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Seismic Performance of Actual Members Retrofitted with Epoxy Resin Injection

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Abstract

Two beams and one column were taken from a primary school building that was constructed in 1961. Judging from the unit weight of the concrete it was estimated that the members were made of lightweight concrete. The obtained actual members were subjected to reversal loadings in order to investigate the effectiveness of the retrofitting method with epoxy resin. All test members were designed as the shear failure type based on the specifications of the Seismic Evaluation Standard of Japan. One of the beams was repaired with the epoxy resin injection method because honeycombs and cold joints in the concrete were observed. The column damaged by the first loading was also repaired using the same method as the beam. Shear and flexural cracks occurred during the early stage of loading and the strengths rapidly decreased at deflection angle R=1/100rad~1/200rad in all members. Although the crack patterns of the retrofitted members were similar to the members without epoxy resin injection, the maximum strength of the retrofitted members increased to 1.3~1.6 times that of the members without epoxy resin injection. The validity of the empirical equations for shear cracking and shear strength is discussed considering the light weight concrete, the low strength concrete, and the epoxy resin injection.

Keywords: actual member, existing RC building, seismic performance, lightweight concrete, shear strength

1. Introduction

In Japan, seismic performance of existing buildings has typically been evaluated by the standard [1] based on their structural drawings. In many existing buildings however, differences between actual members and the structural drawings have been found. Therefore, it is very difficult to evaluate the seismic performance of an existing building accurately. In the field of civil engineering, performance examinations have been carried out using the RC members of an old RC railway bridge, and the applicability of formulas has been evaluated [2~4]. However, in the field of building engineering, there are very few experimental tests concerning the actual RC members of old buildings, although full scale experiments have been done using existing buildings [5, 6]. Therefore, the research by Aoyama [7] and Araki [8] on the seismic performance of RC members obtained from old buildings is extremely valuable. In this paper, the seismic performance of the actual lightweight concrete beams and columns are investigated.

2. Existing building

The target building was a three story reinforced concrete building constructed in 1961, and used as an elementary school as shown in Fig.1. This building was judged to have low seismic performance in the seismic evaluation. Because this building was designed based on the old structural code of Japan, the main reason for the low seismic performance was the small amount of shear reinforcement. The other reason was the low strength concrete being less than the specified concrete strength of the construction year, estimated 13.2N/mm^2 ~17.6N/mm^2 (135kg/cm^2 ~ 180kg/cm^2).

The concrete strength in the report of the seismic evaluation was 10N/mm^2 under the lower limit of 13.5N/mm^2, as recommended in the applicable standards [1]. Two beams and one column were obtained when the part constructed in 1961 was demolished as shown in Fig. 2. The eleven concrete cylinders were obtained by boring from the structural members for material tests as shown in Fig.3.


Fig. 1 – School building


Fig. 2 – Obtained beam


Fig. 3 – Concrete cylinder

3. Exprimental procedure

3.1 Mechanical properties of lightweight concrete

In seismic evaluation assessments, concrete cylinders used for the compressive tests are usually obtained from the non-structural members, for example, the wing wall or spandrel wall. This is because the work process of concrete boring from those members is relatively easy. However, it is reported that the concrete strength of those members is frequently lower than the concrete strength from the structural members. Therefore, the concrete cylinders in this paper were obtained from the structural members, the columns and the girders of each floor. The diameter (d) and height (h) of the test concrete core was 100mm and 200mm, following the Japanese Industrial Standard (JIS A 1107) [9]. The average compressive strength of all test concrete cores was 12.5N/mm^2. The maximum and minimum compressive strengths were 10.4N/mm^2 and 16.0N/mm^2 respectively. The average of the unit weight was 17kN/m^3. The concrete used in this building was classified as lightweight concrete from the unit weight, although there was no classification of concrete in the RC standard (1958) [10] when the building was constructed. The coarse aggregate was milky white and porous. From the principal component analysis of concrete, it was found that the coarse aggregate was made from rhyolitic welded tuff. It was estimated that this aggregate was the main reason for the low strength concrete. 23kN/m^3 of unit weight is

(a) Welded steel plate (b) Form work (c) Concrete casting for both stubs

Fig. 4 – Process of manufacturing test member

recommended for normal strength concrete. Tensile strengths were obtained by splitting tensile tests using two concrete cores. The average tensile strength of this building was approximately 2N/mm^2.

3.2 Test beams and column

Two beams in the second floor and one column in the third floor were taken without any damage using a wire saw. The reinforced concrete stubs were manufactured at both ends of each member to fix to the testing machine. Steel plates t=12mm were welded at both ends of the main reinforcement for anchorage before casting concrete for stubs. In order to ensure the connection between the original concrete to the stub concrete, shear keys of 12-D16 were installed to the member sides with epoxy mortar. The process of manufacturing the test column is shown in Fig. 4. According to the structural drawing, the main reinforcements and shear reinforcements were plain round bar. The sectional area of the beam and the column were 300mm×600mm and 300mm×750mm respectively. All test members were designed to have a common shear span length of 1200mm in order to evaluate the validity of the current equation for the shear capacity.

It is most important to evaluate the shear capacity of the RC members in the seismic evaluation when the concrete strength does not satisfy the specified concrete strength, or less than 13.5N/mm^2. The flexural strength Q_{mu} and the shear strength Q_{su} are calculated by the following equations (1) and (2) in the standard [1].

$$
\begin{aligned}
&M_u = 0.9a_t \cdot \sigma_y \cdot d && \text{beam} \\
&M_u = 0.8at \cdot \sigma_y \cdot D + 0.5N \cdot D\left(1 - N/F_c \cdot b \cdot D\right) && \text{column} \qquad (1)\\
&Q_{mu} = 2M_u / L
\end{aligned}
$$

where M_u is the yield flexural moment [N·mm], Q_{mu} the strength at the flexural failure [N], a_t is the area of main reinforcement in tension [mm^2], σ_y is the yield strength of main reinforcement [N/mm^2], d is the effective depth of the beam, D is the depth of the column, b is the member width [mm], N is the applied axial force for the column [N], and L is the length of shear span [mm].

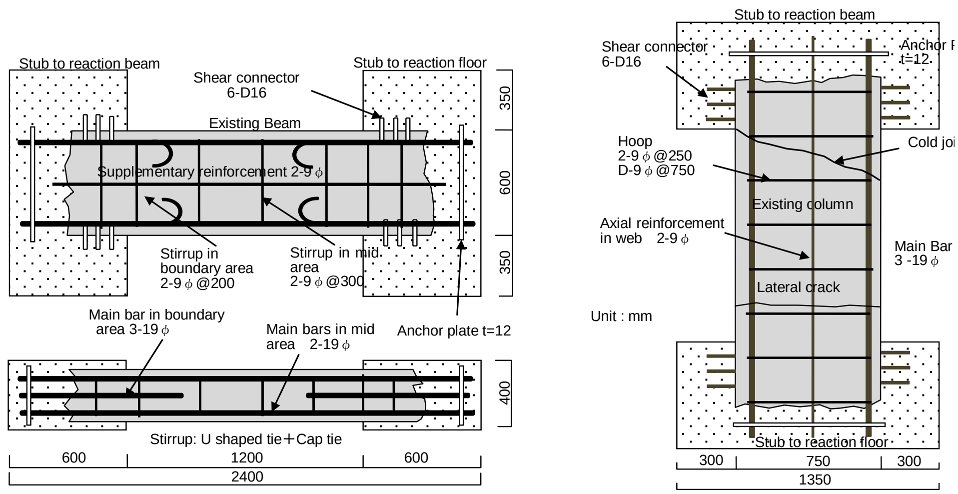
$$
Q_{su} = \left\{ \frac{0.053 p_t^{0.23}\left(18 + F_c\right)}{M/\left(Q \cdot d\right) + 0.12} + 0.85\sqrt{p_w \cdot \sigma_{wy}} + 0.1\sigma_0 \right\} b \cdot j \qquad (2)
$$

where Q_{su}, is the strength at the shear failure [N], p_t is the tensile reinforcement ratio [%], F_c is the compressive strength of the concrete [N/mm^2], M/Qd is the shear span ratio, p_w is the shear reinforcement ratio, σ_{wy} is the yield strength of the stirrup and the hoop [N/mm^2], σ_0 is the axial stress(=0 for the beam), and j is the distance between the resultant internal forces (7/8d) [mm]. Eq. (2) is the minimum shear strength as empirically proposed by Arakawa [11], which is most commonly used in Japan. M/Qd=1 was assumed in the equation following the RC standard when M/Qd was less than 1. The yield strength of reinforcements (SR24) was assumed to be 294N/mm^2 and the concrete strength was 13.5N/mm^2 according to the standard [1]. The reduction factors for lightweight concrete for the crack strength and the maximum strength are 0.8 and 0.75 proposed by Arakawa [11]. The shear span was designed to be 1200mm expecting that the failure mechanism of the test member was shear failure mode. Using Eq.(1) and Eq.(2), the rate of Q_{su} to Q_{mu} of the test beam and the test column were 0.84 and 0.61 respectively. Details of the test members are summarized in Table 1.

Table 1 – List of test members

Test member	Section $b \times D$[mm]	Specified strength Fc[N/mm^2]	Shear span [mm]	Main bar SR24		Shear reinforcement SR24	Repair	Qsu/Qmu
AB-1	300×600	13.5	1200	End area	3- 19φ	2-9φ@200	-	0.84
AB-1RE				Mid area	2- 19φ	2-9φ@300	Epoxy	
AC-1	300×750			3- 19φ		2-9φ@250 (D-9φ@750)	-	0.61
AC-1RE*							Epoxy	

*AC-1RE was the damaged test column AC-1 repaired by epoxy resin injection method.



(a) Beam (b) Column

Fig. 5 – Details of test members

3.3 Loading and measurement scheme

The test setup was designed to subject the test members to shear force reversals. The test beam was set vertical to the testing machine as shown in Fig.6. The top stub was fixed to the L shaped steel beam and the bottom stub was fixed to the reaction floor with high tension bolts. Shear force was applied by a horizontal jack under displacement control, attempting one cycle for each of the peak displacement levels of drift angle R=1/800radian, 1/400rad, 1/200rad, 1/133rad, 1/100rad, 1/67rad and 1/50. The applied axial load for the test column was constant at N=304kN ($N/F_c \cdot b \cdot D$=0.1) during lateral loading considering that the column was taken from the third floor. To ensure that the top and bottom stubs remained parallel during reversal loadings, a pantograph system was used.

The shear displacement between the top and bottom stubs was measured by a Linear Viable Differential Transducer (LVDT). In order to measure the local displacements of the test members, 17 LVDTs were mounted on one side of the test member as shown in Fig. 7. The lateral and axial loads were measured by load cells attached to the jacks.

3.4 Epoxy resin injection

Since the 1950s, epoxy resins have been used in the repair of reinforced concrete structures [12]. Epoxy resin is one of the general materials used in repair methods and is usually injected at the concrete surface. Unlike the usual method in which epoxy resin is injected at the concrete surface of the members, in this method epoxy resin is injected at the position of the reinforcing bar, or into the concrete 50mm from the concrete surface. The injected epoxy resin unites cracked concrete blocks and bonds the concrete and the reinforcing bars together. It is expected that the low seismic performance of severely deteriorated or buildings damaged during an earthquake can be restored by epoxy resin injection alone, although epoxy injection is usually used together with wrapping steel plates or CFRP sheets. In the beams and the column, cold joints and honeycombs were found, and some cracks caused by drying shrinkage. To investigate the effect of retrofitting, epoxy resin was injected to one of the


Fig. 6 – Test apparatus


Fig. 7 – Measurement

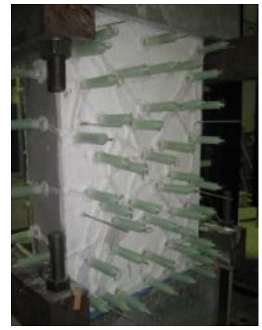


Fig. 8 – Epoxy resin injection

beams and the damaged column after the first reversal loading. In addition, the damaged surfaces of the test members were repaired with fiber mortar, the strength of which was the same as the original concrete. Epoxy resin of 100~200mPa.s at a very low pressure of 0.06N/mm^2 was injected with spring capsules at the location of the deficiencies as shown in Fig.8. The total amount of epoxy resin injected into the test beam was 4.5kg. It is presumed that there was a large void in the concrete. 15.15kg of epoxy resin was injected into the damaged column due to the shear cracks. Assuming a unit weight of epoxy resin 1.1, the total volumes of injected epoxy resin were approximately 4100 and 13800 cubic centimeter. To designate the retrofitted test members, "RE" was added to the name of the original member AB-1 and AC-1. In the case of the column, the epoxy resin was injected in a state where the test column was subjected to the axial force.

4. Test results

4.1 Crack patterns

4.1.1 Beams

Crack patterns of the test beams at the maximum strength are shown in Fig.9 (a). In both test beams, slight flexural cracks occurred at the ends of the beams in the first cycle at drift angle R=1/800 radians. In the same cycle, shear cracks occurred at the mid spans of the beams. As the controlled displacement increased, the shear cracks occurred in the entire beam. The width of the specific shear cracks were enlarged while the flexural cracks did not progress. In the original test beam AB-1, the width of the shear cracks near the cold joint rapidly expanded at drift angle R=1/200 radians. After this event, a new crack did not occur. The retrofitted test beam AB-1RE showed almost the same crack propagation as AB-1. No significant difference in crack patterns between the two beams was observed, and the collapse mechanism was apparently the shear failure mode. Loading was discontinued when shear force decreased to less than half of the maximum strength.


(a) AB-1

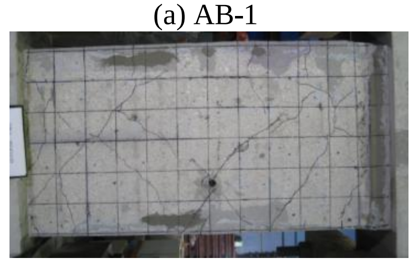

(b) AB-1RE

Fig. 9 (a) – Crack patterns of test beams

4.1.2 Columns

The test columns showed almost the same crack propagation as the test beams shown in Fig.9 (b). While the angles of the shear cracks of the test beams were approximately 45 degree, the shear cracks of the test columns occurred diagonally due to the axial force. The shear cracks of original test

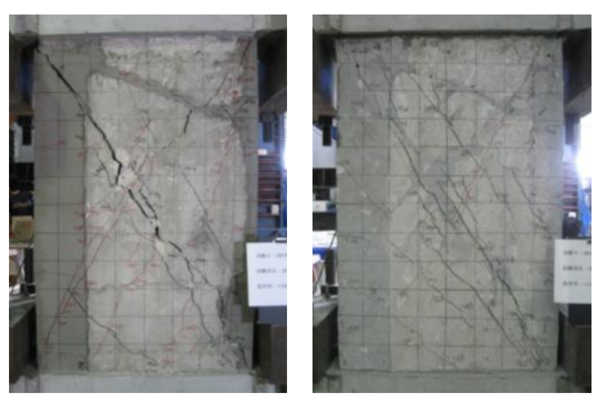

(a) AC-1 (b) AC-1RE

Fig. 9 (b) – Crack patterns of test columns column AC-1 occurred intensively in the diagonal direction although the cracks of the retrofitted test column AC-1RE occurred in the entire column. The locations of the main shear cracks were different to those of the original test columns AC-1. It is estimated that the large amount of epoxy resin injected in to the damaged column united the cracked concrete blocks. At the same time, the bond strength of the plain round bars in the concrete increased.

4.2 Shear force and drift angle response

The relationships of shear force P versus drift angle R are shown in Fig.10. The flexural strength and shear strength calculated by Eq. (1) and Eq. (2) based on the structural drawings are inserted in the Figure.

4.2.1 Beams

For the original test beam AB-1, the degradation of stiffness was observed at the early stage of loading due to the occurrence of the shear cracks and the maximum strength 154.9N was recorded at drift angle R=1/133 radians. The observed maximum strength was slightly less than the calculated shear strength167kN by Eq. (2). The hysteresis loops showed a slip shape in the vicinity of the origin due to the shear cracks. For the retrofitted test beam AB-1RE, the strength had reached the maximum value 240.6kN at drift angle R=1/200 radians in the third cycle of loadings. The maximum strength exceeded even the calculated flexural strength 199kN by Eq. (1), although the failure mechanism was the shear failure mode. The peak strength of each cycle gradually decreased and the strength at drift angle R=1/57 radians maintained 80% of the maximum strength. The hysteresis loops showed a slip shape as per the original test beam AB-1. In the large drift angle, the retrofitted test beam AB-1RE showed more ductile behavior.

4.2.2 Columns

For the original test column AC-1, the maximum strength 305.5kN was recorded until drift angle R=1/100 radians. The maximum shear force was greater than the strength calculated by Eq. (2) and decreased rapidly after the maximum strength until drift angle R=1/100 radians. After the degradation, the feature of hysteresis loops changed from the spindle type to slip type. In the retrofitted test column AC-1RE, the maximum strength 426.8

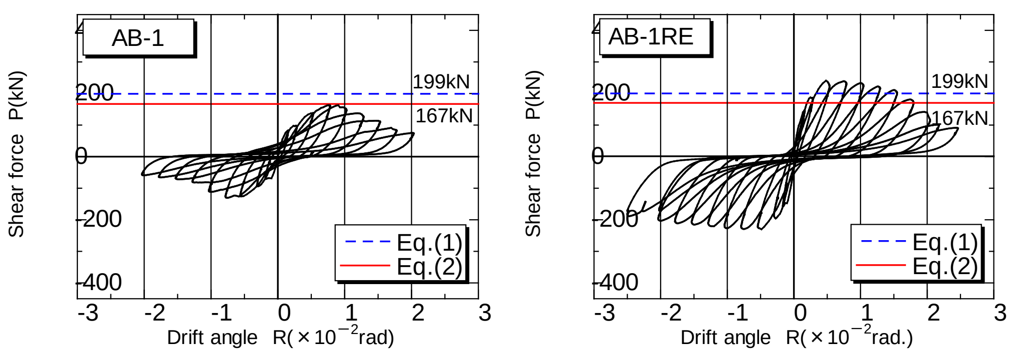


Fig. 10(a) – Shear force and drift angle response of test beams

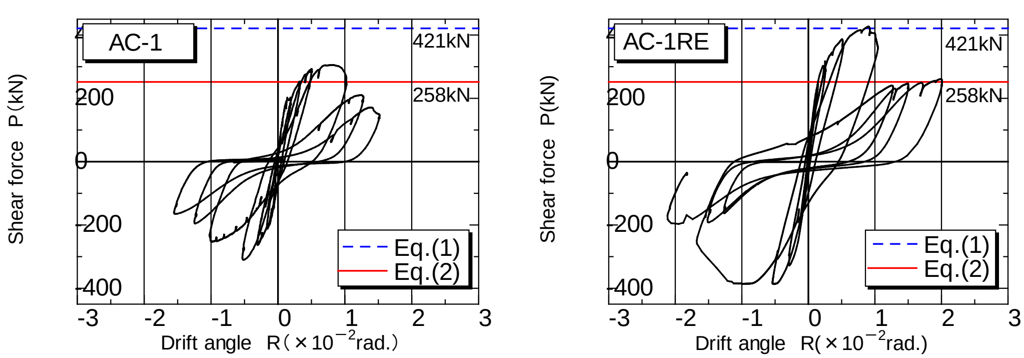


Fig. 10(b) – Shear force and drift angle response of test columns

Table 2 – Increasing rate of strength

Test member	Strength of shear crack		Maximum strength			
	Obtained (kN)	Increasing rate	Obtained (kN)			Increasing rate
			Positive	Negative	Average	
AB-1	82.8	1.77	154.9	138.5	146.7	1.60
AB-1RE	146.4		240.6	230.3	235.5	
AC-1	182.5	1.57	305.5	309.8	307.7	1.32
AC-1RE	287.0		426.8	385.8	406.3	

kN was much greater than the calculated shear strength 258kN by Eq. (2), and increased to the calculated flexural strength 421kN. The collapse mechanism of the column was the shear failure type as with the beam.

4.3 Effect of retrofitting on strength

The maximum strengths of the retrofitted test members were greater than those of the original members. The increasing rate of the strength due to the epoxy resin injection retrofitting methods is summarized in Table 2. The strength of the shear crack increased to 1.77~1.57 times the strength of the original members. The maximum strength increased to 1.60~1.32 times that of the original members. Although the 4.5kg of epoxy resin injected into the beam was smaller than the 15.15kg for the column the increasing rate of the strength for the beam was larger than that of the column. In the beam, not only strength, but also ductility increased. On the other hand, in the column, while the shear strength increased, the ductility did not increase due to the axial force. It is anticipated that there is a strong possibility for use of epoxy resin alone for strengthening the RC members of existing buildings.

4.4 Inspection of bar arrangements after loadings

Bar arrangements in the members were inspected by removing the concrete cover. In the beam, the 9ϕ stirrup was arranged with 200mm~300mm space as shown in the structural drawing. The stirrups consisted of a cap tie and U shaped tie as shown in Fig 11 (a). It was confirmed that two 19ϕ main bars were arranged at the top and bottom through the beam length and a cutoff bar was arranged at the bottom of the beam. Two or three cutoff bars were found at the top of the beam. Those cutoff bars were anchored with a 180 degree hook as shown in Fig 11 (b). The locations of the cutoff were considered to have an influence on the occurrence of the main shear cracks. There was no significant difference in the bar arrangements between the structural drawing and the obtained members. Material tests were performed using reinforcing bars that were taken out after the loading tests. Averages of the yield strengths of 19ϕ and 9ϕ were 320N/mm^2 and 270N/mm^2 respectively from the tensile tests.

(a) Stirrup (Beam) (b) Hoops (Column)

Fig.11 – Inspections for members after loadings

5. Discussion

5.1 Initial stiffness

Initial stiffness obtained from the positive first loops as shown in Fig.12. The theoretical stiffness *K* by Eq. (3) is inserted in the figures.

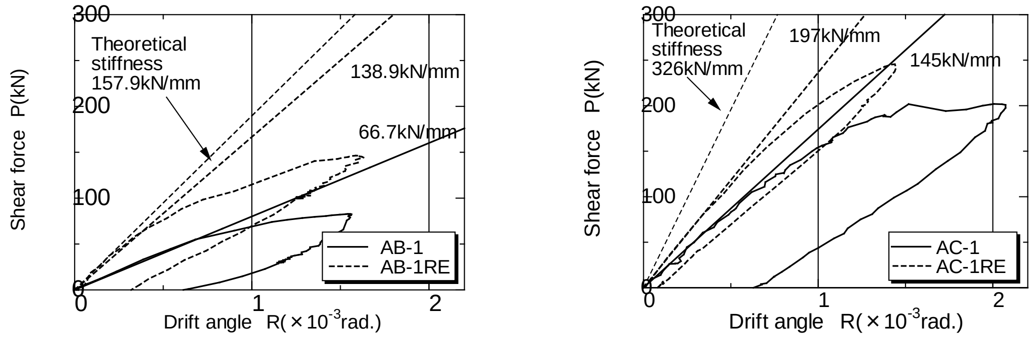


Fig. 12– Initial stiffness of test members

$$\delta_{Total} = \delta_S + \delta_F = \frac{PL}{GA} + \frac{PL^3}{12EI}$$
$$K = \frac{P}{\delta_{Total}} = \left(\frac{L}{GA} + \frac{L^3}{12EI} \right)^{-1} \qquad (3)$$

where, δ_{Total} [mm]is the total displacement of the members, δ_S and δ_F are the shear displacement and the flexural displacement, K[N/mm] is the theoretical stiffness, G is the shear stiffness of concrete [=E/2.3], L is the clear span of the test members [=1200mm], I is the moment of inertia [bD^3/12], A is the sectional area [bD], and P is shear force [N]. Modulus of elasticity of the test beam 6.63kN/mm^2 and the test column 8.46kN/mm^2 from the material tests are used to calculate the theoretical stiffness. The obtained initial stiffness' of the original members before cracking were less than half of the theoretical values.

It is estimated that the stiffness degradation occurred in the existing building due to the quality of construction, deteriorated materials used, and long term degradation. The degradation of the stiffness is not considered in the seismic evaluation for the existing RC buildings at the present time. The stiffness of the test members retrofitted by the epoxy resin injection increased, but did not reach the theoretical stiffness.

5.2 Rate of displacements

In this experimental work, the local displacements were measured by the displacement transducers in order to investigate the rate of the shear displacement δ_S and the flexural displacement δ_F, which included the rotational displacement due to the pullout of the main reinforcements. The rate of the shear displacement to the total displacement ($\delta_S+\delta_F$) is shown in Fig.13. The theoretical rates of the test beam and test column in the elastic range are 0.365 and 0.473 respectively. The theoretical rate is proportional to the D/L of the members. The initial rate of the original beam AB-1 is 0.5~0.6, which is greater than the theoretical rate due to the cold joints or honeycombs in the member. The rate of AB-1 decreased temporarily due to the occurrence of the flexural cracks. The rate began to increase gradually at drift angle 1/600 radians and finally converged to 0.75. The initial rate of the retrofitted test beam AB-1RE is 0.3~0.4 which is approximately consistent with the theoretical value. The converged value is 0.9. The initial rate of the original column increased gradually without decrease. The

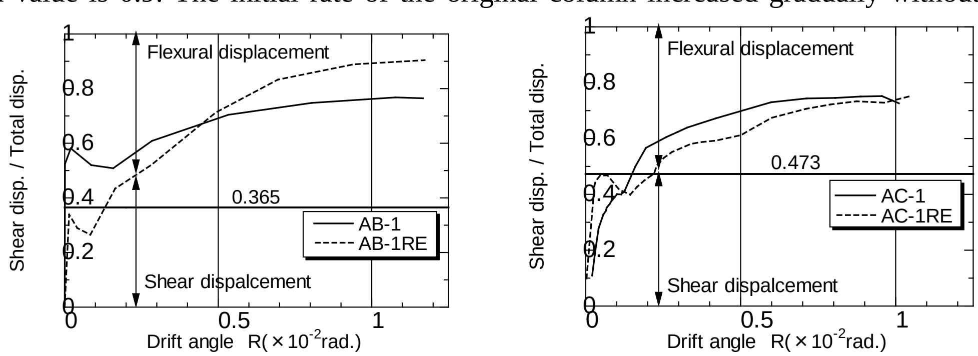


Fig. 13 – Rate of shear displacement to total displacement

Table 3－Mechanical properties of used materials

Concrete			Reinforcement		
Member	Compressive Strength σ_B (N/mm^2)	Modulus of Elasticity Ec (kN/mm^2)	Diameter	Yield Strength σ_y (N/mm^2)	Modulus of Elasticity Es (kN/mm^2)
Beam	11.02	7.63	9ϕ (shear)	270	189.7
Column	13.74	8.46	19ϕ (main)	321	171.0

peak of the rate at the initial stage was not observed. On the other hand, the peak of the rate of the retrofitted column was consistent to the theoretical rate. The rate of both columns converged to the same value 0.7.

5.3 Strength

5.3.1 Strength of shear crack

It is important to investigate the strength of shear cracks to guarantee serviceability under a long term load. The following two equations for the strength of shear crack are commonly used in Japan. Eq. (4) is theoretically derived from the principal stress theory [13]. Eq. (5) is empirically derived from the experimental data using RC members by Arakawa [11]. The tensile stress σ_T is recommended in the standard [13]. The material strengths used in the equations were obtained by material tests after loading test of the members. The observed mechanical properties of the materials were summarized in Table 3.

$$V_c = \phi\left(\sqrt{\sigma_T{}^2 + \sigma_T \cdot \sigma_0}\right)\frac{b \cdot D}{\kappa} \qquad (4)$$

$$\sigma_T = 0.33\sqrt{\sigma_B}$$

$$Q_{sc} = \left\{\frac{0.85k_c\left(50+\sigma_B\right)}{M/\left(Q\cdot d\right)+1.7}\right\} b \cdot j \qquad (5)$$

where, σ_T is the tensile stress [N/mm^2], σ_0 is the axial stress [N/mm^2], ϕ (=1.0) is the reduction factor , κ (=1.5) is the shape factor of the section in Eq. (4) and k_c (=0. 72) is the scale factor in Eq.(5). For the concrete strength σ_B of the test beam and the test column 11.02N/mm^2 and 13.74N/mm^2 were used respectively. These values were the average strengths of the concrete cores for both members. The comparison between the observed and calculated strength of the shear cracks is shown in Table 4. The reduction factor 0.8 is recommended for the crack strength of the lightweight concrete member. The observed crack strengths of the original members were less than the calculated values from the two equations. Specifically, in the test beam and column, the ratios of the observed value to the value calculated by Eq. (5) are 0.56 and 0.88 respectively. The observed strength of the shear crack of both retrofitted members exceeded the calculated values.

Table 4 – Strength of shear crack

Test member	Observed (kN)	V_c×0.8 (kN)	Obs./Cal.	Q_{sc}×0.8 (kN)	Obs./Cal.
AB-1	82.8	105.2	0.79	146.7	0.56
AB-1RE	146.4		1.39		1.00
AC-1	182.5	212.9	0.86	206.3	0.88
AC-1RE	287.0		1.35		1.39

5.3.2 Maximum strength

The validity of the present equation for the shear strength was discussed in comparison with the observed maximum strength. In this paper, the theoretical equation and the empirical equation are used. Shear strength V_u was derived from the ultimate strength concept using truss and arch theory [13] in Eq. (6). The equation (7) expresses the mean values of the test results in the previous study on RC members [11]. This equation was used to directly compare the observed maximum strength in shear failure.

Table 5 – Maximum shear strength

Test member	Observed (kN)	V_c×0.75 (kN)	Obs./Cal.	Q_{sc}×0.75×kr (kN)	Obs./Cal.
AB-1	154.9	161.8	0.96	158.9	0.97
AB-1RE	240.6	249.3	0.97	200.8	1.20
AC-1	309.8	280.1	1.11	301.3	1.03
AC-1RE	426.8	525.9	0.81	397.1	1.07

$$V_u = b \cdot j_t \cdot p_w \cdot \sigma_{wy} \cdot \cot\phi + \tan\theta(1-\beta) \cdot b \cdot D \cdot \nu \cdot \sigma_B / 2$$
$$\beta = \left\{\left(1+\cot^2\phi\right) p_w \cdot \sigma_{wy}\right\} / \left(\nu \cdot \sigma_B\right) \tag{6}$$
$$\tan\theta = \left\{\sqrt{\left(L/D\right)^2+1} - L/D\right\}$$

$$Q_{su} = \left\{\frac{0.068 p_t^{0.23}\left(18+F_c\right)}{M/\left(Q \cdot d\right)+0.12} + 0.85\sqrt{p_w \cdot \sigma_{wy}} + 0.1\sigma_0\right\} b \cdot j \tag{7}$$

where, ν is the reduction factor for concrete and ϕ is the angle of the compressive strut. The other calculated shear strength Q_{su} obtained by Eq. (7) was multiplied by 0.75 and *kr*. The reduction factor 0.75 for the lightweight concrete was proposed by Arakawa [11]. The reduction factor *kr* was empirically derived for low strength concrete of less than 13.5N/mm^2 proposed by Yamamoto [14] and shown in Eq.(8). When the concrete strength σ_B is greater than 13.5N/mm^2 , the redution factor *kr* was 1.0.

$$kr = 0.244 + 0.056 \cdot \sigma_B \tag{8}$$

Therefore, the reduction factor *kr* was 0.86 for the test beam due to the concrete strength 11.02N/mm^2, where *kr* for the test column was 1.0. For the retrofitted members, it is impossible to evaluate the maximum shear strength without consideration of the epoxy resin. In this paper, a trial to estimate the shear strength of the retrofitted members was performed assuming the injected epoxy resin results in the effects of the shear reinforcement. The product of the volume ratio and the tensile strength of the epoxy resin was added to the product of the shear reinforcing bars as shown Eq. (9).

$$p_w \cdot \sigma_{wy} \Rightarrow p_w \cdot \sigma_{wy} + \rho_e \cdot \sigma_e \tag{9}$$

where, ρ_e is volume ratio and σ_e is the tensile strength of the used epoxy resin. A unit weight of the epoxy resin 1.1g/cc is assumed. The observed and calculated maximum strengths are summarized in Table 5. For the original members, no significant difference between the values calculated by Eq. (6) and Eq. (7) was observed. From the results of the comparions, it is reasonable to consider that the reduction factors included the deficiency of the construction or low strength concrete. For the retrofitted test members, the shear strength calculated by Eq. (7) was estimated to be less than the observed strength, while the shear strength calculated by Eq.(6) was overesitemated in comparison to the the observed strength.

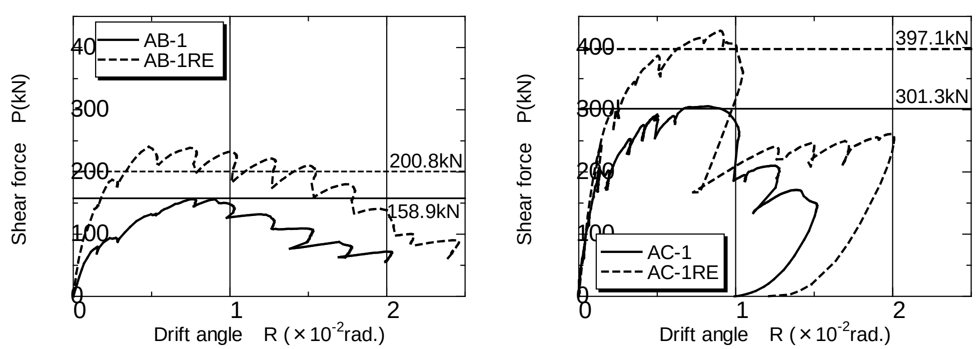


Fig. 14 – Envelope curve and calculated maximum strength considering retrofitting

The comparisons between the shear strength calculated by Eq. (7) and the observed envelope curves of the shear force responses are shown in Fig.14. However, it is noted that the amount of the injected epoxy resin is depend on the cracks or the vacant space in the members due to the minor eqrthquakes or deficiency of the construction. Therefore, to evaluate the quantitative effect of the injected epoxy resin, further experimental works with the member from the existing building are required.

6. Conclusions

Based on the experimental investigations of actual lightweight concrete beams and column, the following conclusions are drawn.

1. The unit weight of concrete was classified as lightweight concrete. It was found that the coarse aggregate was made from rhyolitic welded tuff from the principal component analysis of concrete.
2. The compressive strength of the beam and the column were $11.02 N/mm^2$ and $13.74 N/mm^2$ respectively and the tensile strength was $2 N/mm^2$ from the material tests.
3. The present equation for the strength of shear crack recommended in the standard tends to significantly underestimate the observed value of the original members.
4. The present equation for shear capacity could predict the observed value considering the reduction factors for the lightweight and low strength concrete.
5. Epoxy resin injection significantly improved the seismic performance of the RC members.
6. It is possible to estimate the maximum shear strength evaluating the effect of the injected epoxy resin. In order to evaluate the quantitative effect of the epoxy resin, further experimental works with the member from the existing building are required.

7. Acknowledgements

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8. References

[1] Japan Building Disaster Prevention Association, 2001, “Standards of seismic evaluation of existing RC buildings” Revised Edition.

[2] Yokozawa, A. *et al*. 2005, “Degradation and structural performance of RC pier superstructure in service”, *Conference on Structural Engineering and Construction*, Vol. 27, No. 2, pp.1591-1596.

[3] Niitani, K. *et al*. 2005, “Studies on loading and durability of prestressed concrete member which was in service for 38 years”, *Pacific Conference on Structural Engineering and Construction*, Vol.27, No.2, pp.1585-1590.

[4] Hattori, N. *et al*. 2006, ”Mechanical Properties of RC railway member has been in service for 60 years” ,*Pacific Conference on Structural Engineering and Construction.*Vol28, No.2, pp.1621-1626.

[5] Osawa, Y. *et al.* 1968, “Destruction of the old building vibration and Tokyo Marine Building, Part Examination of safety against seismic force”, *Concrete Journal*, Vol.6, No. 6, pp.16-26.

[6] Matsushima, Y. 1970, “Experimental tests reported of full scale destruction of the reinforced concrete wall-type five-storey apartment”, *Concrete Journal*, Vol.8, No.1, pp.11-21.

[7] Aoyama, H. et al. 1983, “Study on earthquake resistance of a historic SRC building (Old AIJ Building) Part 1-3”, Summaries of Technical of Annual Meeting Architectural Institute of Japan,pp.2465-2470.

[8] Araki, H. et al. 2013, "Seismic performance of actual RC members retrofitted with CFRP and epoxy resin", *Proceedings of APFIS2013*, Melbourne.

[9] Japan Industrial Standard, 2009, "Method of sampling and testing for compressive strength of drilled cores of concrete, JIS A 1107.

[10] Architectural Institute of Japan, 1991, 1971, 1962, 1958, "Standard for structural calculation of reinforced concrete structures".

[11] Arakawa, T. 1960, "Shear resistance of reinforced concrete beam", *Transactions of Architectural Institute of Japan*, No.66, pp.437-440.

[12] Rooney, H.A., (1968) Epoxies in Concrete Construction and Maintenance, paper Special Publication, 21-2, ACI

[13] Architectural Institute of Japan,1990 "Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Ultimate Strength Concept".

[14] Yamamoto, Y., 2005, "Points at low strength concrete in seismic retrofitting for existing building (Earthquake and strengthening), *Japan association of architectural firms*, pp.77-91,Oomiya.