Seismic Performance of Reinforced Concrete Girders of an Existing Building Constructed in 1971

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ABSTRACT: Experiments were performed on two reinforced concrete (RC) girders taken from a two-story residential building constructed in 1971. The girders were designed to have shear spans of 1200 and 1000mm, such that the validity of the current equations for determining the shear capacity for seismic assessment could be evaluated. The girders were initially subjected to reversed loadings with displacement control up to 1% of the drift angle. After initial loading, assuming that the RC members were moderately damaged because of earthquake, the damaged girders were repaired via epoxy resin injection to investigate the effect of retrofitting. Then, the retrofitted girders were reloaded until failure occurred. In initial loadings, the maximum strength of the original girders reached the estimated flexural strength; however, shear cracks were observed. The maximum strength of the retrofitted girders reached 1.18 and 1.13 times those of the original girders.

1 INTRODUCTION

The seismic performance of existing buildings in Japan is typically evaluated based on their structural drawings in accordance with the *Standards of Seismic Evaluation of Existing RC Buildings* (2001). However, the actual components of the existing buildings are different from their structural drawings; this complicates the accurate evaluation of their seismic performance. In the field of building engineering, few experimental tests have been conducted on the actual RC members of buildings constructed decades ago; however, full-scale experiments were conducted on existing buildings by Osawa (1968) and Matsushima (1970). Therefore, the research by Aoyama (1983) and Araki (2013 and 2017) on the seismic performance of RC members obtained from old buildings is extremely valuable. In this study, the seismic performances of actual reinforced concrete girders were investigated.

2 EXISTING BUILDING

The building under study was a two-story RC building constructed in 1971 and used as an apartment building. Fig.1 shows an image of the building. This building was designed based on the old structural code of Japan. Its poor seismic performance could be attributed to a low amount of shear reinforcement. Concrete with low compressive strength was used in this building. The concrete strengths were distributed over a wide range: from 7.30 N/mm^2 to 22.1 N/mm^2. The average concrete strengths of each story were 17.3 and 9.54 N/mm^2, respectively. The minimum concrete strength in the building was 7.30 N/mm^2, which is much lower than 13.5N/mm^2, which is the applicable lower limit recommended in the *Standards* (2001). Two girders were obtained from each story when the building was demolished in 2017.

Figure 1. Target building.

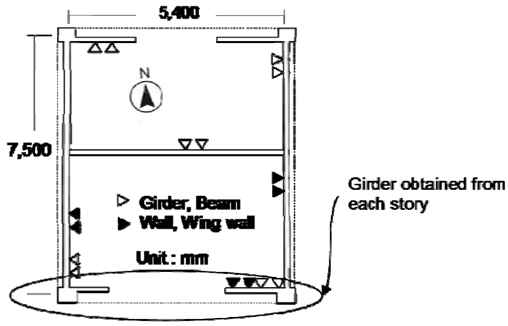


Figure 2. Locations of the obtained specimens.

3 EXPERIMENTAL PROCEDURE

3.1 Original test girders

To estimate the mechanical property of concrete, 32 concrete cylinders were obtained for material tests. Two girders were taken from each story without any damage. Fig.1 and Fig. 2 shows the locations of the obtained girders and concrete cylinders. The sectional area of the girders in the structural drawing was 300 × 600 mm^2. The obtained girders were designed with shear span lengths of 1200 and 1000 mm to validate of the shear capacity equation currently used for seismic evaluation. The original test girders of the first and second stories were termed KB-1 and KB-2, respectively. The structural drawing indicated that the main and shear reinforcements were plain round bars (i.e. 19 and 9φ, respectively). The flexural Q_{mu} and shear strengths Q_{su} were calculated using Eqs. (1) and (2) respectively, which are provided by the *Standard* (2001). Eq. (2) is the minimum shear strength as empirically proposed by Arakawa (1960) and commonly used in Japan. Although concrete strength does not satisfy the specified concrete strength or is less than 13.5 N/mm^2, the shear capacity of the RC members should be evaluated. The other calculated shear strength Q_{su} obtained using Eq. (2) was multiplied by kr. The reduction factor kr was empirically derived for concrete with strength of less than 13.5 N/mm^2, as proposed by Yamamoto (2005) and shown in Eq. (3). The reduction factor kr was 1.0 when the concrete strength σ_B was greater than 13.5 N/mm^2. Therefore, in this study, the reduction factor kr of 0.74 was used for the original test girders (KB-2) given that the estimated concrete strength was 8.89 N/mm^2.

$$\begin{aligned} M_u &= 0.9at \cdot \sigma_y \cdot d \\ Q_{mu} &= {2M_u}/{L} \end{aligned} \tag{1}$$

where M_u is the yield flexural moment [N·mm], 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 [mm], Q_{mu} is the strength at the flexural failure [N], and L is the length of shear span [mm].

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

$$kr = 0.244 + 0.056\sigma_B \tag{3}$$

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 [N/mm^2], and j is the distance between

Table 1. Details of original test girders

Test Girder	Section b×D [mm]	Shear span [mm]	Main bar [SR24]	Stirrup [SR24]	Qsu / Qmu
KB-1	300×600	1200	3-19φ Pg=0.34%	2-9φ@250	1.21
KB-2		1000	2-19φ Pg=0.76%	Pw=0.17%	1.02
Specified concrete strength	Fc 17.6 [N/mm^2] (180kg/cm^2)				

the resultant internal forces (7/8*d*) [mm]. Eq. (2) denotes 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. The estimated concrete strength was obtained by subtracting half the standard deviation of the concrete strength from the average value in accordance with the *Standard* (2001). Using Eqs. (1) and (2), the rate of Q_{su} to Q_{mu} of the test girders were 1.21 and 1.02, respectively. Table 1 presents the details of the original test girders. RC stubs were manufactured at both ends of each girder to enable fixing to the testing machine. Steel plates t = 10 mm were welded at both ends of the main bars for anchorage before casting concrete for stubs. Shear connectors of 24-D16 were installed to the girder sides with epoxy mortar to ensure that the original girder was connected to the stub concrete. Fig. 3 presents the details of the test girder KB-1.

3.2 *Loading and measurement*

The test setup was designed to subject the test girders to shear-force reversals. The top stub was fixed to an L-shaped steel beam, while the bottom stub was fixed to the reaction floor with high-tension bolts. Shear force was applied using a horizontal jack under displacement control. One cycle was attempted per peak displacement level with drift angle R = 1/800, 1/400, 1/200, 1/133, 1/100 rad for KB-1 and KB-2. A pantograph system was used to ensure that the top and bottom stubs remained parallel during reverse loadings. Fig.4 shows the test setup. The shear displacement between the top and bottom stubs was measured using a linear viable differential transducer (LVDT). To measure the local displacements of the test girders, 17LVDTs were mounted on one side of the test girder. Finally, lateral load was measured using load cell.

3.3 *Retrofitting*

Epoxy resin was injected into the original girders after loading to investigate the retrofitting effect. The injected epoxy resin united the cracked concrete blocks and bonded the concrete and the reinforcing bars together. The epoxy resin injection alone may improve the seismic

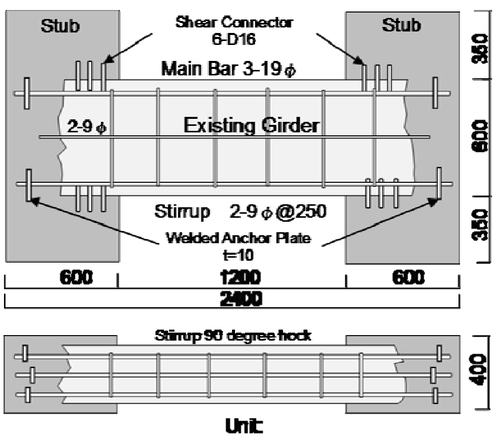


Figure 3. Dtails of the test girder KB-1.

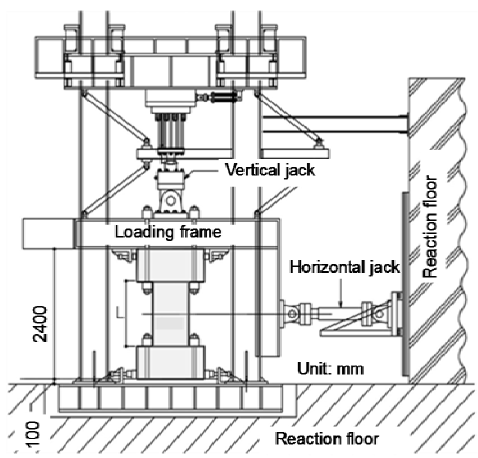
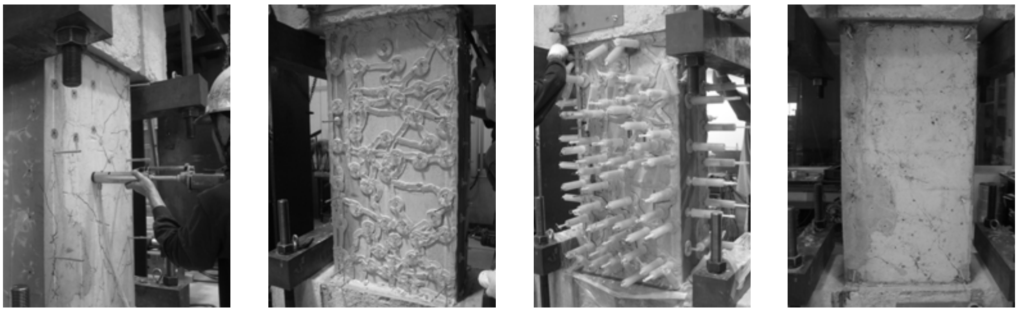


Figure 4. Test setup.



(a) (b) (c) (d)

Figure 5. Retrofitting process of test girder KB-1RE: (a) perforation, (b) sealing and attachments for capsules, (c) epoxy resin injection with capsules, (d) seal removal.

performance of moderately damaged buildings, although epoxy resin injection is usually used with wrapping steel plates or CFRP sheets.

In contrast to the conventional method in which epoxy resin injected at the concrete surface of the members, epoxy resin was injected at the position of the reinforcing bar or into the concrete 50 mm from the concrete surface. Epoxy resin was injected with spring capsules at the location of deficiencies. Fig.5 presents the retrofitting process. The total amounts of epoxy resin injected into the original test girders were 5.5 kg and 4.9 kg. “RE” was added to the name of the original girders to designate the retrofitted test girders, KB-1RE and KB-2RE.

4 EXPERIMENTAL RESULTS

4.1 Material tests

The properties of the concrete in existing RC buildings are directly related to the seismic performance of the buildings; thus, they are important for seismic evaluations. Therefore, compressive and tensile tests were performed. Table 2 summarizes the mechanical properties of concrete. Although the average compressive strength in the second story was approximately half that in the first story, the COV of the first story was close to the applicable upper limit of 0.25 for the seismic evaluation of existing buildings, as shown in a previous study, Sezen & al. 2011. The tensile splitting strengths were distributed over a range: from 1.13 N/mm^2 to 2.67 N/mm^2. The average tensile splitting strengths in the first and second stories were 1.99 N/mm^2 and 1.67 N/mm^2, respectively.

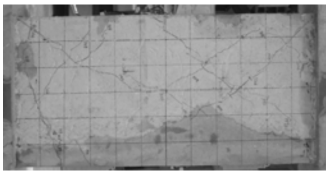
4.2 Crack patterns

Fig.6 (a) and (b) illustrate the crack patterns of the original test girders at maximum strength. Slight flexural cracks were developed in the ends of both girders during the positive loading of the first cycle at drift angle R = 1/800 rad. The shear cracks appeared throughout the entire girders under increasing controlled displacement until drift angle R = 1/100rad. The specific width of flexural cracks increased, whereas the shear cracks did not progress. The cracks did not concentrate on repair portion by the fiber mortar, but along cold joint at the center of the test girder KB-2. Cold joints were frequently observed in the horizontal structural members in the

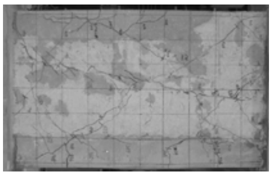
Table 2. Summary of mechanical properties of concrete

Story	Unit weight γ [kN/m^3]	Compressive strength σ_B [N/mm^2]	Modulus of elasticity Ec [kN/mm^2]	Tensile strength σ_{st} [N/mm^2]
1F	21.2 (0.020)	17.3 (0.245)	13.9 (0.083)	1.99 (0.202)
2F	20.9 (0.028)	9.54 (0.140)	9.82 (0.265)	1.36 (0.138)
Total	21.1 (0.025)	13.4 (0.374)	11.9 (0.242)	1.67 (0.266)

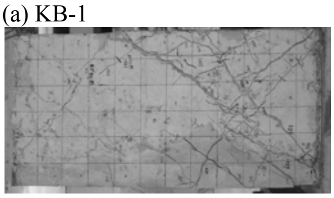
(COV) : Coefficient of variation



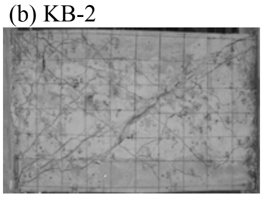
(a) KB-1



(b) KB-2



(c) KB-1RE



(d) KB-2RE

Figure 6. Crack patterns at final stages.

existing RC buildings. No significant difference in crack propagations between the original girders was observed. Fig.6 (c) and (d) show the crack patterns of the retrofitted test girders at maximum strength. The crack propagation process was approximately the same of the original test girders until drift angle R = 1/100 rad. The shear cracks occurred near the shear crack location in the original girders, but not same location. After drift angle R = 1/100 rad., the width of the shear cracks expanded with the increasing drift angle, whereas the flexural cracks did not progress. The final collapse mechanisms were the shear failure type.

4.3 Shear force and drift angle response

Fig.7 depicts the relationships of the shear force *Q* with the drift angle R. The calculated flexural strengths of the original girders were 206 kN and 180 kN. The calculated shear strengths were 251 kN and 184 kN (Fig.7). The stiffness degradations for the all test girders were observed at the first loading cycle. In both of the original girders, the peaks of the shear forces were measured at drift angle R = 1/133 rad. The maximum shear forces almost reached or exceeded the calculated flexural strength, and the apparent strength degradations were not observed. Therefore, the main bars were estimated to be yielding. In contract, the hysteresis loops were of a slip type from the initial stage. The bond slippage of the main bars from the

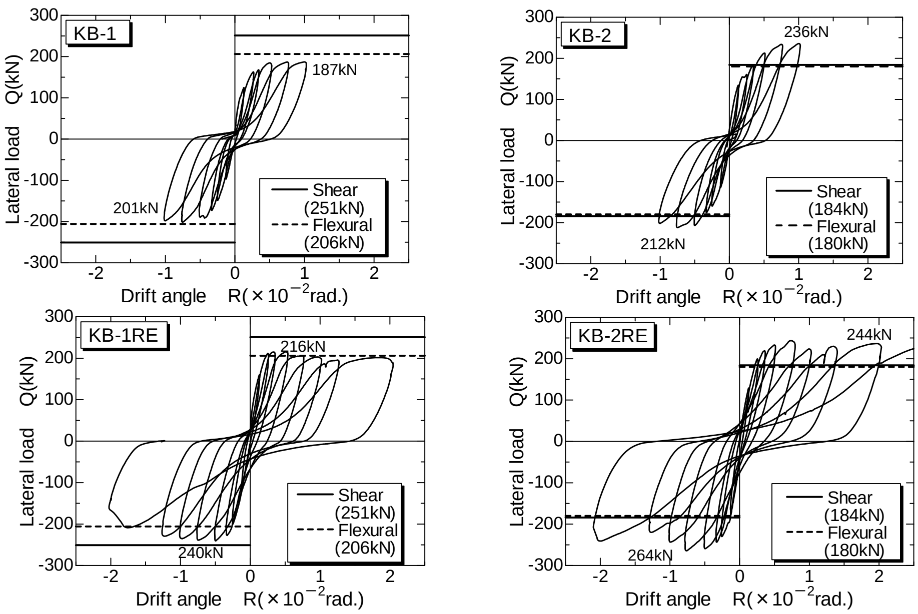


Figure 7. Shear force and drift angle response.

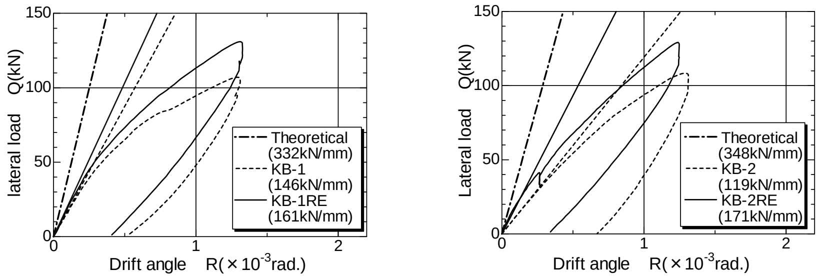


Figure 8. Initial stiffness.

concrete may have occurred because the reinforcement was a plain round bar, and the concrete strength was low. The maximum shear forces in both the retrofitted test girders exceeded the calculated flexural strength. No significant degradation of the shear force was observed until the drift angle R = 1/50 rad. The maximum strength increased in comparison with those of the original test girders. The hysteresis loops apparently showed slip type.

5 DISCUSSIONS

5.1 Initial stiffness

The stiffness degradation of the existing building might be attributed to the construction quality, use of deteriorated materials, and long-term degradation (minor or moderate earthquake). Stiffness degradation is currently ignored in the seismic evaluation of existing RC buildings. The initial stiffness was obtained herein from the first positive loops as shown in Fig.8. The theoretical stiffness K obtained with Eq. (4) is inserted in the figure.

$$\delta_{Total} = \delta_S + \delta_F = \frac{QL}{GA} + \frac{QL^3}{12EI}$$
$$K = \frac{Q}{\delta_{Total}} = \left(\frac{L}{GA} + \frac{L^3}{12EI} \right)^{-1} \quad (4)$$

where, δ_{Total} is the total displacement of the members [mm], δ_S and δ_F are the shear displacement and the flexural displacement [mm], respectively, K is the theoretical stiffness [N/mm], E is the modulus of elasticity [N/mm^2], G is the shear stiffness of concrete [=E/2.3], I is the moment of inertia, A is the sectional area, and Q is shear force [N]. The modulus of elasticity of the test girders (i.e. 13.9kN/mm^2 and 9.82kN/mm^2 in Table 2) from the material tests was used to calculate the theoretical stiffness. The obtained initial stiffness of the original test girders before cracking was 146 kN/mm and 119 kN/mm and approximately one-half and one-third of the theoretical values. The stiffness of the test girders retrofitted through epoxy resin injection increased to 161 kN/mm and 171 kN/mm but did not reach the theoretical values.

5.2 Strength

5.2.1 Shear cracks strength

Investigating the shear crack strength is important for guaranteeing serviceability under a long-term load. The following two equations for the shear crack strength are commonly used in Japan. Eq. (5) is theoretically derived from the principal stress theory, and Arakawa (1960) empirically derived Eq. (6) from the many experimental data of RC members. The tensile stress σ_T was recommended by Collins et al. (1991). The concrete strengths used in the equations were obtained through compressive tests.

$$V_c = \sigma_T \frac{b \cdot D}{\kappa} = 0.33\sqrt{\sigma_B} \frac{b \cdot D}{\kappa} \quad (5)$$

Table 3. List of strength

Test Girder	Strength of shear crack [kN]			Maximum strength [kN]		
	Observed Q_c	Eq.(5) V_c	Eq.(6) Q_{sc}	Observed Q_{max}	Eq.(1) Q_{mu}	Eq.(7) Q_{su}
KB-1	190	165 (1.15)*	220 (0.86)	201	213 (0.94)	302 (0.67)
KB-1RE	187	165 (1.13)	220 (0.85)	240	213 (1.12)	302 (0.79)
KB-2	157	133 (1.18)	213 (0.74)	236	232 (1.02)	286 (0.83)
KB-2RE	203	133 (1.53)	213 (0.95)	264	232 (1.14)	286 (0.92)

( ) *: Obs./Cal.

$$Q_{sc} = \left\{ \frac{0.085 k_c \left(50 + \sigma_B\right)}{M/\left(Q \cdot d\right) + 1.7} \right\} b \cdot j \tag{6}$$

where, σ_T is the tensile stress [N/mm^2], κ (=1.5) is the shape factor of the section in Eq. (5), and k_c (=0. 72) is the scale factor in Eq. (6). For the concrete strength σ_B of the test girders 17.3 N/mm^2 and 9.54N/mm^2 were used. These values were the average strengths of the concrete cores for both members. Table 3 shows the comparisons of the observed and calculated strengths of the shear cracks. The shear crack strength as calculated by Eq. (5) was underestimated while that as calculated by Eq. (6) was overestimated. The crack strength of the test girder KB-2 was lower than the strength of the other test girders because of the cold joint.

5.2.2 Maximum strength

The validity of the present equation for the shear strength was compared with that of the observed maximum strength. The flexural strength was calculated using Eq. (1). The yield strength of the reinforcement in the equation was obtained through tensile tests. The test peaces for the tensile test were taken out from the test girders after loading. The empirical equation for the shear strength was used in this study. Eq. (7) expresses the mean values of the test results taken from the previous studies on RC members. This equation was used for the direct comparison of the observed maximum strength in shear failure.

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

Table 2 summarizes the observed and calculated values for maximum strength. The predicted maximum strengths as calculated by Eq.(1) were in agreement with the observed values of the original girders. The maximum strengths for the retrofitted test girders were 12% and 14% greater than the flexural strengths calculated by Eq. (1) because of the epoxy resin injected to the cracks or around the main bars.

5.2.3 Envelopes and calculated strength

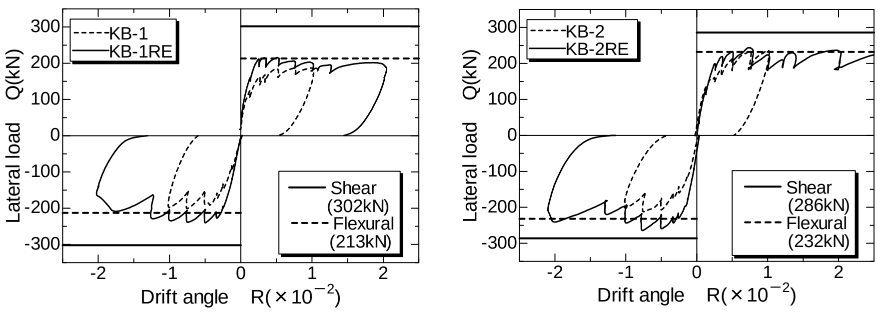


Figure 9. Envelopes of shear force and drift angle response and calculated strengths.

Fig.9 shows the comparisons between the observed envelope curves of the shear force responses and the shear strength calculated by Eqs. (1) and (7).

6 CONCLUSIONS

The following conclusions are drawn from the results of the experimental investigation on actual RC girders obtained from an existing building constructed in 1971:

1. Material tests revealed that eight concrete cylinders obtained from each floor of the building had average compressive strengths of 17.3 N/mm^2 and 9.54 N/mm^2.
2. The failure modes of the original girders could be appropriately predicted through the method recommended in the standard. However, the phase of the bond slippage was observed in the hysteresis loops because of the plain round bars.
3. The initial stiffness of the original girders was approximately one-half and two-thirds of the theoretical value, while that of the retrofitted girders was 1.10-1.40 times that of the original girders.
4. The predicted flexural strengths agreed with the observed maximum values.
5. Epoxy resin injection improved the seismic performance of the RC girders.

Further experimental works with actual RC members from the existing building are required to evaluate the validity of the current equations and the quantitative effect of epoxy resin.

ACKNOWLEDGMENTS

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