Loading tests of seismic retrofitting method by outer reinforced concrete frames with framed steel brace

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ABSTRACT

The authors have proposed a seismic retrofitting method installing a new reinforced concrete frame with framed steel brace outside the existing reinforced concrete frame. In the actual retrofitting design, the size of the new outer reinforced concrete frame is determined depending on not only the size of the existing frame but also the presence or absence of the bay window. In this paper, loading tests conducted to understand the effects of the size of the outer frame on the seismic performance of the retrofitted frame are reported. Two 1/2.5 scaled one bay one story specimen frames were laterally loaded in the test.

1. INTRODUCTION

In the seismic retrofit of existing reinforced concrete buildings in Japan, seismic retrofitting methods installing framed steel braces inside the existing reinforced concrete frames have been generally used. However, this method has a critical problem that the building cannot be used during the retrofitting construction. Therefore, external type seismic retrofitting methods which allow the building to remain in use during the construction are preferable. The authors have proposed a seismic retrofitting method installing a new outer reinforced concrete frame with the framed steel brace outside the existing reinforced concrete frame in Ken Harayama (2011), as shown in Fig. 1.

In the actual retrofitting design of this external type method, the size of the new outer reinforced concrete frame is determined depending on not only the size of the existing frame or the new brace but also the presence or absence of the bay window. However, the effects of the size of the outer frame on the seismic performance of the retrofitted frame have not been sufficiently investigated. In this paper, the test results of two 1/2.5 scaled one bay one story specimen frames retrofitted by the outer frame with a different size are described in detail.
2. SPECIMENS

The details of two specimens are shown in Table 1 and Fig. 2. The both specimens are 1/2.5 scaled one bay one story frames, and have transverse beams and a slab like the frame of actual buildings.

In SPA5, the section of the existing columns is 240 mm x 240 mm and has main re-bars of 8-D13, and the section of the existing beams is 200 mm x 320 mm and has main re-bars of 12-D13. The section of the outer columns is 130 mm x 240 mm and has main re-bars of 6-D13, and the section of the outer beams is 120 mm x 320 mm and has main re-bars of 8-D13. In SPA6, the section of the existing columns is 160 mm x 240 mm and has main re-bars of 6-9 φ. The section of the existing upper beams is 160 mm x 240 mm and has main re-bars of 10-13 φ, and the section of the existing lower beams is 160 mm x 320 mm and has main re-bars of 10-13 φ. The section of the outer columns is 244 mm x 240 mm and has main re-bars of 9-D13. The section of the outer upper beams is 244 mm x 240 mm and has main re-bars of 8-D13, and the section of the outer lower beams is 244 mm x 320 mm and has main re-bars of 8-D13. The width of the existing frame of SPA6 is smaller than that of the existing frame of SPA5, however, the width of the outer frame of SPA6 is larger than that of the outer frame of SPA5. The existing frame of SPA5 was designed to behave in the shear failure mode of the columns. On the other hand, the existing frame of SPA6 was designed to behave in the flexural yielding mode of the columns.
Pin-ended hollow tube braces with 70 mm diameter and 5 mm thickness are installed in the outer frame in the two specimens.

As shown in Fig. 3, SPA5 has joint anchors between the outer beam and the existing beam so that the two beams move together. SPA6 has joint anchors not only between the beams, but also between the columns for the transferring of the axial force of the columns.

The compressive strengths of cement materials used for specimens are shown in Table 2, and the strengths of the steel materials are shown in Table 3.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Column Section</th>
<th>Main Re-Bars</th>
<th>Hoop Section</th>
<th>Main Re-Bars</th>
<th>Stirrup Section</th>
<th>Main Re-Bars</th>
<th>Stirrup</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPA5</td>
<td>Existing Frame</td>
<td>240×240</td>
<td>8-D13</td>
<td>200×320</td>
<td>6-D13</td>
<td>200×320</td>
<td>6-D13</td>
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<tr>
<td></td>
<td>Outer Frame</td>
<td>130×240</td>
<td>6-D13</td>
<td>120×320</td>
<td>4-D13</td>
<td>120×320</td>
<td>4-D13</td>
</tr>
<tr>
<td>SPA6</td>
<td>Existing Frame</td>
<td>160×240</td>
<td>6-9φ</td>
<td>160×240</td>
<td>5-13φ</td>
<td>160×320</td>
<td>5-13φ</td>
</tr>
<tr>
<td></td>
<td>Outer Frame</td>
<td>244×240</td>
<td>9-D13</td>
<td>244×240</td>
<td>4-D13</td>
<td>244×320</td>
<td>4-D13</td>
</tr>
</tbody>
</table>

Table 1 Details of specimens (continued)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Steel Brace</th>
<th>Steel Frame</th>
<th>Horizontal Joint Anchors of Outer Frame</th>
<th>Anchors between outer frame and existing frame</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPA5</td>
<td>Pin-ended Hollow Tube 70 φ-5</td>
<td>12-D10</td>
<td>Upper Beam: 34-D10 \ Lower Beam: 32-D10</td>
<td></td>
</tr>
<tr>
<td>SPA6</td>
<td></td>
<td>10-D10</td>
<td>Upper Beam: 62-D10 \ Each Column: 51-D10 \ Lower Beam: 60-D10</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 2 Dimensions and details of specimens
Fig. 2 Dimensions and details of specimens (continued)

Fig. 3 Anchors between existing frame and outer frame
Table 2 Compressive strengths of cement materials

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive Strength of Concrete (N/㎟)</th>
<th>Infilled Mortar (N/㎟)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Upper Beam and Column</td>
<td>Lower Beam</td>
</tr>
<tr>
<td>SPA5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Existing Frame</td>
<td>11.1</td>
<td>28.7</td>
</tr>
<tr>
<td>Outer Frame</td>
<td>24.0</td>
<td></td>
</tr>
<tr>
<td>SPA6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Existing Frame</td>
<td>15.5</td>
<td>31.7</td>
</tr>
<tr>
<td>Outer Frame</td>
<td>27.5</td>
<td></td>
</tr>
</tbody>
</table>

Table 3 Strengths of steel materials

(a) SPA5

<table>
<thead>
<tr>
<th>Steel Materials</th>
<th>Yield Strength (N/㎟)</th>
<th>Tensile Strength (N/㎟)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D13(SD345)</td>
<td>409</td>
<td>531</td>
</tr>
<tr>
<td>D13(SD295)</td>
<td>381</td>
<td>491</td>
</tr>
<tr>
<td>D10(SD295)</td>
<td>373</td>
<td>491</td>
</tr>
<tr>
<td>D4(SD295)</td>
<td>363</td>
<td>559</td>
</tr>
<tr>
<td>H-Shaped Steel Frame</td>
<td>357</td>
<td>435</td>
</tr>
<tr>
<td>Web t=3.2(SS400)</td>
<td>289</td>
<td>418</td>
</tr>
<tr>
<td>Flange t=4.5(SS400)</td>
<td>409</td>
<td>506</td>
</tr>
<tr>
<td>Steel Brace</td>
<td>70 φ−5(STKM13A)</td>
<td>409</td>
</tr>
</tbody>
</table>

(b) SPA6

<table>
<thead>
<tr>
<th>Steel Materials</th>
<th>Yield Strength (N/㎟)</th>
<th>Tensile Strength (N/㎟)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D13(SD345)</td>
<td>368</td>
<td>531</td>
</tr>
<tr>
<td>9 φ (SR235)</td>
<td>343</td>
<td>447</td>
</tr>
<tr>
<td>13 φ (SR235)</td>
<td>344</td>
<td>454</td>
</tr>
<tr>
<td>D10(SD295)</td>
<td>380</td>
<td>520</td>
</tr>
<tr>
<td>D4(SD295)</td>
<td>368</td>
<td>510</td>
</tr>
<tr>
<td>H-Shaped Steel Frame</td>
<td>360</td>
<td>468</td>
</tr>
<tr>
<td>Web t=3.2(SS400)</td>
<td>343</td>
<td>474</td>
</tr>
<tr>
<td>Flange t=4.5(SS400)</td>
<td>343</td>
<td>457</td>
</tr>
<tr>
<td>Steel Brace</td>
<td>70 φ−5(STKM13A)</td>
<td>343</td>
</tr>
</tbody>
</table>

3. TEST PROCEDURE

A test setup is shown in Fig. 4. Loading schedule is shown in Fig. 5. The long term axial force was applied to the columns of the existing frame and maintained at 170 kN for SPA5 and at 115 kN for SPA6. The lateral force was applied to the upper beam ends of the existing frame. The loading was controlled by the story drift angle \( R \), where \( R = \frac{\delta}{h} \); \( \delta \) was the lateral displacement of the upper beam, \( h \) was the height of the existing frame.
4. TEST RESULTS

Crack patterns of the specimens after the test are shown in Fig. 6. Shear force $Q$ - story drift angle $R$ relationships are shown in Fig. 7. Shear force is the value measured by the load cell attached to the horizontal hydraulic jack.

In failure process of SPA5, the flexural cracks occurred at the bottom of the tensile outer column in the loading cycle of $R=0.002$ rad. The flexural cracks occurred at the top of the tensile outer column in the loading cycle of $R=0.004$ rad., and in the same loading cycle, the shear cracks occurred at the existing columns. In the loading cycle of $R=0.01$ rad., the separation cracks occurred between the outer column and the existing column, and the maximum lateral strengths in the positive and negative were observed. Finally, the shear failure occurred at the existing columns, and the horizontal joint failure occurred in the outer frame accompanying the punching shear failure at the top of the
tensile outer column. After that, the deterioration in the lateral strength was observed. The deformation capacity of SPA5, in which the stable cyclic $Q - R$ relationship could be expected, was approximately $R=0.01$ rad.

In failure process of SPA6, the flexural cracks and the punching shear cracks occurred at the outer column in the loading cycle of $R=0.002$ rad. In the loading cycle of $R=0.0067$ rad., the cracks occurred at the joint mortar around the foot of the compressive steel brace, however, significant damage did not occurred. In the loading cycle of $R=0.015$ rad., yielding of the tensile steel brace occurred, but the punching shear failure did not occurred in the columns of the outer frame, and the deterioration in the lateral strength was not observed. The loading was quit after the loading cycle of $R=0.015$ rad. because of a large deformation occurring in the out-of-plane direction. Therefore, the deformation capacity is considered to be larger than $R=0.015$ rad.

Fig. 6 Crack patterns of specimens after the test
5. EVALUATION OF LATERAL STRENGTH

In the proposed retrofitting method, the lateral strength of the retrofitted frame can be evaluated by the superposition of that of the outer reinforced concrete frame with the framed steel brace and that of the existing reinforced concrete frame. In case that the outer frame with the framed steel brace behaves in the steel brace yielding mode, the lateral strength of the retrofitted frame is expressed by Eq. (1). In case that the outer frame with the framed steel brace behaves in the horizontal joint failure mode accompanying the punching shear failure of the tensile outer column, the lateral strength of the retrofitted frame is expressed by Eq. (2). The lateral resistance mechanism of the outer frame in Eq. (2) is shown in Fig. 8.

The experimental lateral strength $\sigma Q_u$ and the calculated lateral strength $c Q_u$ of the specimens are shown in Table 4, where, $\sigma Q_u$ is the smaller one of the maximum lateral strengths in the positive and negative loading direction. The value of $c Q_u/\sigma Q_u$ of SPA5 is 1.07, where $c Q_u$ was given by Eq. (2). The value of $c Q_u/\sigma Q_u$ of SPA6 is 0.96, where $c Q_u$ was given by Eq. (1). The calculated lateral strength is well corresponding with the experimental lateral strength in the both of specimens.

$$\sigma Q_u = \Sigma Q_c + \sigma Q_y + Q_{s1} + Q_{s2}$$  \hspace{1cm} (1)

where $c Q_u$=lateral strength of the retrofitted frame in the steel brace yielding mode; $\Sigma Q_c$=lateral strength of the existing frame; $\sigma Q_y$=lateral strength of the steel brace; $Q_{s1}$=...
lateral strength of the tensile column in the outer frame; \( Q_{c2} \) = lateral strength of compressive column in the outer frame.

\[
\ddot{Q}_u = \Sigma Q_c + pQ_c + aQ_j + fQ_j + Q_{c2} \quad (2)
\]

where \( \ddot{Q}_u \) = lateral strength of the retrofitted frame in the horizontal joint failure mode; \( \Sigma Q_c \) = lateral strength of the existing frame; \( pQ_c \) = punching shear strength of the tensile column in the outer frame in JBDPA (2001); \( aQ_j \) = shear strength of anchors at the upper horizontal joint in JBDPA (2001); \( fQ_j \) = shear resisting force by the friction in Takanori Kawamoto (2010) and Ken Harayama (2012); \( Q_{c2} \) = lateral strength of the compression column in the outer frame.

![Fig. 8 Lateral resistance mechanism of the outer frame in Eq. (2)](image)

### Table 4 Comparison of experimental lateral strength \( \ddot{Q}_u \) and calculated lateral strength \( cQ_u \)

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Failure mode</th>
<th>( \ddot{Q}_u ) [kN]</th>
<th>( cQ_u ) [kN]</th>
<th>( \ddot{Q}_u / cQ_u )</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPA5</td>
<td>Horizontal joint failure mode</td>
<td>538.3</td>
<td>502.0</td>
<td>1.07</td>
</tr>
<tr>
<td>SPA6</td>
<td>Steel brace yielding mode</td>
<td>481.3</td>
<td>498.8</td>
<td>0.96</td>
</tr>
</tbody>
</table>

### 6. CONCLUSIONS

The following conclusions can be derived from this experimental study.

1) In SPA5 specimen with the outer frame of a relatively small section, the horizontal joint failure occurred in the outer frame accompanying the punching shear failure at the top of the tensile column of the outer frame. The yielding of the steel brace installed in the outer frame did not occur. The lateral strength of the specimen can be evaluated by Eq. (2). The deformation capacity of the specimen was approximately 0.01 (rad.) in the story drift angle.

2) On the other hand, in SPA6 specimen with the outer frame of a relatively large section, the horizontal joint failure or the punching shear failure did not occurred in the outer
The yielding of the steel brace installed in the outer frame occurred. The lateral strength of the specimen can be evaluated by Eq. (1). The deformation capacity of the specimen was considered to be larger than 0.015 (rad.) in the story drift angle.

REFERENCES

The Japan Building Disaster Prevention Association (2001), Guideline of Seismic Retrofit of Existing Reinforced Concrete Buildings (In Japanese)

