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Seismic performance of the retrofitting method by framed steel bracing system partially connected using joint anchors

*Yuji Takahashi[1], Eiichi Inai[2], Takanori Kawamoto[3],Ken Harayama[4] and Takuya Kakihara[5]

[1), 2)] Graduate School of Science and Eng., Yamaguchi Univ., 2-16-1, Tokiwadai, Ube, Yamaguchi 755-8611, Japan
[3), 4), 5)] Construction Material Div., Ube Industries, LTD.
[1)] uz.g.zzz@gmail.com

ABSTRACT

The authors have proposed a seismic retrofitting method by the framed steel bracing system partially connected using joint anchors for existing reinforced concrete frames. The failure mechanism of the reinforced concrete frame retrofitted by this method becomes the horizontal joint failure accompanying the punching shear failure at the top of the tensile column in many cases. In this paper, a design equation for the lateral strength of the retrofitted frame in the horizontal joint failure mode is proposed and verified using the test results of nine specimens. It is concluded that the proposed design equation is effective for the strength evaluation.

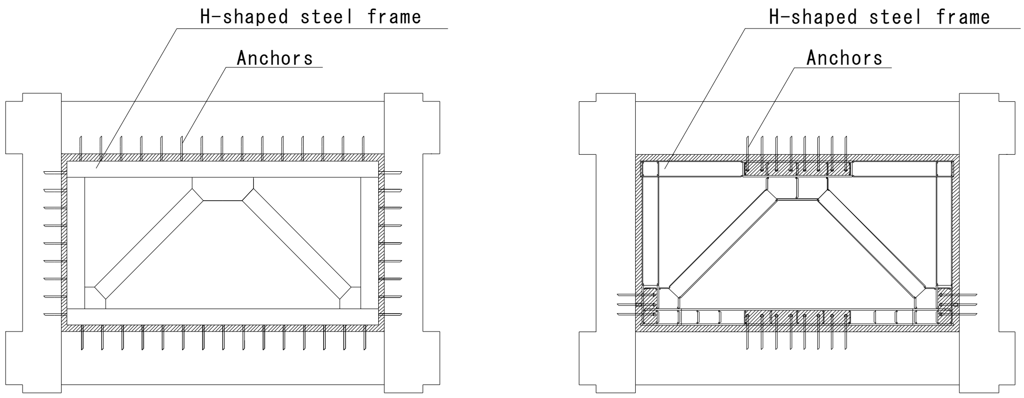
1. INTRODUCTION

In the seismic retrofit of existing reinforced concrete buildings, a seismic retrofitting method installing framed steel braces in the existing reinforced concrete frames as shown in Fig. 1 (a) has been generally used in Japan. However, in this method, the improvement of workability is strongly required because noises, dusts and vibration occur in the installation work of joint anchors. The authors have proposed a retrofitting method by the framed steel bracing system partially connected using joint anchors as shown in Fig. 1 (b). This retrofitting method has a possibility to reduce the amount of joint anchors between the existing reinforced concrete frame and the steel frame having the braces. From the loading tests of five 1/2.5 scaled one bay one story specimen frames retrofitted by this method in Takanori Kawamoto (2010) and Ken Harayama (2012), it was found that the horizontal joint failure accompanying the punching shear failure at the top of the tensile column occurred before yielding of the braces in case that the horizontal joint has a relatively small amount of joint anchors. It

[1)] Graduate Student
[2)] Professor, Dr. Eng.
[3)] Manager, Dr. Eng.
[4)] Engineer, Dr. Eng.
[5)] Engineer, M. Eng.

was also found that the retrofitted frame had a sufficient lateral strength and a deformation capacity even if the horizontal joint failure occurred. However, the evaluating method for the lateral strength of the retrofitted frame in the horizontal joint failure mode has not been sufficiently established to use its strength for the actual retrofit.

In this paper, at first, test results of new five 1/2.5 scaled one bay one story specimen frames with various types of brace; H-shaped steel brace, pin-ended hollow tube brace, mansard type brace, are reported. The concrete strength and the ratio of the height to the span of the existing frame are also test parameters. Then, a design equation for the lateral strength of the retrofitted frame in the horizontal joint failure mode is proposed and verified. A total of nine specimens with the horizontal joint failure including four specimens in Takanori Kawamoto (2010) and Ken Harayama (2012) were used for the verification.



(a) Ordinary connection (b) Partial connection

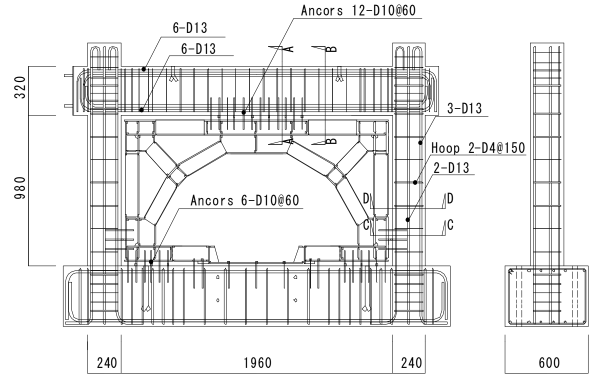
Fig. 1 Reinforced concrete frames retrofitted by framed steel brace system

2. SPECIMENS

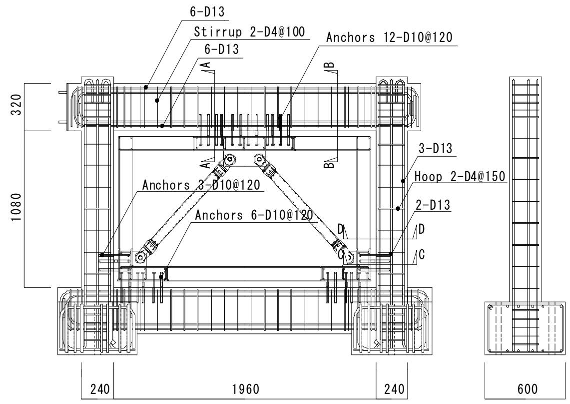
New five specimens are shown in Fig. 2 and are listed in Table 1. All the specimens are 1/2.5 scaled one bay one story RC frame retrofitted by the steel braces. The sections of the specimen frames of PA6, PA12, PA16 and PA21 are shown in Fig. 3, and those of the specimen frames of PA17 are shown in Fig. 4. The concrete strength of the specimen frames is 8.0 – 14.4 N/mm^2. The mechanical properties of the steel materials used for the specimens are shown in Table 2.

PA6 is the frame retrofitted by mansard type brace (H-100×100×3.2×4.5). PA12 is the frame retrofitted by pin-ended hollow tube brace (70ϕ-5). PA16, PA17 and PA21 are the frames retrofitted by H-shaped steel brace (H-100×100×3.2×4.5). The span of the frame of PA21 is shorter than the other specimens.

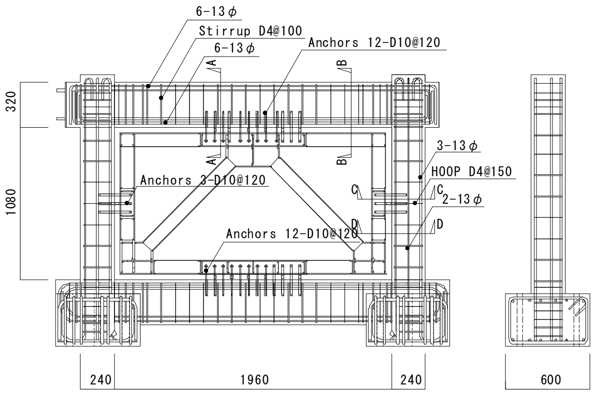
The failure mechanism of the reinforced concrete frame without the braces of PA6, PA12, PA16 and PA21 is designed to be the shear failure of the columns. The failure mechanism of the reinforced concrete frame without the braces of PA17 is designed to be the shear failure of the upper beam. The failure mechanism of the five retrofitted frames is designed to be the horizontal joint failure accompanying the punching shear failure at the top of the tensile column.



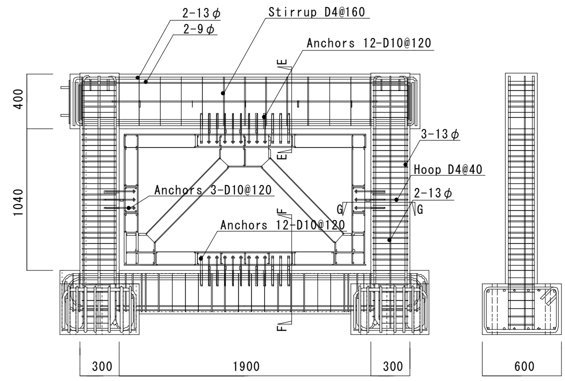
(a) PA6



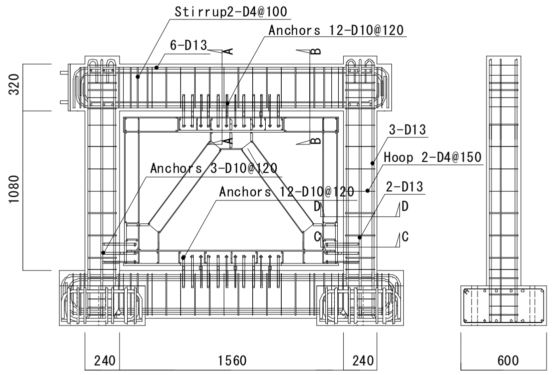
(b) PA12



(c) PA16



(d) PA17



(e) PA21

Fig. 2 Details of specimens

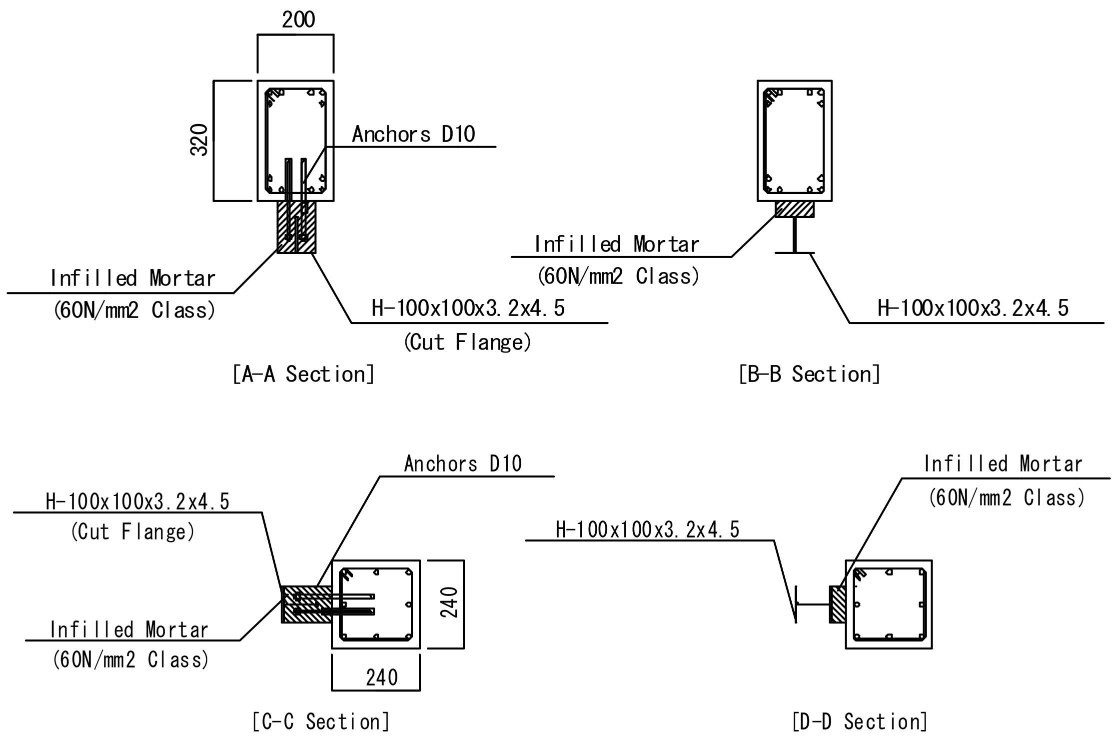


Fig. 3 Sections of specimen frames of PA6, PA12, PA16 and PA21

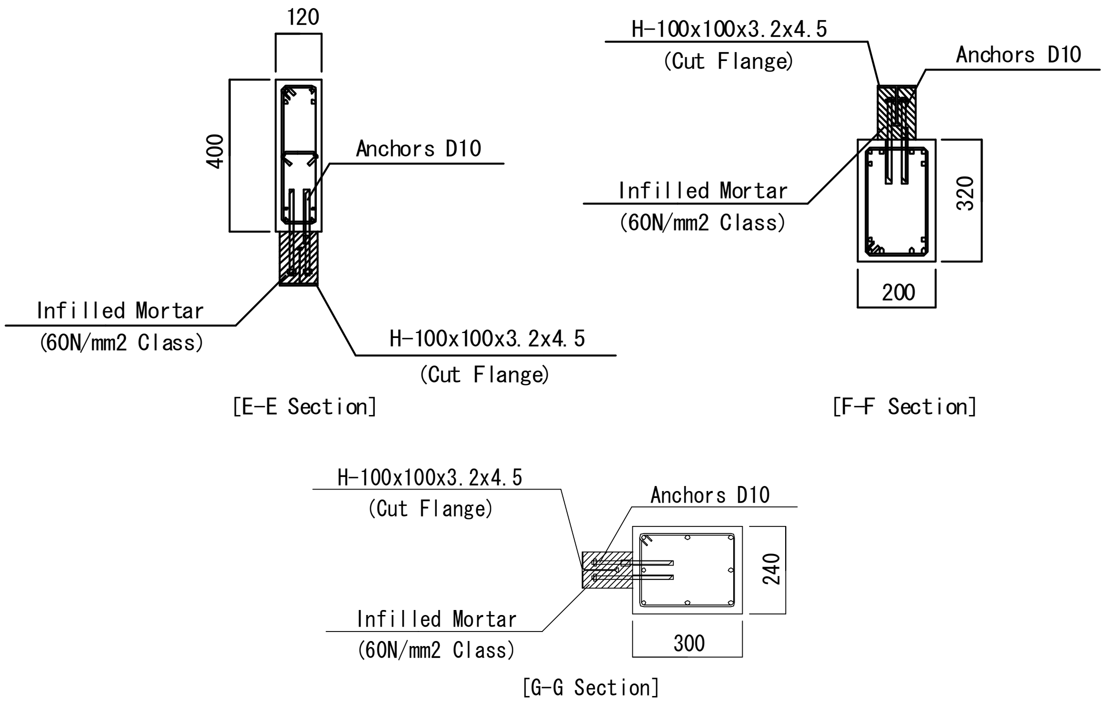


Fig. 4 Sections of specimen frame of PA17

Table 1 List of specimens

<table>
<tr><th rowspan="2">Specimen</th><th rowspan="2">Concrete Strength of Existing Frame(N/mm^2)</th><th colspan="3">Column</th><th colspan="3">Upper Beam</th><th colspan="3">Lower Beam</th><th rowspan="2">Horizontal Joint Anchors</th><th rowspan="2">Brace Type</th><th rowspan="2">Ratio of Height to Span</th></tr>
<tr><th>Section</th><th>Main Re-Bars</th><th>Hoop</th><th>Section</th><th>Main Re-Bars</th><th>Stirrup</th><th>Section</th><th>Main Re-Bars</th><th>Stirrup</th></tr>
<tr><td>PA6</td><td>11.3</td><td rowspan="3">240mm×240mm</td><td rowspan="2">8-D13</td><td rowspan="2">2-D4@150</td><td rowspan="3">200mm×320mm</td><td rowspan="2">upper:6-D13
lower:6-D13</td><td rowspan="2">2-D4@100</td><td colspan="3"></td><td rowspan="5">12-D10</td><td>Mansard Type
H-100×100×3.2×4.5</td><td>0.59</td></tr>
<tr><td>PA12</td><td>10.9</td><td rowspan="4">200mm×320mm</td><td>upper:6-D13
lower:6-D13</td><td>2-D4@100</td><td>K Type
Pin-Ended
Hollow Tube Brace:70φ-5</td><td>0.64</td></tr>
<tr><td>PA16</td><td>8.0</td><td rowspan="2">8-13φ</td><td>D4@150</td><td>upper:6-13φ
lower:6-13φ</td><td>D4@100</td><td rowspan="2">upper:6-13φ
lower:6-13φ</td><td rowspan="2">D4@100</td><td rowspan="3">K Type
H-100×100×3.2×4.5</td><td>0.64</td></tr>
<tr><td>PA17</td><td>8.3</td><td>240mm×300mm</td><td>D4@40</td><td>120mm×400mm</td><td>upper:2-13φ
2-9φ
lower:2-13φ
2-9φ</td><td>2-D4@160</td><td>0.64</td></tr>
<tr><td>PA21</td><td>14.4</td><td>240mm×240mm</td><td>8-D13</td><td>2-D4@150</td><td>200mm×320mm</td><td>upper:6-D13
lower:6-D13</td><td>2-D4@100</td><td>upper:6-D13
lower:6-D13</td><td>2-D4@100</td><td>0.78</td></tr>
</table>

Table 2 Mechanical properties of steel material

(a) PA6

Steel Materials		Yield Strength (N/mm^2)	Tensile Strength (N/mm^2)
D4(SD295)		411	575
D10(SD295)		357	502
D13(SD295)		394	558
H-Shaped Steel Frame and Brace	Web t=3.2 (SS400)	373	446
	Flange t=4.5 (SS400)	339	428

(b) PA12

Steel Materials		Yield Strength (N/mm^2)	Tensile Strength (N/mm^2)
D4(SD295)		383	559
D10(SD295)		373	511
D13(SD295)		381	528
H-Shaped Steel Frame	Web t=3.2 (SS400)	357	435
	Flange t=4.5 (SS400)	289	418
Pin-Ended Hollow Tube Brace	t=5 (STKM13A)	409	506

(c) PA16

Steel Materials		Yield Strength (N/mm^2)	Tensile Strength (N/mm^2)
D4(SD295)		333	490
D10(SD295)		366	490
13φ(SR235)		339	452
H-Shaped Steel Frame and Brace	Web t=3.2 (SS400)	364	466
	Flange t=4.5 (SS400)	301	414

(d) PA17

Steel Materials		Yield Strength (N/mm^2)	Tensile Strength (N/mm^2)
D4(SD295)		333	490
D10(SD295)		366	490
13φ(SR235)		339	452
9φ(SR235)		363	478
H-Shaped Steel Frame and Brace	Web t=3.2 (SS400)	364	466
	Flange t=4.5 (SS400)	301	414

(e) PA21

Steel Materials		Yield Strength (N/mm^2)	Tensile Strength (N/mm^2)
D4(SD295)		368	510
D10(SD295)		380	520
D13(SD295)		368	531
H-Shaped Steel Frame and Brace	Web t=3.2 (SS400)	360	468
	Flange t=4.5 (SS400)	343	474

3. TEST PROCEDURE

Test setup is shown in Fig. 5. Loading schedule is shown in Fig. 6. The long term axial force was applied to the columns of the specimen frame and maintained at 170 kN which was corresponding to the standard axial load of the columns at the first story of five story RC school buildings. The lateral force was applied to the upper beam ends of the specimen frame. The loading was controlled by the story drift angle R, where R was δ/h; δ was the lateral displacement of the upper beam, h was the height of the specimen frame.

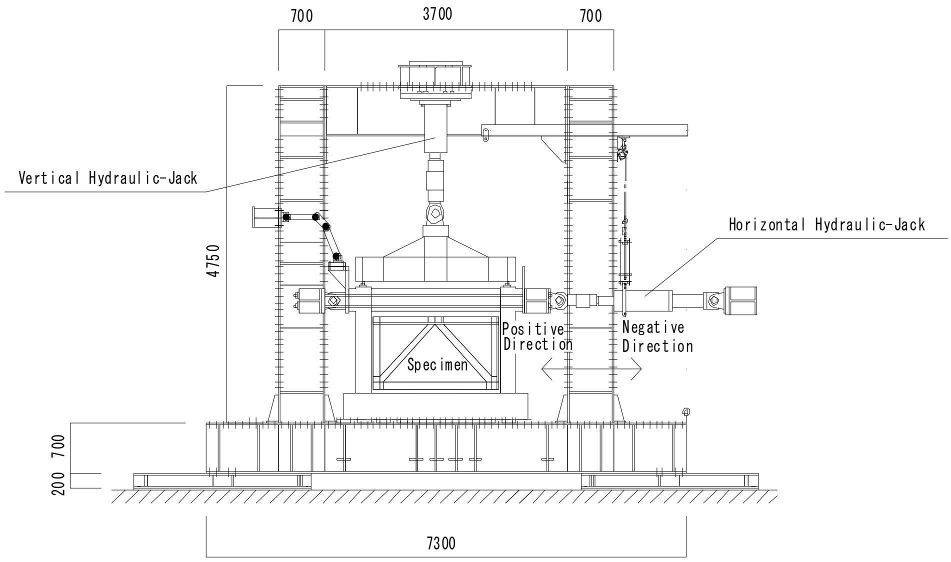


Fig. 5 Test setup

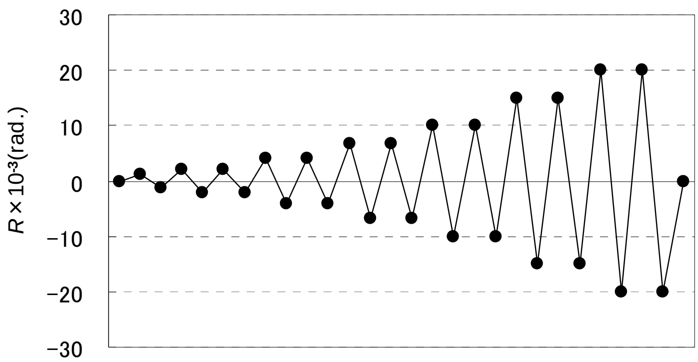


Fig. 6 Loading schedule

4. TEST RESULTS

The crack patterns of the specimens at R=0.01 radian are shown in Fig. 7. Shear force Q - story drift angle R relationships are shown in Fig. 8. The failure processes are described in the following.

In specimen PA6, the punching shear cracks occurred at the top of the tensile column in the cycle of R=0.002 rad. The cracks at the mortar of the upper horizontal joint occurred in the same cycle. And then, the cracks developed. The maximum lateral strengths in the positive and negative loading direction were observed in the cycle of R=0.0067 rad. Yielding of the steel brace was not observed. The final failure mechanism was the horizontal joint failure accompanying the punching shear failure at the top of the tensile column.

In specimen PA12, the punching shear cracks occurred at the top of the tensile column in the cycle of R=0.002 rad. The cracks at the mortar of the upper horizontal joint occurred in the cycle of R=0.004 rad. The maximum lateral strength in the negative loading direction was observed in the cycle of R=0.0067 rad. The maximum lateral strength in the positive loading direction was observed in the cycle of R=0.01 rad. Yielding of the steel brace was not observed. The final failure mechanism was the horizontal joint failure accompanying the punching shear failure at the top of the tensile column.

In specimen PA16, the bending cracks occurred at the bottoms of the tensile column and the compressive column in the cycle of R=0.002 rad. The shear cracks occurred at the tensile column in the same cycle. The punching shear cracks occurred at the top of the tensile column in the cycle of R=0.004 rad. The maximum lateral strength in the positive loading direction was observed in the same cycle. The maximum lateral strength in the negative loading direction was observed in the cycle of R=0.0067 rad. Yielding of the steel brace was not observed. The final failure mechanism was the horizontal joint failure accompanying the punching shear failure at the top of the tensile column.

In specimen PA17, the shear cracks occurred at the upper beam in the cycle of R=0.0013 rad. and then developed. The bending cracks occurred at the bottoms of the columns in the cycle of R=0.002 rad. The shear cracks occurred at the tensile column in the same cycle. Then, the punching shear cracks occurred at the top of the tensile column in the cycle of R=0.0067 rad. The maximum lateral strengths in the positive and negative loading direction were observed in the same cycle. Yielding of the steel brace was not observed. The final failure mechanism was the horizontal joint failure accompanying the punching shear failure at the top of the tensile column.

In specimen PA21, the punching shear cracks occurred at the top of the tensile column in the cycle of R=0.004 rad. The cracks at the mortar of the upper horizontal joint occurred in the same cycle. Yielding of the compressive steel brace occurred in the cycle of R=0.0067 rad. The maximum lateral strength in the negative loading direction was observed in the same cycle. The punching shear cracks and the cracks at the mortar of the horizontal joint greatly developed in the cycle of R=0.01 rad. The local buckling occurred at the steel brace, however, the lateral resistance continued to increase, and then the horizontal joint failure occurred. The maximum lateral strength in the positive loading direction was observed in the same cycle. The final failure mechanism was the horizontal joint failure accompanying the punching shear failure at the top of the tensile column.

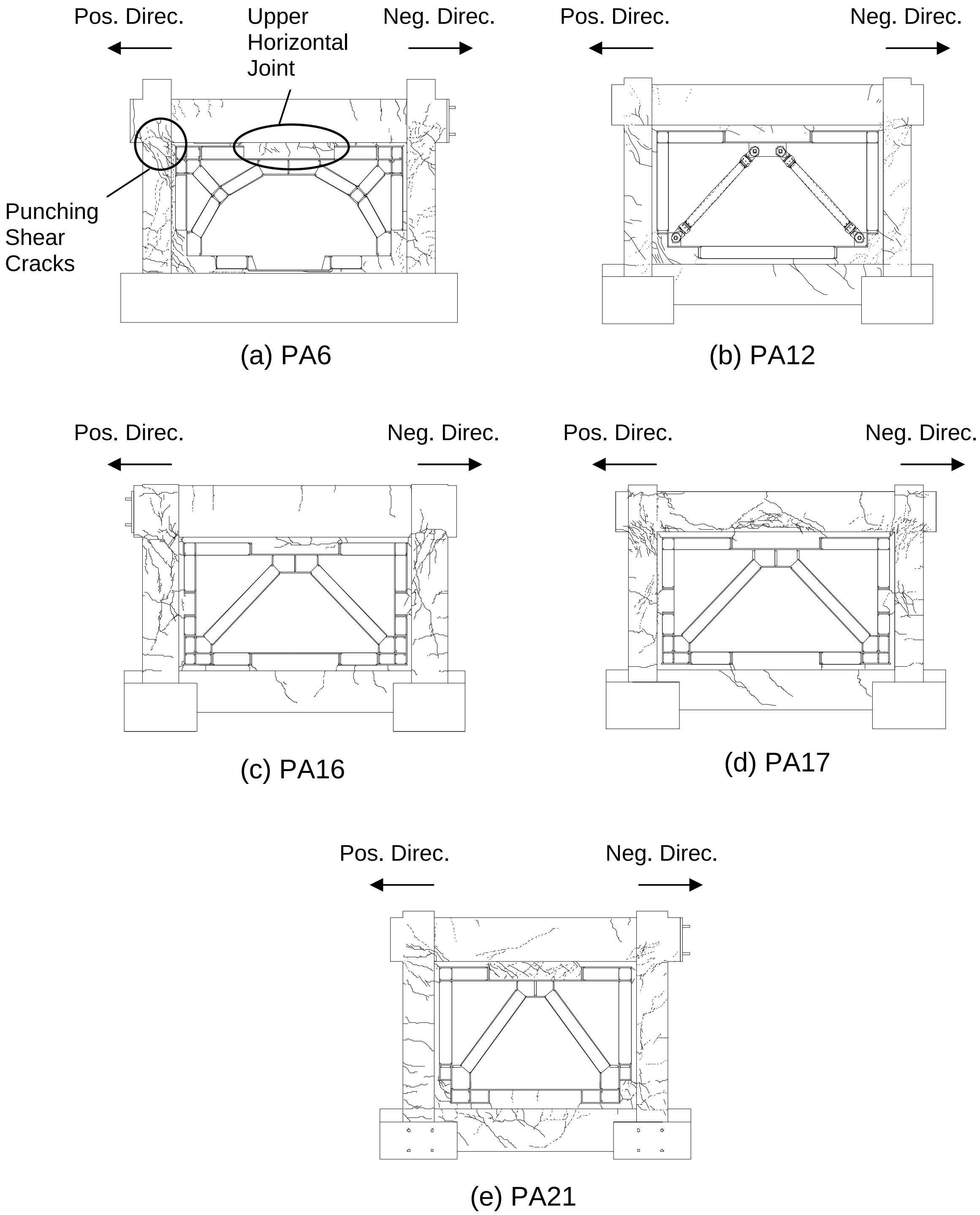


Fig. 7 Crack patterns of specimens at R=0.01rad.

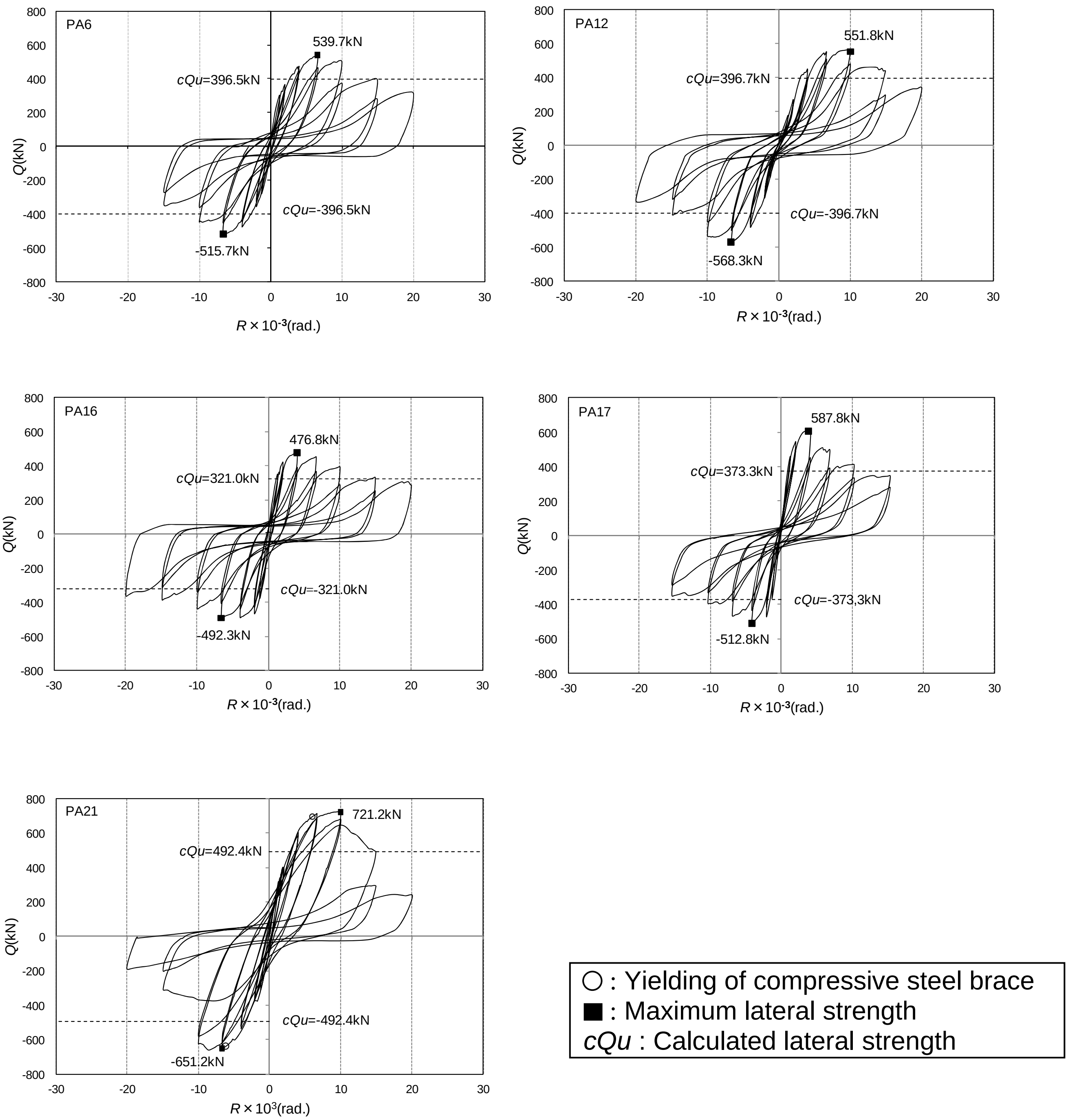


Fig. 8 Shear force Q - story drift angle R relationships

5. EVALUATION OF LATERAL STRENGTH

The idealized lateral resistance mechanism of the retrofitted frame in the horizontal joint failure mode is shown in Fig. 9. The design equation for the lateral strength of the retrofitted frame in the horizontal joint failure mode consists of the following four components on the basis of this mechanism; 1) the shear strength of the upper horizontal joint anchors ${}_aQ_j$, 2) the strength of punching shear failure at the top of the tensile column ${}_pQ_c$, 3) the lateral strength of the compressive column Q_{c2}, and 4) the frictional resistance between the existing reinforced concrete beam and the steel frame ${}_fQ_j$. The design equation is expressed by Eq. (1).

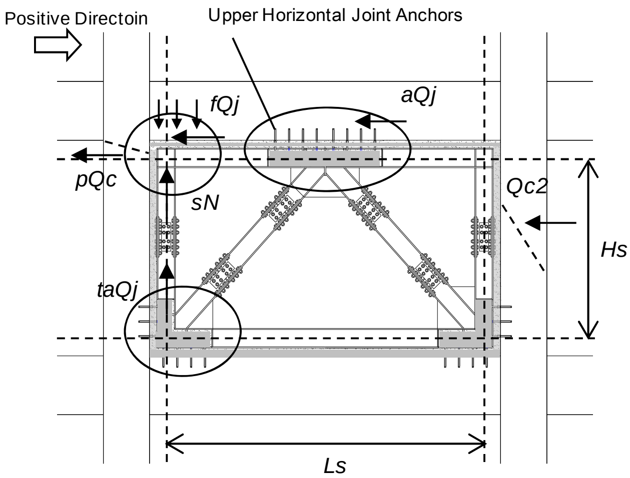


Fig. 9 Idealized lateral resistance mechanism of the retrofitted frame in the horizontal joint failure mode

$$ {}_cQ_u = {}_pQ_c + {}_aQ_j + {}_fQ_j + Q_{c2} \tag{1}$$

$$ {}_fQ_j = \mu \cdot {}_sN \tag{2}$$

$$ {}_sN = \left({}_pQ_c + {}_aQ_j \right) \cdot H_s / L_s - {}_{ta}Q_j \tag{3}$$

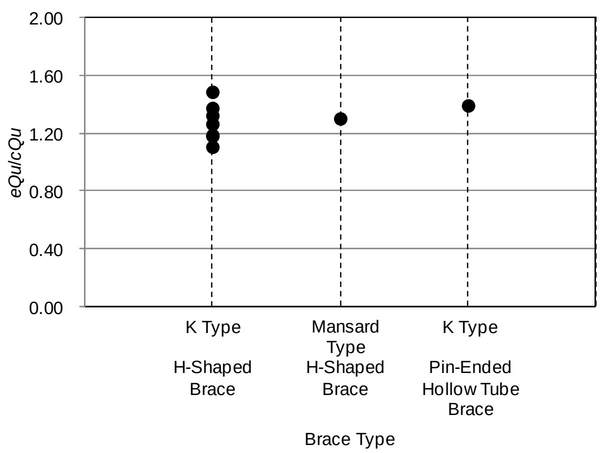
where ${}_cQ_u$=lateral strength of the retrofitted frame in the horizontal joint failure mode (kN); ${}_pQ_c$=punching shear strength of the tension column (kN) in JBDPA (2001); ${}_aQ_j$=shear strength of the upper horizontal joint anchors (kN) in JBDPA (2001); ${}_fQ_j$=frictional resistance between the existing reinforced concrete beam and the steel frame (kN) and is expressed by Eq.(2); μ=coefficient of friction and is assumed to be 0.5; ${}_sN$=compressive load of the vertical steel frame in the neighborhood of the tensile column (kN) and is expressed by Eq.(3); H_s=height of the steel frame; L_s=span of the steel frame; ${}_{ta}Q_j$=vertical strength obtained by adding 1/3 of the tensile strength of the lower horizontal joint anchors to the shear strength of the lower vertical joint anchors (kN); and Q_{c2}=lateral strength of the compressive column (kN) in JBDPA (2001). The shear strength of the column with a concrete strength less than 13.5N/mm^2 is given by the method in Yasutoshi Yamamoto (2005).

In Table 3, four test parameters; brace type, concrete strength of the reinforced concrete frame, the ratio of height to span of the frame, the value of ${}_aQ_j/{}_cQ_u$ which means the ratio of the shear strength of the upper horizontal joint anchors to the whole lateral strength, of nine specimens including PA1, PA2, PA3, PA4 in Takanori Kawamoto (2010) and Ken Harayama (2012) are shown. The experimental lateral strength ${}_eQ_u$ which is the smaller one of the maximum lateral strengths in the positive and negative loading direction, the calculated lateral strength ${}_cQ_u$, the value of ${}_eQ_u/{}_cQ_u$, the calculated values of ${}_pQ_c$, ${}_aQ_j$, ${}_fQ_j$ and Q_{c2} are also shown in Table 3.

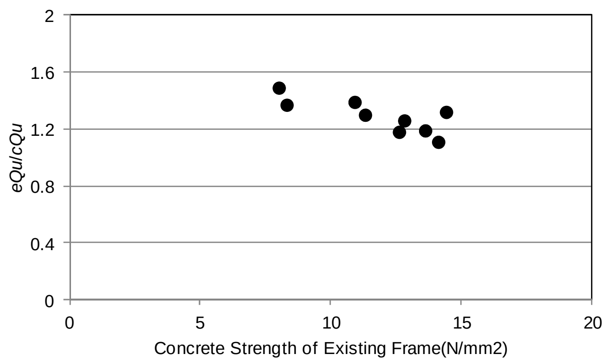
The values of ${}_eQ_u/{}_cQ_u$ of the nine specimens are plotted for each test parameter in Fig. 10. Those values are within a range from 1.11 to 1.49. Therefore, the proposed design equation is considered to be effective in evaluating the lateral strength of the retrofitted frame in the horizontal joint failure mode on the safe side.

Table 3 Comparison of ${}_eQ_u$ and ${}_cQ_u$

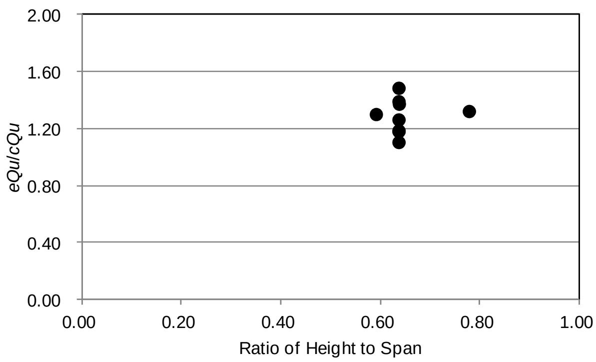
Specimen	Brace Type	Concrete Strength of Existing Frame(N/mm^2)	Ratio of Height to Span	aQj/cQu	eQu (kN)	cQu (kN)	eQu/cQu	pQc (kN)	aQj (kN)	fQj (kN)	$Qc2$ (kN)
PA1	K Type H-Shaped Steel Brace	12.6	0.64	0	354.7	300.3	1.18	190.9	0	48.8	60.6
PA2		13.6	0.64	0.27	505.2	425.7	1.19	206.0	114.8	43.5	61.4
PA3		12.8	0.64	0.27	511.7	405.0	1.26	193.9	110.2	40.1	60.8
PA4		14.1	0.64	0.41	638.7	577.8	1.11	213.6	234.6	67.7	61.9
PA6	Mansard Type H-Shaped Steel Brace	11.3	0.59	0.38	515.7	396.5	1.30	140.9	152.1	41.4	62.1
PA12	K Type Pin-Ended Hollow Tube Brace	10.9	0.64	0.37	551.8	396.6	1.39	157.4	148.5	32.3	58.4
PA16	K Type H-Shaped Steel Brace	8.0	0.64	0.38	476.8	321.0	1.49	115.5	120.8	44.7	40.0
PA17		8.3	0.64	0.32	512.8	373.3	1.37	157.6	120.8	29.3	65.6
PA21		14.4	0.78	0.36	651.2	492.4	1.32	162.0	178.8	88.8	62.8



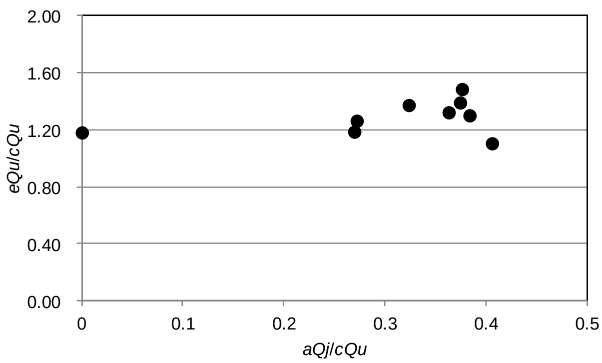
(a) Relationship between ${}_eQ_u/{}_cQ_u$ and brace type



(b) Relationship between ${}_eQ_u/{}_cQ_u$ and concrete strength of existing frame



(c) Relationship between ${}_eQ_u/{}_cQ_u$ and ratio of height to span



(d) Relationship between ${}_eQ_u/{}_cQ_u$ and ${}_aQ_j/{}_cQ_u$

Fig. 10 Relationships between ${}_eQ_u/{}_cQ_u$ and test parameters

6. CONCLUSIONS

The following conclusions were derived from this experimental investigation.

1) The failure mechanism of new five retrofitted specimens was the horizontal joint failure accompanying the shear punching failure at the top of the tensile column.

2) The design equation for the lateral strength of the retrofitted frame in the horizontal joint failure mode was proposed and verified by test results of the nine specimens with the horizontal joint failure. The values of ${}_eQ_u/{}_cQ_u$ of all specimens were within a range from 1.11 to 1.49. The proposed design equation is considered to be effective in evaluating the lateral strength of the retrofitted frame in the horizontal joint failure mode on the safe side.

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