SEISMIC PERFORMANCE OF ACTUAL RC MEMBERS RETROFITTED WITH CFRP AND EPOXY RESIN

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ABSTRACT

Experiments were performed in order to clarify the effects of standard retrofitting methods, utilizing three RC beams taken from an existing building constructed approximately forty years ago. In the structural specifications, plain round bars of φ 19 were arranged for the longitudinal bars. Plain round bars of φ 9 with a spacing of 300mm were used for the stirrups. The specified concrete strength was 18MPa. From the bar arrangements shown in the specification, it was predicted that the failure mode of the original beam would be a flexural failure type. Two of the three specimens were wrapped with CFRP sheet. In one of the retrofitted beams, epoxy resin was injected at the locations of the main bars to prevent bond slip failure of the plain round bars. The beams were subjected to reversal loading under anti-symmetric moment with displacement control. In the original beam, diagonal shear cracks occurred during the early stages of loading, indicating extremely brittle restoring force characteristics. The observed maximum strength was approximately the same as the calculated strength of shear failure based on the actual bar arrangements. In the retrofitted beams, flexural cracks were observed at both ends of the CFRP sheet, and the crack width increased as the drift angle of the beams increased. No shear cracks were observed in the original beams when the CFRP sheet was removed after seismic loadings. The maximum strength exceeded the calculated strength of the flexural failure. The shear strength did not decrease after the maximum strength was reached, indicating very ductile restoring force characteristics. Epoxy resin was believed to contribute to the increase of the energy dissipations concerning the hysteresis loops.

KEYWORDS

CFRP sheet, epoxy resin, actual RC beams, strengthening, seismic performance

INTRODUCTION

In Japan, seismic performance of existing buildings has typically been evaluated by seismic diagnosis based on their structural drafts. In many existing buildings however, differences between the strength of materials, the arrangement of reinforcement between actual members, and the structural drafts have been found. Deteriorations over long time periods, uncertainty of construction, and scale effects etc. were not considered in the seismic diagnosis of both the actual members and the specimens manufactured in the laboratory. Therefore, it is very difficult to evaluate the accurate seismic performance of an existing building. From this point of view, in the field of civil engineering performance examinations have been carried out using RC members of an old RC railway bridge, and the applicability of formulas have been evaluated (Yokozawa et al. 2005; Niitani et al. 2005; Hattori et al. 2006). However, in the field of building engineering, there are very few experimental tests concerning actual RC members of old existing buildings, although full scale experiments have been done using existing buildings (Osawa et al.1968; Matsushima 1970). The research on the seismic performance of the RC member obtained from the old building was very precious (Aoyama et al.1983). In this paper, the seismic performance of the actual RC members retrofitted by CFRP sheet and epoxy resin was investigated.

EXPERIMENTAL PROCEDURE

Existing Building
The target building was a three story reinforced concrete building, constructed in 1967 and used as an elementary school. This building was judged to have low seismic performance in the seismic diagnosis. The subjected beams located at the end of the slab were removed to reduce story weight and the necessity for repair in the retrofitting scheme. The school building is shown in Figure 1. Elevations of the building are shown in Figure 2.

**Test Specimen**

The actual beams were taken out of the roof slab without any damage using a wire saw. The hoisting of a beam after cutoff is shown in Figure 3. According to the structural draft, the main reinforcing was round bar 4-19φ (SR24) and the transverse reinforcement was round bar 2-9φ (SR24) @300. The sections were 250mm×450mm. The reinforced concrete stubs were manufactured at both ends of the existing beams. All beams were designed to have a common shear span length of 1350 mm and shear span ratio of 1.5. Figure 4 shows the details of the test specimens (reinforcing and dimensions). Old concrete at both ends of the obtained beams was removed to expose the longitudinal reinforcements and to allow welding of a steel plate to ensure anchorage before manufacturing the concrete stubs.

Two of the three test specimens were wrapped by the CFRP sheets to prevent a brittle shear failure. The number of fiber sheets used was based on the Draft of Architectural Institute of Japan (2001). In one of the retrofitted test specimens, epoxy resin was injected with a spring capsule to prevent bond slip failure of the round reinforcing bars. It was reported that the maximum flexural strength did not reach the calculated flexural strength due to bond slippage when the plain round bars were used (Araki 2011). The retrofitting process is shown in Figure 5. The test specimens are summarized in Table 1. Compressive strength of the concrete was estimated by the material tests using a concrete core obtained from the existing concrete near the beams. The average concrete strength of twelve concrete cores was 18.1N/mm². The yield strengths of 19φ and 9φ were 328 N/mm² and 281 N/mm² respectively.
The yield strength of $1 \varphi$ and $9 \varphi$ was 328 N/mm$^2$ and 281 N/mm$^2$ respectively. The reinforcement ratio of the main bars and the stirrup was 0.56% and 0.17% respectively. The material properties of the carbon fiber sheet were referenced from the declared values in the brochure. The thickness and tensile strength of the carbon fiber sheet were 0.11mm and 3400 N/mm$^2$ respectively. Low viscous epoxy resin of 100~200mPa.s was injected at a very low pressure of 0.06N/mm$^2$. Unlike the usual method in which epoxy resin is injected at the concrete surface, in this test, epoxy resin was injected at the position of the longitudinal reinforcing bars in the concrete to prevent bond slippage. Tensile strength and compressive strength were 58.8 N/mm$^2$ and 83.3N/mm$^2$ respectively. Bond strength was more than 6 N/mm$^2$ according to the brochure. The newly constructed concrete stubs were reinforced with the deformed bars of D19 and D10. The concrete strength of the stub was 23.4 N/mm$^2$. The estimated failure modes of the all test specimens based on the structural drafts were flexural failure types.

Loading and Measurement

The test specimens were subjected to cyclic loadings by using a universal testing machine with a maximum vertical load of 100 ton and the ability to apply displacement control under anti-symmetric moment, as shown in Figure 6. The relative shear displacement between the stubs and the local displacements of the beam were measured using electric displacement transducers instrumented on the rear side of the beam as shown in Figure 7. The drift angle was calculated by dividing the relative displacement by the shear span 1350mm. The loading cycles were as follows, $R = 1/400$ rad., $1/200$ rad., $1/100$ rad., $1/66$ rad., $R=1/50$rad., $R=1/33$ rad., $R=1/25$ rad., $R=1/20$ rad. and $R=1/10$ rad.

TEST RESULTS

Failure Mode

Figure 8 shows the failure mode of each specimen. A large difference in failure mechanism between the original beam E15 and the retrofitted beams E15-C0, E15-C1 was observed. For the existing beam E15, flexural cracks occurred at the boundary surface between both ends of the beam and stubs, and shear cracks occurred in the mid area of the beam at drift angle $R=1/400$rad. With the increase in deformation, the width of the shear cracks increased when displacement increased. The final failure mechanism was due to shear failure, with large X
shaped diagonal cracks observed, as shown in Figure 8-(a). For the retrofitted existing beams E15-C0, the flexural cracks were observed at both ends of the beam at drift angle R=1/400rad. Fiber sheet at the end of the beam was broken at drift angle R=1/33rad. It was not possible to observe cracks in the mid area of the beam due to the fiber sheets when the load was applied. After loading, the fiber sheets were removed to inspect the appearance of the mid span of the beam. Crushing of the concrete cover was observed at the boundary between the beam and the stubs. Flexural and shear cracks in the mid area of the beam body were not observed, as shown in Figure 8-(b). For the retrofitted Specimen E15-C1 with injected epoxy resin, no cracks were observed except for the flexural cracks between the stubs and beam. Although slight spallings of the concrete cover were observed, crushing of the concrete and fracture of the fiber sheet did not occur. From the difference in the crack patterns of the retrofitted beams, it could be judged that partial bond slippage of the main bars in E15-C0 occurred, because the main bars in compressive area could not resist the compression force with concrete.

Shear Force and Drift Angle Response

Figure 9 shows the relationship of shear strength versus drift angle. The calculated values of flexural strength and shear strength using the following equations (Architectural Institute of Japan; 1991) are shown in the Figure.

\[
M_u = 0.9a \cdot \sigma_y \cdot d \\
Q_{mu} = 2M_u / L
\]

(1)

where \(M_u\), \(Q_{mu}\), \(a\), \(\sigma_y\), \(d\) and \(L\) are, respectively, the yield flexural moment, area of main reinforcement, the flexural strength, the yield strength of main reinforcement, the effective depth of the beam and shear span.

\[
Q_s = \left\{ \frac{0.053p_{p}^{0.23}(18 + F_c)M}{QD} + 0.12 + 0.85p_{\sigma_s^y} + p_{\sigma_y^f}\sigma_f^y \right\}b \cdot j
\]

(2)

where \(Q_{sat}\), \(p_{p}\), \(F_c\), \(M/QD\), \(p_{\sigma_s^y}\), \(b\) and \(j\) are the shear strength, tensile reinforcement ratio, concrete compressive strength, shear span ratio, shear reinforcement ratio, yield strength of shear reinforcement, beam width, and distance between the centers of stresses (7/8d), respectively. In the retrofitted specimens, \(p_{\sigma_y^f}\) and yield \(\sigma_f^y\) of fiber were considered as the shear reinforcement in the second term of Eq.2. It was assumed that the arrangement of bars was the same as in the structural drafts. The material strengths obtained from the material tests performed prior to the loading tests were used in the calculations.

For the original specimen E15, maximum strength was reached at drift angle R=1/200rad. After the maximum strength was reached, the strength decreased rapidly. The hysteresis loops showed a pinched shape in the vicinity of the origin due to the diagonal shear cracks. Although the calculated shear strength was greater than the observed maximum strength, the failure mode was the shear failure type. The shear force drift angle response was considerably brittle. The critical drift angle was R=1/100rad. The critical drift angle means the drift angle at 80% of the maximum strength. For the retrofitted specimen E15-C0 the strength had reached maximum value at drift angle R=1/400rad in the first loading cycle. After the strength slightly decreased, it then became constant. It was estimated that the main bars had yielded according to the constant values which agreed with the calculated flexural strength. The shape of the hysteresis loops were a typical spindle type. The shear force drift angle response of the specimen E15-C1 retrofitted with both CFRP sheet and epoxy resin was approximately the same as that of E15-C0. In both retrofitted specimens the displacements did not reach the critical drift angle, and the shear force drift angle responses were very ductile. The seismic performance of the original existing beam was significantly improved.
Energy Dissipations

Figure 10 shows the amount of energy absorption of each specimen. There was an obvious difference between the original specimen and the retrofitted specimens. For the original specimen E15, the increase of energy absorption was approximately linear until the final failure, with a maximum value of approximately 10kN·m at drift angle $R = 1/33$rad. When the deformation surpassed drift angle $R = 1/50$rad, the energy absorption was close to being constant. On the other hand, in the retrofitted specimens E15-C0 and E15-C1, the amount of energy dissipation began to increase rapidly after yielding of the main bars at drift angle $R = 1/100$rad. When the drift angle increased, the amount of energy dissipation continued to increase, and at the final stage was at more than 20kN·m. The amount of energy dissipation of E15-C1 was greater than that of E15-C0 due to the increase of bond strength by epoxy resin.

Details of Bar Arrangements and Cross Sections

After loadings, in order to inspect the real bar arrangements, the cracked concrete and the broken CFRP sheets were removed. The observed bar arrangement was different from that shown in the structural drafts. Specifically, the spaces between the stirrups were larger than 300mm, and in places more than 500mm as shown in Figure 11. The ends of the stirrups were anchored to the top main bars with 90degree hocks. The amount of lateral reinforcement would undoubtedly have an influence on shear strength.

DISCUSSIONS

The maximum strength of each specimen is summarized in Table 2. Calculated strengths were obtained from the above two equations based on the actual bar arrangements and the measured sections. From the calculated results, it is found that the failure mode was the shear failure type, although the estimated failure mode based on the structural draft was the flexural failure type. The observed failure mode of the original specimen agreed with the calculated results. In the other retrofitted specimens, the observed failure modes were judged to be flexural failure from the crack pattern and the hysteretic responses. These results corresponded with the calculated results.

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Observed[kN]</th>
<th>Flexural</th>
<th>Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Positive</td>
<td>Negative</td>
<td>Average</td>
</tr>
<tr>
<td>E15</td>
<td>92.4</td>
<td>-126.9</td>
<td>109.7</td>
</tr>
<tr>
<td>E15-C0</td>
<td>89.4</td>
<td>-77.5</td>
<td>83.5</td>
</tr>
<tr>
<td>E15-C1</td>
<td>99.0</td>
<td>-111.1</td>
<td>105.5</td>
</tr>
</tbody>
</table>

Effective depth $d = 380 \rightarrow 400$  Stirrup 2-9ϕ@300 → Ignoring
CONCLUSIONS

Based on the experimental results the following conclusions can be made;
1) According to the structural drafts, the failure mechanism was expected to be flexural failure. The failure mode of the original specimen was shear failure.
2) There were obvious differences in the existing beam cross-section when comparing the actual section and structural drafts, particularly section shape and bar arrangements.
3) The maximum strength of existing beams can be approximately estimated by examining the cross-sectional properties.
4) The seismic performance of the retrofitted specimens using CFRP sheets and epoxy resin significantly improved.
5) Samples from existing buildings should be accumulated to obtain more exact information about the performance of the existing members, and that will be helpful for retrofitting those buildings in the future.

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