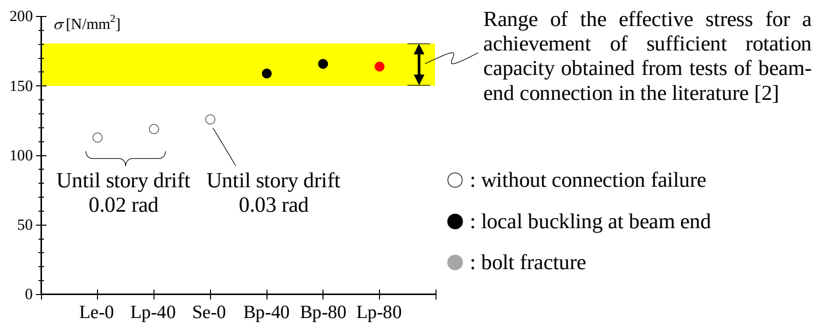
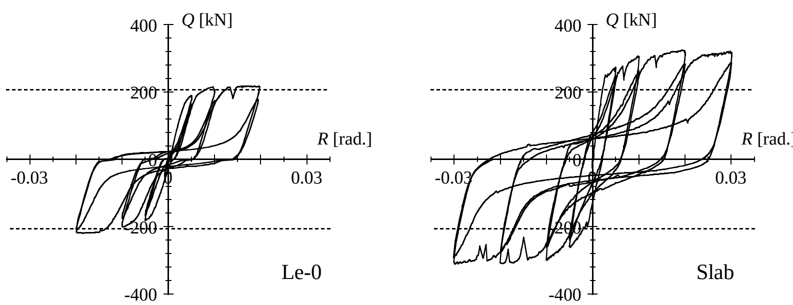


Fig. 13 – Stress at the effective cross section

3.3 Effects of concrete slab

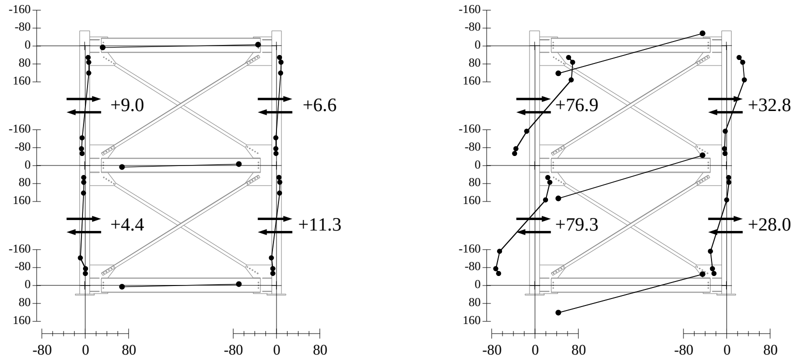
Finally, test results of specimen Slab are discussed, and effects of the presence of concrete slabs on the structural behavior of braced frames are considered. Relationships between the average story drift R and story shear Q of the standard specimen Le-0 and specimen Slab are illustrated in Fig. 14. It is observed that the maximum strength of specimen Slab exceeds that of the standard specimen by more than 100 kN. It indicates that bending moments at beam-end connections increase due to compressive resistances of the concrete slabs, and lateral strength of the beam-column subassemblies increases.



(a) Without concrete slab (b) With concrete slab

Fig. 14 – Relationship between average story drift and story shear (Le-0 and Slab)

Bending moment distributions of column at the average story drift of +0.02 rad are plotted in Fig. 14. It is confirmed that large bending moment about 70 kNm is applied to beam-column nodes at the side of the beam-end under the positive bending moment. On the other hand, full-plastic moment of column is 79.7 kNm. As considering that the additional axial force, it can be said that the column might lead to a plasticity.



(a) Without concrete slab (Le-0) (b) With concrete slab (Slab)

Fig. 14 – Bending moment distribution and shear force of column at the average story drift of +0.02 rad. (Le-0 and Slab)

4. Conclusion

In this paper, the cyclic loading tests of 2 story-1 bay full-scale braced frames with various connections were carried out, and effects of differences in connection details on the seismic performance of the braced frames are investigated. Test results obtained in this paper are summarized as the following: (1) in braced frames which have the connection eccentricities of braces, the additional stress due to the connection eccentricities of braces reduces strength and stiffness; (2) the maximum strength and the elastic stiffness can be evaluated by considering the negative shears of columns; (3) the requirement for the sufficient deformation capacity proposed in the literature [2] is applicable to braced frames; (4) in braced frames which have the concrete slabs on beams, the flexural strength of beam-end connections assumed as pin connections increases, and lateral strength of the beam-column subassemblies increase.

5. Acknowledgements

This research was supported by Building Standard Promotion of Ministry of Land, Infrastructure, Transport and Tourism in 2013 and 2014 fiscal year and Grant-in-Aid for Scientific Research (C) of JSPS KAKENHI Grant Number 26420569.

6. References

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