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Seismic performance of the retrofitting method using the pin-ended doubly circular hollow tube brace for existing reinforced concrete frames

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ABSTRACT

A pin-ended doubly circular hollow tube brace consists of an inner tube transferring the axial force and an outer tube restrainingthe buckling of the inner tube, and has a very large energy absorbing ability after yielding. In this study, the seismic performance of a retrofitting method installing thisbrace inside of the existing reinforced concrete frames was experimentally investigated.Three specimen frameswere loaded laterally. One is an un-retrofitted reinforced concrete frame. Two are reinforced concrete frames retrofitted by the brace. It was found that the maximum lateral strength,the deformation capacity and the equivalent damping factor due to hysteresis loop of the retrofitted frame weremuch larger than that of the un-retrofitted frame.

1. INTRODUCTION

In the seismic retrofit of existing reinforced concrete buildings, seismic retrofitting methods of the strength resistance type have been widely used.However, in recent years, seismic retrofitting methods of the response control type using the energy absorbing devices have attracted interest.A pin-ended doubly circular hollow tube brace is one of such devices. The doubly circular hollow tubehasa feature that the axial compressive strength is the same as the axial tensile strength. The reason for this is that the outer hollow tube restrains the buckling of the inner hollow tube. The large energy absorptionof the inner hollow tube can be expected after yielding.Kazuaki Miyagawa (2003) has proposedaseismic retrofitting method installing the peripheral H-shapedsteel

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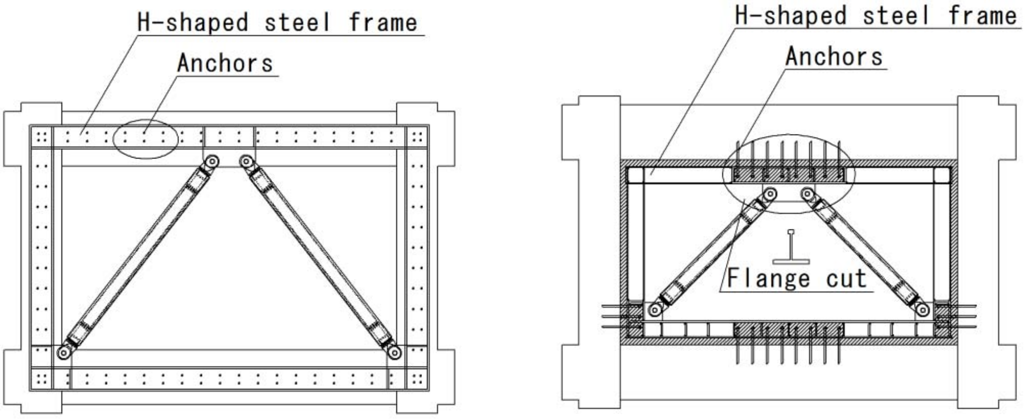
frame havingthe pin-ended doubly circular hollow tube bracesoutside of existing reinforced concrete frames. However, this installation method requires a great many joint anchors as shown in Fig. 1 (a).

The authors have proposed a method installing the H-shaped steel frame having steel braces inside of existing reinforced concrete framesusing relatively a few joint anchors, as shown in Fig. 1 (b), in Takanori Kawamoto (2010) and Ken Harayama (2012). This method can reduce noises, dusts and vibrations during the installation construction of joint anchors. In this paper, loading tests of a seismic retrofitting method installing the H-shaped steel frame havingpin-ended doubly circular hollow tube braces inside of existing reinforced concrete frames by the authors' proposed method are reported in detail.



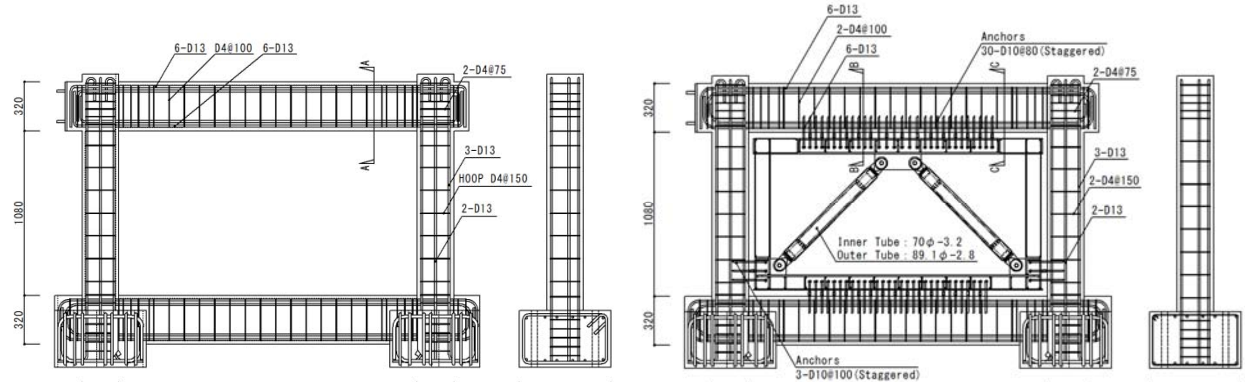
(a)External installationmethod(b)Inner installation method
in Kazuaki Miyagawa (2003) proposed by authors

Fig. 1 A seismic retrofitting method
using the pin-ended doubly circular hollow tube brace

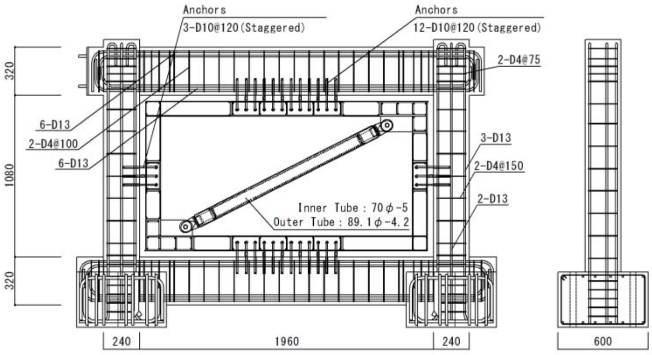
2. SPECIMENS

Three specimens are shown in Fig. 2 and are listed in Table 1.All thespecimensare1/2.5 scaledframe with one bay and one story. Specimen RC1is an un-retrofitted reinforced concrete frame. Specimens RC2 and OPA2 are reinforced concrete frames retrofitted by the pin-ended doubly circular hollow tube braces.RC1 is almost identical to the reinforced concrete frames of RC2 and OPA2. The failuremechanism of RC1 is designed to be the shear failure of the columns.The arrangement of the braces is K type forRC2 and shed type for OPA2. The low yield point steel was used for the inner tube of the K type braceofRC2, and the ordinary strength steel was used for the inner tube of the shed type braceof OPA2. In the installation of the steel frame ofRC2, thirty D10 anchors were used at each horizontal joint andthree D10 anchors were used at each vertical joint.In the installation of the steel frame of OPA2, twelve D10 anchors were used at each horizontal joint andthree D10anchors were used at each vertical joint.

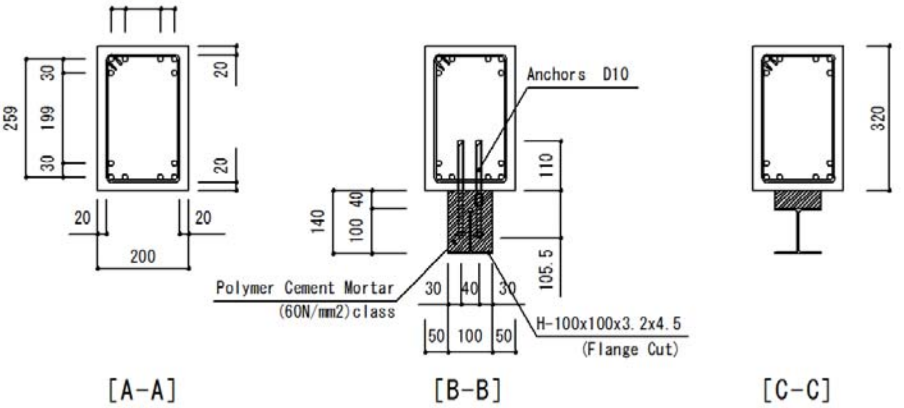
The concrete strength of the reinforced concrete frames is 21.9 - 28.4 N/mm^2. Mechanical properties of steel materials used for specimens are listed in Table 2.



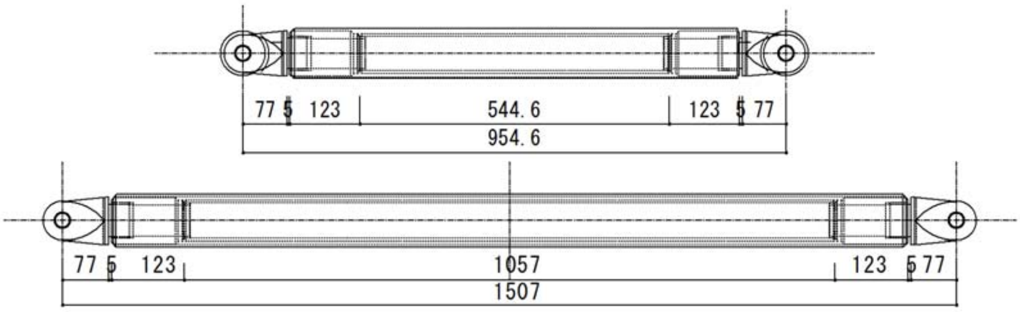
(a) RC1 (b) RC2



(c) OPA2



(d) Sections



(e) Pin-ended doubly circular hollow tube brace

Fig.2 Details of specimens

Table 1 List of specimens

Specimens	Compressive Strength of Concrete (N/mm^2)	Column	Beam	Steel Frame	Steel Brace
RC1	21.9	Section: 240mm×240mm Main Reinforcing Bars: 8-D13 Hoop: D4@150	Section: 200mm×320mm Main Reinforcing Bars: 6-D13(upper) + 6-D13(lower) Stirrup: D4@100	—	—
RC2	23.0			H-100×100×3.2×4.5	Inner Tube: 70 ϕ -3.2 Outer Tube: 89.1 ϕ -2.8
OPA2	28.4				Inner Tube: 70φ-5 Outer Tube: 89.1φ-4.2

Table 2 Mechanical properties of steel materials

(a) RC1 and RC2

Materials Usedfor Specimens		Yielding Strength (N/mm^2)	Tension Strength (N/mm^2)
D13(SD295)		368	531
D10(SD295)		380	520
D4(SD295)		368	510
H-shaped Steel	Web t=3.2(SS400)	360	468
	Flange t=4.5(SS400)	343	474
Inner Tube of Brace	t=3.2(LY-225)	245	350

(b) OPA2

Materials Usedfor Specimens		Yielding Strength (N/mm^2)	Tension Strength (N/mm^2)
D13(SD295)		381	528
D10(SD295)		373	511
D4(SD295)		383	559
H-shaped Steel	Web t=3.2(SS400)	357	435
	Flange t=4.5(SS400)	289	418
Inner Tube of Brace	t=5(STKM13A)	409	506

3. TEST PROCEDURE

Test setup is shown in Fig. 3. Loading schedule is shown in Fig. 4.The long term axial force was applied to the columns of the specimen 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.

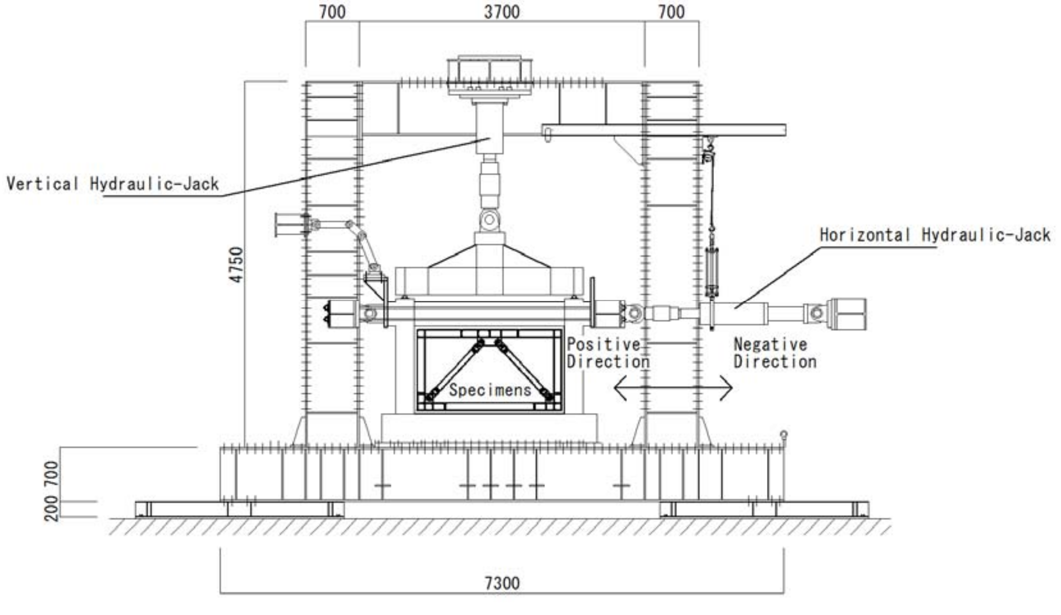


Fig. 3Test setup

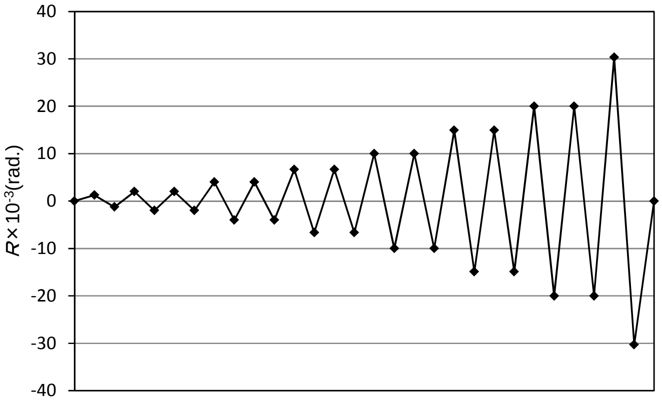


Fig. 4Loadingschedule

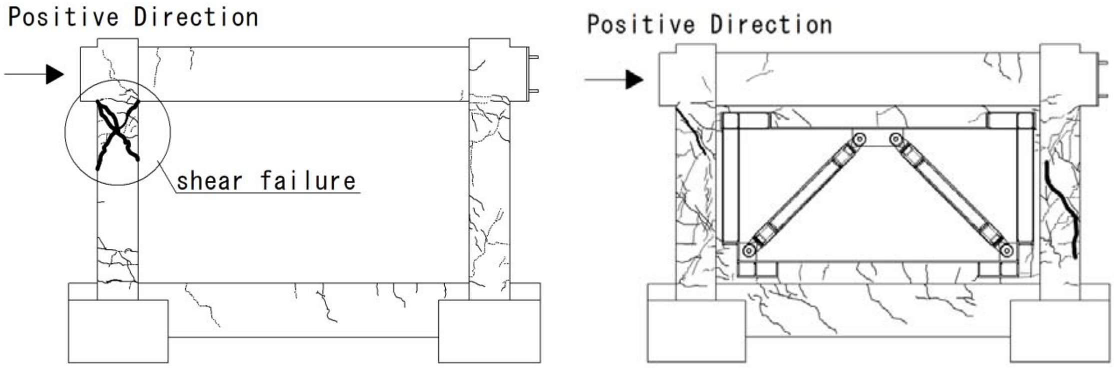
4. TEST RESULTS

Crack patterns of the specimens at *R*=0.015 rad.are shown in Fig. 5. Shear force *Q*–story drift angle *R* relationships are shown in Fig. 6. Shear force is the value measuredby the load cell attached to the horizontal hydraulic jack.

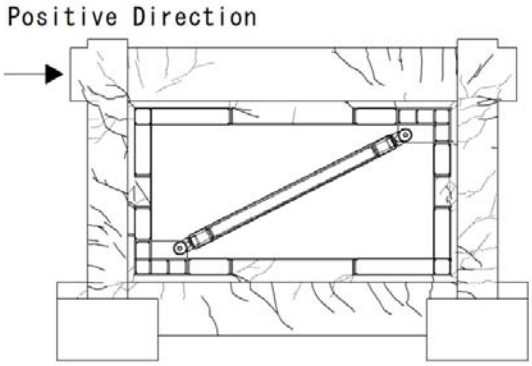
In failure process of RC1, the flexural cracksoccurred at the bottom ofthe tensile column in theloading cycle of *R*=0.00125rad.The flexural cracksoccurred at the top of the tensile column in theloading cycle of*R*=0.002rad.The shear cracks occurred at the top of the tensile column in theloading cycle of *R*=0.0067rad.The maximum lateral strength in the negative loading direction was observed in the loading cycle of *R*=0.01rad. The maximum lateral strength in the positive loading direction was observed in the loading cycle of *R*=0.015rad.The deformation capacity of RC1, in which the stable cyclic *Q* – *R* relationship could be expected, was approximately *R*=0.01 rad. The failure mechanism of RC1 was the shear failure of the columns.

In specimen RC2, theflexural cracks occurred at the bottom of the tensile column in theloading cycle of*R*=0.002rad. The shear cracks occurred at the top and center of the tensile column in theloading cycle of *R*=0.004rad.The yielding of the tensile and compressive bracesoccurred in the loading cycle of *R*=0.0067rad. The yielding of hoops in the columnswas observed in the loading cycle of *R*=0.01rad. The maximum lateral strength in the negative loading direction wasobserved in the loading cycle of *R*=0.015rad.In the same loading cycle, a local buckling occurred in thevertical steel frame at neighborhood of the tensile column. The maximum lateral strength in the positive loading direction was observed in the loading cycle of *R*=0.02rad.The deformation capacity of RC2was approximately *R*=0.015 rad.

In specimen OPA2, the shear cracks occurred at the center of the tensile column in the loading cycle of *R*=0.01rad.In the same loading cycle, the yielding of the brace occurred. The maximum lateral strengthsin the positive and negative loading direction wereobserved in the loading cycle of *R*=0.015rad.The deformation capacity of OPA2 was approximately *R*=0.015 rad.



(a)RC1　(b)RC2



(c) OPA2

Fig. 5 Crack patterns at R=0.015 rad.

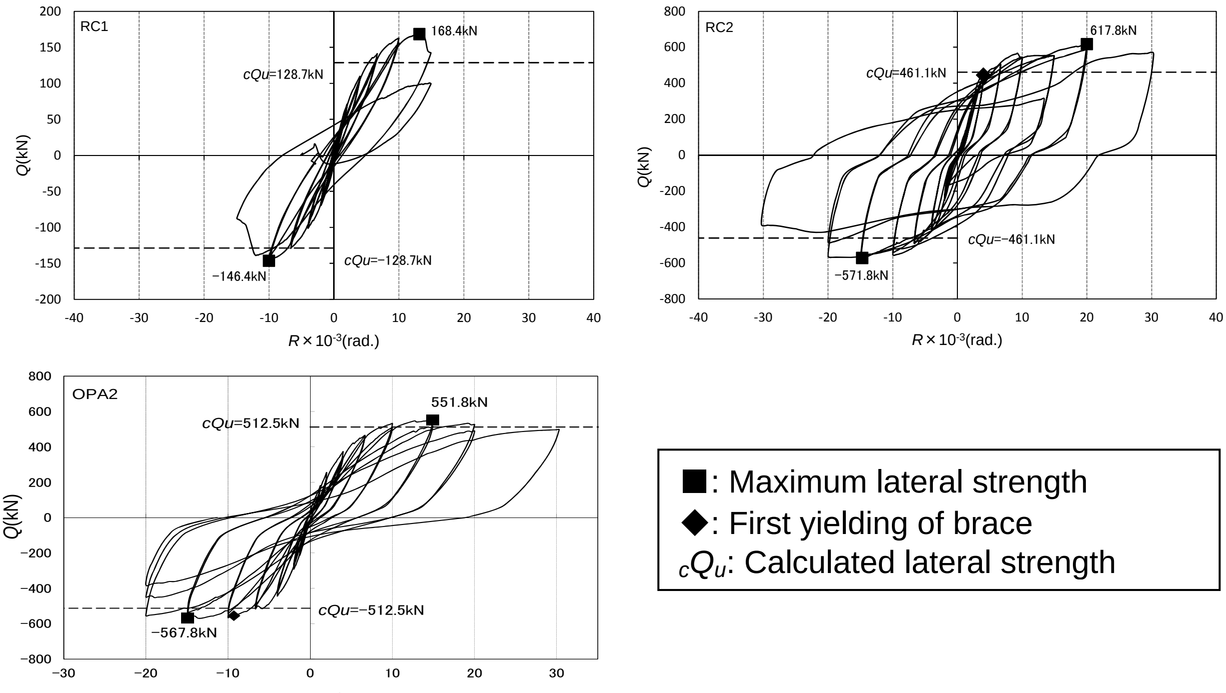


Fig. 6Shear force Q–story drift angle R relationships

5. EVALUATION OF LATERAL STRENGTH

Comparisons of the experimental lateral strength $_eQ_u$ and the calculated lateral strength $_cQ_u$ of the specimens are shown in Table 3. $_eQ_u$ is the smaller one of the maximum lateral strengths in the positive and negative loading direction. $_cQ_u$ of RC1, RC2 and OPA2 was obtained from Eq. (1), Eq. (3) and Eq. (2), respectively. $_cQ_u$ in Eq. (1) is the lateral strength of the reinforced concrete frame. $_cQ_u$ in Eq. (2) and Eq. (3) is the lateral strength of the frame retrofitted by the brace in the brace yielding mode.The details of Eq. (1) and Eq. (2) are described in JBDPA (2001). In Eq. (2), $_sQ_y$ is the lateral strength of the brace determined by the yield axial strength. However, in Eq. (3) evaluating the lateral strength of RC2, the lateral strength of the brace determined by the tensile axial strength $_sQ_u$ is used instead of $_sQ_y$. The reason for this is that the low yield point steel used as the inner tube of the brace has a feature that the increase of the strength becomes large to the cyclic loading after yielding. The values of $_eQ_u/_cQ_u$ in Table 3 arewithin a range from 1.08 to 1.24. The lateral strength of the frame retrofitted by the brace can be evaluated by Eq. (2) and Eq. (3).

In RC2 and OPA2, the lateral strength of the retrofitted frame determined by the horizontal joint failure $_cQ_j$, which is obtained from the method described in Takanori Kawamoto (2010) and Ken Harayama (2012), is larger than the lateral strength of the retrofitted frame in the brace yielding mode as shown in Table 3. In the other words, the horizontal joints of RC2 and OPA2 have a sufficiently strength that the braces yield and absorb the energy.

$$_cQ_u = Q_{c1} + Q_{c2} \tag{1}$$

where $_cQ_u$=the lateral strength of reinforced concrete frame; Q_{c1}=lateral strength of the tensile column; Q_{c2}=lateral strength of the compressive column.

$$_cQ_u = {}_sQ_y + Q_{c1} + Q_{c2} \tag{2}$$

where $_cQ_u$=lateral strength of the retrofitted frame in the brace yielding mode; $_sQ_y$=lateral strength of the brace determined by the yieldaxial strength.

$$_cQ_u = {}_sQ_u + Q_{c1} + Q_{c2} \tag{3}$$

where $_cQ_u$=lateral strength of the retrofitted frame in the brace yielding mode; $_sQ_u$=lateral strength of the brace determined by the tensileaxial strength.

Table 3 Comparison of experimental lateral strength and calculated lateral strength

Specimens	$_eQ_u$	$_eQ_u/_cQ_u$	Q_{c1}	Q_{c2}	$_sQ_y$	$_sQ_u$	$_cQ_u$	$_cQ_j$
RC1	146.4	1.14	61.2	67.5	—	—	128.7	—
RC2	571.8	1.24	62.0	68.3	—	330.9	461.1	864.8
OPA2	551.8	1.08	66.5	73.3	372.7	—	512.5	566.0

6.COMPARISON OFLATERAL STRENGTHANDEQUIVALENT DAMPING FACTOR DUE TO THE HYSTERESIS LOOP

The envelope curves of the Q–R relationships in the positive loading direction of the three specimens are shown in Fig. 7. The maximum lateral strength in the positive loading direction of RC2 is about 1.1 times that of OPA2, and about 3.5 times that of RC1.

The equivalent damping factor due to the hysteresisloopheq–R relationshipsof the three specimens are shown in Fig. 8. The equivalent damping factorof RC2 at R=0.01 rad.is about 1.4 times that of OPA2, and about 2.5 times that of RC1. Thereason for this is that the inner tube of the brace of RC2 is the low yield point steel.

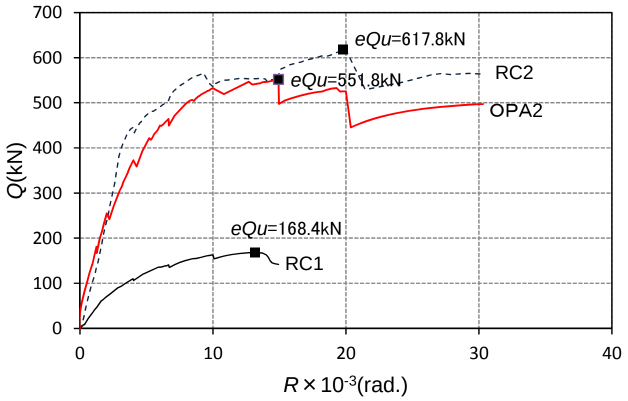


Fig. 7Envelope curves of$Q$– Rrelationships

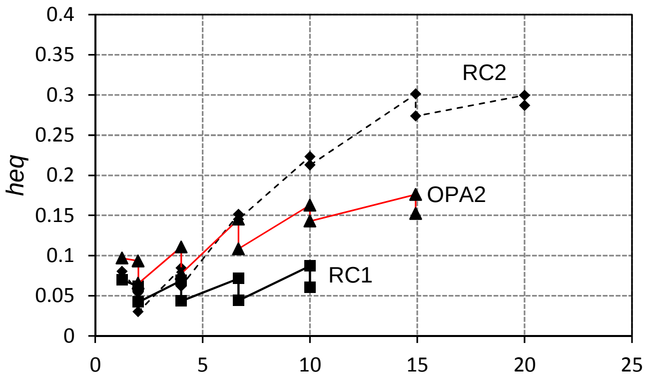


Fig. 8heq–Rrelationships

7. CONCLUSIONS

The following conclusions were derived from this experimental investigation.

1) The failure mechanism of an un-retrofitted reinforced concrete frame was the shear failure of the columns. The deformation capacity of the un-retrofittedframe wasapproximately$R$=0.01 rad. The lateral strength of the un-retrofitted frame can be evaluated by Eq. (1).

2) The maximum lateral strength in the positive loading direction of the specimen frame

retrofitted by the K type braceusing the low yield strength steel was1.1 times that ofthe specimenframe retrofitted by the shed type brace, and about 3.5 times that of the un-retrofitted frame. The lateral strength of the specimen frames can be evaluated by Eq. (2) and Eq. (3).The deformation capacity of the specimen frame retrofitted by the K typebrace or the shed type brace was approximately$R$=0.015 rad.

3) The equivalent damping factordue to the hysteresis loop of the specimen frame retrofitted by the K type brace using the low yield strength steel at R=0.01 rad.was about 1.4 times that of the specimen frame retrofitted by the shed type brace, and about 2.5 times that of the un-retrofitted frame.

4) The lateral strength of the retrofitted frame determined by the horizontal joint failure was larger than the lateral strength of the retrofitted frame in the brace yielding mode. Therefore, the installation method proposed by the authors is effective for theseismic retrofitting methods of the response control type.

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