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Seismic Performance of Lightweight Concrete Beams from Existing Building

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ABSTRACT: Experimental works were performed using two beams taken from a primary school building that was constructed in 1961. Judging from the unit weight of the concrete it was estimated that the beams were made of lightweight concrete. While the specified concrete strength was not shown in the structural draft, the average concrete strength obtained from the material tests was $12.5N/mm^2$. One of the beams was repaired with epoxy resin injection, because honeycombs and cold joints in the concrete were observed in both beams. Shear failure modes were observed in both beams. Although the crack patterns of both beams were similar, the maximum shear strength of the retrofitted beam increased to 1.5 times that of the original beam. Discussion on the validity of the empirical equation for shear strength was performed considering the lightweight concrete, the low strength concrete, and the epoxy resin injection.

1 INTRODUCTION

In Japan, seismic performance of existing buildings has typically been evaluated by the standard (Japan Building Disaster Prevention Association, 2001) based on their structural drafts. In many existing buildings however, differences between actual members and the structural drafts have been found. 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 the RC members of an old RC railway bridge, and the applicability of formulas has 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 the actual RC members of old buildings, although full scale experiments have been done using existing buildings (Osawa *et al*.1968; Matsushima 1970). Therefore, the research on the seismic performance of RC members obtained from old buildings is extremely valuable (Aoyama *et al*.1983; Araki *et al*. 2013). In this paper, the seismic performance of the actual lightweight concrete beams is 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 Figure 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 of 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 $13.2N/mm^2$ ~$17.6N/mm^2$ (135 kg/cm^2~$180kg/cm^2$).

Figure 1. School building

Figure 2. Hoisting a beam

Figure 3. Concrete boring

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 (JBDPA, 2001). Two beams and eleven concrete cylinders were obtained when the part constructed in 1961 was demolished as shown in Figure 2. The concrete cylinders were obtained by boring from the structural members as shown in Figure 3.

3 EXPRIMENTAL PROCEDURE

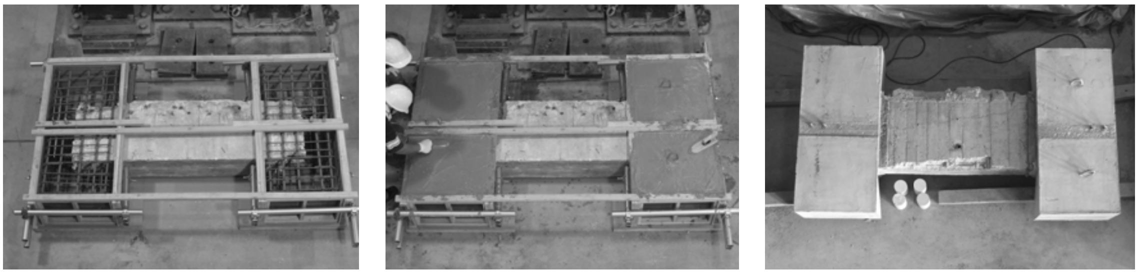
3.1 Mechanical properties of lightweight concrete

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 and height of the concrete test core was 100mm and 200mm, following the Japanese Industrial Standard (JIS A 1107). The average compressive strength of the total concrete test cores was 12.5N/mm^2. The maximum and the 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 was classified as lightweight concrete from the unit weight, although there was no classification of concrete in the standard (RC standard, Architectural Institute of Japan, 1958) 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. 23kN/m^3 of unit weight is recommended for normal strength concrete. Tensile strengths were obtained by splitting tests. The average tensile strength was approximately 2N/mm^2.

3.2 Shear tests

3.2.1 Test beams

Two actual beams in the stair hall of the second floor were taken out without any damage using a wire saw. The reinforced concrete stubs were manufactured at both ends of the beams to fix to the testing machine. Steel plates t=12mm were welded at both ends of the main reinforcement for anchorage before casting the concrete. In order to ensure the connection between the original concrete to the stub concrete, shear keys of 12-D16 were installed to the beam sides with epoxy mortal. The process of manufacturing the test beams is shown in Figure 4. According to the structural draft, the main reinforcement and shear reinforcement were plain round bar 3-19φ (SR24) and plain round bar 2-9φ (SR24) @200 at the boundary area of the beam. In the mid section, the reinforcements were 2-19φ and 2-9φ@300. The sectional area of the beam was 300mm×600mm and the effective depth d is estimated as 530mm. Both test beams were



(a) Form work (b) Concrete casting for stubs (c) Removal of forms

Figure 4. Process of manufacturing the test beam

designed to have a common shear span length of 1200mm (shear span ratio *M/Qd* of 1.13) 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 is 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 (JBDPA, 2001).

$$M_u = 0.9 a_t \cdot \sigma_y \cdot d \qquad (1)$$
$$Q_{mu} = 2M_u / L$$

where M_u is the yield flexural moment [N·mm], Q_{mu} the flexural strength [N], a_t is the area of main reinforcement [mm^2], σ_y is the yield strength of main reinforcement [N/mm^2], d is the effective depth of the beam, and L is the length of the shear span [mm].

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

where Q_{su}, is the shear strength [N], p_t is the tensile reinforcement ratio [%], F_c is the specified concrete strength [13.2N/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 [N/mm^2], b is the beam width [mm], and j is the distance between the resultant internal forces (7/8d) [mm]. Using Eq.(1) and Eq.(2), the calculated flexural strength and shear strength were 199kN and 164kN respectively, assuming that yield strength of the reinforcements (SR24) was 294N/mm^2 according to the standard (JBDPA, 2001), and the concrete strength used was 13.2N/mm^2, the minimum strength in the structural draft. In the standard (RC standard AIJ, 1971) the reduction factors for lightweight concrete for the crack strength and maximum strength are 0.8 and 0.75. Because the rate of the shear strength to the flexural strength was 0.83, the failure mechanism was expected to be shear failure mode.

3.2.2 Epoxy resin injection

In both beams, cold joints and honeycombs were found, and some cracks caused by drying shrinkage. Epoxy resin was therefore injected into one of the beams to investigate the effect of retrofitting. In addition, the damaged surface of the beams was repaired with fiber mortal, of which strength was the same as the original concrete. Epoxy resin of 100~200mPa.s was injected at a very low pressure of 0.06N/mm^2 with spring capsules at the location of the deficiencies as shown in Figure 5. The total amount of epoxy resin injected was 4.5kg. Assuming the unit weight of epoxy resin 1.15, the total volume of injected epoxy resin was 4000 cubic centimeters. To designate the repaired test beam, “RE” was added to the name of the original beam AB-1.

Figure 5. Epoxy resin injection with spring capsule

Figure 6. Test apparatus

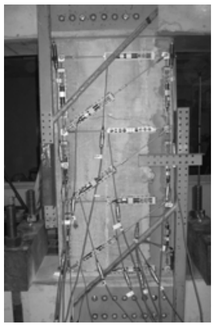


Figure 7. Measurement system

3.2.3 Loading and measurement scheme

The test setup was designed to subject the test beam to shear force reversals. The test beam was set vertical to the testing machine as shown in Figure 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. 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 the bottom stubs was measured by a linear viable differential transducer (LVDT). In order to measure the local displacements, 17 LVDTs were mounted on one side of the test beam as shown in Figure 7.

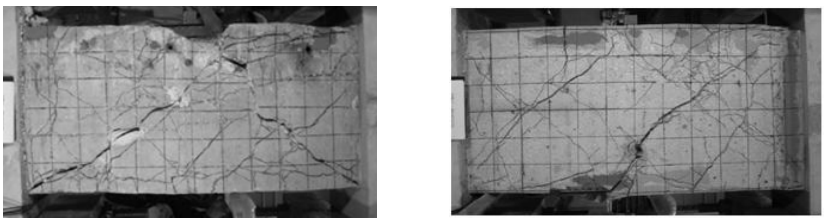
4 TEST RESULTS

4.1 Crack patterns

Crack patterns at the final stage are shown in Figure 8. In both test beams, slight flexural cracks occurred at the ends of the beams in the first cycle, with drift angle R=1/800 radians. In the same cycle, shear cracks occurred at the mid section of the beams. As the controlled displacement increased, 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 both 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 load.

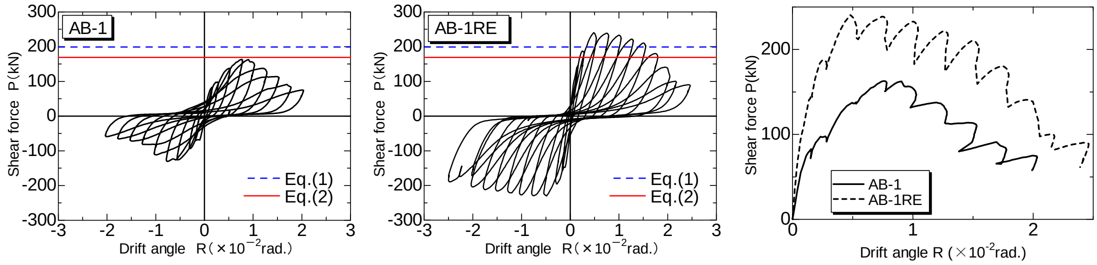
4.2 Shear force and drift angle response

The relationships of shear force P versus drift angle R are shown in Figure 9 (a) and (b). The calculated values of flexural strength and shear strength based on the structural draft are inserted



(a) AB-1 (b) AB-1RE

Figure 8. Final crack patterns



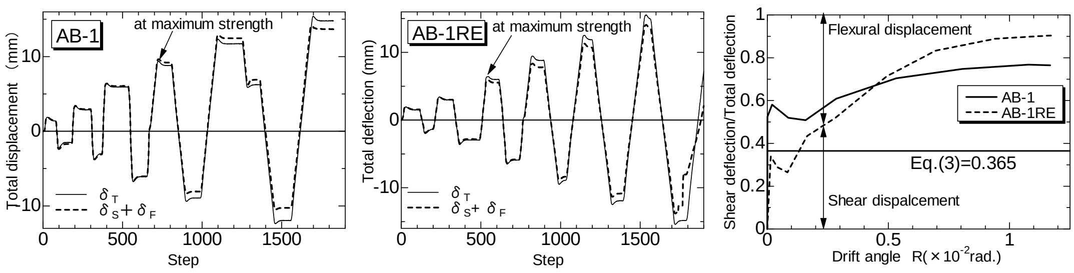
(a) AB-1 (b) AB-1RE (c) Envelope curves

Figure 9. Shear force and drift angle response

in the Figure. For the original test beam AB-1in Figure 9 (a), degradation of stiffness was observed at the early stage of loading and the maximum strength 155kN was recorded at drift angle R=1/133 radians. The strength did not reach the calculated shear strength. After the maximum, the strength decreased rapidly. The hysteresis loops showed a pinched shape in the vicinity of the origin due to the diagonal shear cracks. The shear force drift angle response was considerably brittle. For the retrofitted test beam AB-1RE in Figure 9 (b), the strength had reached the maximum value 241kN at drift angle R=1/200rad in the third cycle of loading. The maximum strength exceeded the calculated flexural strength 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 has maintained 80% of the maximum strength. The hysteresis loops showed a pinched shape as per the original test beam AB-1. Envelope curves of the test beams are compared as shown in Figure 9 (c). The maximum strength and the initial stiffness of AB-1RE had 1.5 times and 2 times those of AB-1. The retrofitted test beam AB-1RE exhibited excellent initial stiffness, the maximum strength and the ductility compared with the original test beam AB-1.

4.3 Rate of displacements

In this experimental work, the total shear displacement between the top and bottom stub and the local displacements were measured as shown in Figure 7. Comparisons between the total shear displacement δ_T and the sum of the local shear displacements δ_S and flexural displacements δ_F are performed as shown in Figure 10 (a) and (b). The horizontal axis (step) in the figure is the number of measurement points. The total displacement δ_T is approximately consistent with the sum of the local shear displacements (δ_S+δ_F), even when the displacement increased. The rate of the shear displacement δ_S to the total displacement (δ_S+δ_F) is shown in Figure 10 (c). The difference between the total displacement and the shear displacement is considered as the flexural displacement which included the rotational displacement due to the pullout of the main rebars. The theoretical rate is 0.365 as estimated by the following Eq. (3) in the elastic range.



(a) AB-1 (b) AB-1RE (c) Rate of shear displacement to total displacement

Figure 10. Rate of displacements

$$\delta_S / \left(\delta_S + \delta_F\right) = \frac{Ph}{GA} \Bigg/ \left(\frac{Ph}{GA} + \frac{Ph^3}{12EI}\right) \tag{3}$$

where, G is the shear stiffness [=E/2.3], h is the clear span of the beam [=1200mm], A is the sectional area [b×D=300mm×600mm], and P is shear force [N]. The initial rate of the original test beam AB-1 is 0.67, which is greater than the theoretical rate due to the cold joints or honeycombs in the beam. The rate of AB-1 began to increase gradually at drift angle 1/600 radians after it temporarily decreased and finally converged to 0.75. The initial rate of the retrofitted test beam AB-1RE is approximately 0.3~0.4 which is consistent with the theoretical value. The converged value is 0.9. When the displacement was large the flexural displacement of the retrofitted test beam was relatively smaller than that of the original test beam. It is estimated that bond characteristics of the plain round bars were improved by the epoxy resin injections and the pullout of the bars were suppressed.

5 DISSCUSSIONS

5.1 *Inspection of bar arrangement*

Bar arrangements in the beams were inspected by removing the concrete cover. The stirrups 9ϕ were arranged with 200mm~300mm space as shown in the structural draft. The stirrups consisted of a cap tie and U shaped tie as shown in Fig 11 (a). It was confirmed that two main bars 19ϕ 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 location of the cutoff was considered to have an influence on the occurrence of the main shear cracks. There was no significant difference in the bar arrangements between the beams. 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.

5.2 *Calculated strength*

5.2.1 Strength of shear crack

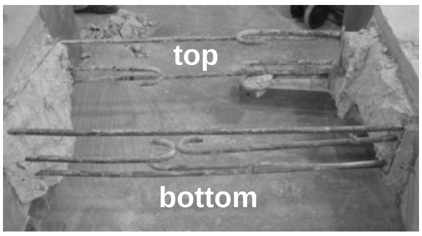
It is important to investigate the strength of shear cracks to guarantee serviceability to long term load. The following two equations for the strength of shear cracks are commonly used in Japan.

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

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

where, σ_0 is the axial stress [N/mm^2], ϕ (=1.0) is the reduction factor and 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).

(a) Stirrups



(b) Main reinforcements

Figure 11. Inspections after loadings

Table 1. Strength of shear crack

Test Beam	Observed (kN) R=1/800rad.	V_c (kN)	Obs./Cal.	Q_{sc}×0.8 (kN)	Obs./Cal.
AB-1	83	140	0.60	158	0.53
AB-1RE	146		1.05		0.92

In the calculations, the concrete strength σ_B was 12.5N/mm^2. The calculated result from Eq. (5) was multiplied by 0.8 as recommended for lightweight concrete (Arakawa, 1960), and is shown in Table 1. The observed values were the peaks of the first cycles of the loading. The crack strength of the original test AB-1 beam was much lower than the calculated values due to the deficiencies of the construction, although that of the retrofitted test beam AB-1RE was approximately the same as the predicted value.

5.2.2 Maximum strength

When the flexural strength Q_{mu} was calculated by Eq. (1) the yield strength 320N/mm^2 obtained by the tensile tests was used assuming that the effective depth d was 550mm based on the inspection. The calculated flexural strength Q_{mu} was 225kN. The calculated shear strength was obtained by two equations. Shear strength V_u was derived from the ultimate strength concept using truss and arch theory (Design guideline, AIJ, 1990) in Eq. (6).

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

where p_w is the ratio of stirrups, σ_{wy} is the yield strength of the stirrup, ν is the reduction factor for concrete, and ϕ is the angle of the compression strut. The other calculated shear strength Q_{su} is obtained by Eq. (2) multiplied by 0.75 and kr. 0.75 is the reduction factor for the lightweight concrete (Arakawa, 1960). The reduction factor kr was empirically derived for low strength concrete of less than 13.5N/mm^2 (Yamamoto, 2005) in Eq. (7).

$$kr = 0.244 + 0.056 \cdot \sigma_B \tag{7}$$

In this study, kr corresponding to σ_B=12.5N/mm^2 is 0.944. In the original test beam AB-1, the predicted shear strength by Eq. (2) considering the lightweight and low strength concrete is artificially consistent with the observed maximum strength. However, it is reasonable to consider that the reduction factors included the deficiency of the constructions. In the retrofitted test beam AB-1RE, the observed shear strength 241kN is much greater than the calculated shear strengths 160kN. The observed shear strength exceeded even the calculated flexural strength. The shear strength of the retrofitted beam increased to 1.5 times that of the original beam. It is estimated that the epoxy resin efficiently repaired the defective or deteriorated beams and improved their seismic performance.

Table 2. Maximum shear strength

Test Beam	Observed (kN)	Flexural strength		Shear strength			
		Q_{mu}(kN)	Obs./Cal.	V_u (kN)	Obs./Cal.	Q_{su}×0.75×kr (kN)	Obs./Cal.
AB-1	155	225	0.70	191	0.81	160	0.97
AB-1RE	241		1.07		1.25		1.51

6 CONCLUSIONS

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

1) The unit weight $17kN/m^3$ 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 and the tensile strength were $12.5N/mm^2$ and $2N/mm^2$ from the material tests.
3) The present equation for the strength of shear cracks as recommended in the standard trends to significantly underestimate the observed value.
4) The present equation for shear capacity could predict the observed value using the reduction factors for the lightweight and low strength concrete.
5) Epoxy resin injection significantly improved the seismic performance of the short beam.

Acknowledgements

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