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Seismic Performance of the Reinforced Concrete Girders Obtained from an Existing Building Constructed in 1961

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Abstract. *This study examines, through experiments, two reinforced concrete girders obtained from a residential building constructed in 1961. The average concrete strength obtained from the material tests was approximately consistent with the specified strength. Honeycombs were observed in girders; hence, one of the girders was repaired using an epoxy resin injection to investigate the effect of retrofitting. The original and retrofitted girders were subjected to reverse loadings with displacement control and had a sectional area of 250 mm × 800 mm, according to the structural draft. Both girders were designed to exhibit a common shear span length of 1,200 mm to evaluate the validity of the current equations for shear capacity in seismic evaluation, as recommended in the criteria for seismic assessment in Japan. Using these equations, the ratio of the shear strength to flexural strength of the test girders was found to be 1.67. No significant difference was observed in the crack patterns of both girders. Their maximum strength reached the calculated flexural strengths; however, the shear cracks apparently progressed with the increase in drift angle. The hysteresis loops were slip-type because of the bond slippage of the main bars. The final failure mechanism was shear failure mode. The equation for flexural strength predicted the observed value of the original girder; the maximum strength of the retrofitted girder was approximately 1.1 times that of the original. Consequently, the epoxy resin injection recovered the seismic performance of the girders in this building.*

Keywords: *Reinforced Concrete Girder, Existing Building, Seismic Performance, Shear Capacity, Retrofitting.*

1 Introduction

The seismic performance of existing buildings in Japan is typically evaluated based on their structural drawings in accordance with the standards established in the Japanese Building Disaster Prevention Association's (JBDPA) 2001 guidelines. However, the actual components of the existing buildings frequently differ from their structural drawings; this complicates the accurate evaluation of their seismic performance. The equations recommended in the seismic evaluation were empirically derived using small scaled test specimens manufactured in the laboratory. In the field of building engineering, few experimental tests have been conducted on the actual reinforced concrete (RC) members of buildings constructed decades ago; however, full-scale loading tests were conducted on existing buildings by Osawa *et al*. (1968) and Matsushima (1970). Therefore, the research by Aoyama *et al*. (1983) and Araki *et al*. (2013, 2017) on the seismic performance of RC members obtained from old buildings is extremely valuable. In this study, the mechanical properties of concrete and the seismic performance of actual RC girders were investigated through experimental evaluations.

(a) (b) (c)

Figure 1. Existing building (a) comprehensive view of the test building; (b) concrete core boring; (c) cutting girder with a wire saw.

2 Existing Building

The building under investigation is a five-story RC building; it was constructed in 1961 and was used for residual purposes. Fig. 1(a) shows an image of the building. Its standard plan is star shaped, which was very popular in Japan between 1956 and 1964. The design of this building was based on the old structural code established by the Architectural Institute of Japan (AIJ) in 1958; the building's poor seismic performance could be attributed to a low amount of shear reinforcement. To estimate the mechanical properties of concrete, concrete cylinders were obtained from the first, the fourth, and the fifth stories by concrete core cutter when the building was demolished in 2017 as shown in Fig. 1(b). Additionally, two girders were obtained from the roof floor without any damage using a wire saw as shown in Fig.1(c).

3 Experimental Procedure

The material tests were performed using concrete cores and steel bars and seismic tests for the girders were performed to check the validity of the present equations used in the standard.

3.1 Materials

The mechanical properties of the concrete in existing RC buildings are directly related to the seismic performance of the structures and are important for seismic evaluations. Therefore, compressive and splitting tensile tests were performed. Table 1 summarizes the mechanical properties of concrete. The average concrete strengths of the roof floor were 19.2 N/mm^2. The concrete's strength in the building was distributed over a wide range from 14.3 N/mm^2 to 28.4N/mm^2. Although the average compressive strength exceeded the specified concrete strength, the COV of the compressive strength (0.263) exceeded the applicable upper limit of 0.25 for the seismic evaluation of existing buildings, as shown in a previous study by Sezen *et al*. (2011). The tensile splitting strengths were distributed over a range: from 1.13 N/mm^2 to 2.85 N/mm^2. The average yield strength of the main bars ϕ19 and the stirrups ϕ9 obtained by the tensile tests were 325 N/mm^2 and 292 N/mm^2, respectively.

Table 1. Mechanical properties of concrete.

Floor level	Compressive strength [N/mm^2]	Modulus of elasticity [kN/mm^2]	Tensile strength [N/mm^2]
5F	19.2 (0.263)	17.9 (0.238)	1.72 (0.305)

(COV): Coefficient of variation

3.2 Test Girders

The sectional area of the girders in the structural drawing was 250 mm × 800 mm. The obtained girders were designed with shear span lengths of 1,200 mm, to investigate the validity of the shear capacity equation currently used for seismic evaluation. The original test girders were termed SG-1 and SG-2, respectively. The structural drawing indicated that the main and shear reinforcements were plain round bars (22 ϕ and 9 ϕ, respectively). Table 2 presents the details of the original test girders. Their flexural Q_{mu} and shear strengths Q_{su} were calculated using Eqs. (1) and (2), respectively, which are provided in the JDBPA standard (2001). The failure modes of the members are very important factors for the seismic performance of existing buildings; therefore, valuating the strength of the RC members is necessary.

$$M_u = 0.9at \cdot \sigma_y \cdot d$$
$$Q_{mu} = {}^{2M_u}\!/_{L} \tag{1}$$

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].

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

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 [N/mm^2], and j is the distance between the resultant internal forces (7/8d) [mm]. Eq. (2) denotes the minimum shear strength as empirically proposed by Arakawa (1960), 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 294 N/mm^2. This strength is recommended in the standard (JDBPA, 2001), because tensile tests for the reinforcing bars were not required in the seismic evaluation. The estimated concrete strength of 16.7 N/mm^2 was obtained by subtracting half the standard deviation of the concrete strength from the average value in accordance with the standard (JDBPA, 2001). Using Eqs. (1) and (2), the ratio of Q_{su} 314 kN to Q_{mu} 186 kN of the original test girder was 1.66. 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 reinforcements for anchorage before casting concrete for the 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. 2 presents the details of the test girder.

Table 2. Details of the test girders.

Test Girder	Concrete (N/mm^2)	Section [mm]	Shear span [mm]	Main bar [SR24]	Stirrup [SR24]
SG-1	16.67	250 × 800	1200	2-19ϕ	2-9ϕ@250
SG-2				Pg = 0.61%	Pw = 0.20%

Specified concrete strength in the structural draft Fc: 17.6 [N/mm^2] （180kg/cm^2）

3.3 Loading and Measurement of Girders

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 both the girders. A pantograph system was used to ensure that the top and bottom stubs remained parallel during the reverse loadings. Fig. 3 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, 17 LVDTs were mounted on one side of the test girder. Finally, the lateral load was measured using a load cell.

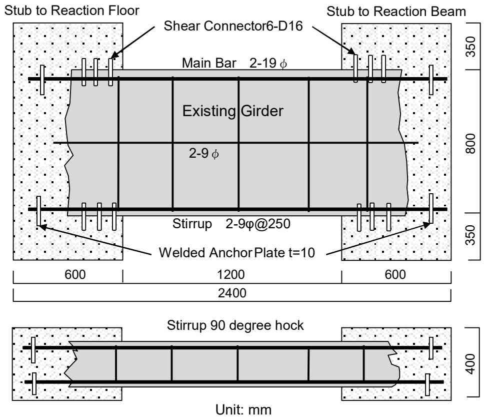

Figure 2. Dtails of the test girder.

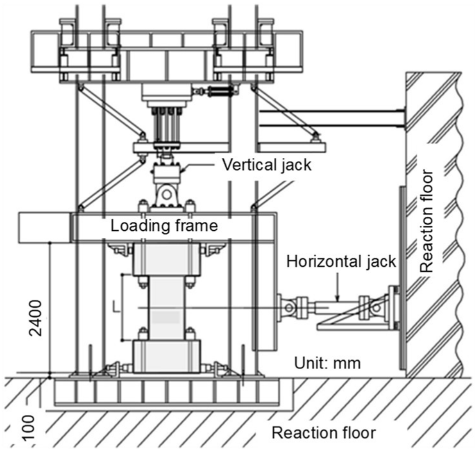

Figure 3. Test setup.

3.4 Retrofitting

Epoxy resin was injected into one of the original girders before loading to investigate the retrofitting effect. The injected epoxy resin filled the vacant space in the concrete and bonded the concrete and the reinforcing bars together. The epoxy resin injection alone may improve the seismic performance of moderately damaged buildings, although epoxy resin injection is usually associated with wrapping steel plates or CFRP sheets. In contrast to the conventional

(a) (b) (c)

Figure 4. Retrofitting process of test girder SG-2: (a) perforation; (b) sealing and attachments for capsules; (c) epoxy resin injection with capsules.

method in which epoxy resin injected at the concrete surface of the members, epoxy resin was injected with spring capsules at the position of the reinforcing bar or at the distance of 50 mm from the concrete's surface. The epoxy resin was also injected at the location of deficiencies as honeycombs. Fig. 4 presents the retrofitting process. The total amounts of epoxy resin injected into the original test girder SG-2 was 5.73 kg.

4 Experimental Results

4.1 Crack Patterns

The figures below show the location the slabs on the right side of the girders. Slight flexural cracks 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 started to occur around the small openings; then, the shear cracks appeared throughout the entire girders under increasing controlled displacement until drift angle R = 1/50 rad. The specific width of the shear cracks increased, whereas the flexural cracks did not progress. Shear cracks of 45 degrees were observed at both ends of the test girder SG-1, and diagonal shear cracks were observed in the retrofitted test girder SG-2. The final collapse mechanisms were the shear failure type. Fig. 5 (a) and (b) illustrate the crack patterns of the test girders at the final stage at drift angle R = 1/30 rad.

(a) SG-1 (b) SG-2

Figure 5. Crack patterns at final stages.

4.2 Shear Force and Drift Angle Response

Fig. 6 depicts the relationships of the shear force *Q* with drift angle *R* in both the test girders. The calculated flexural and shear strengths of the original girders were 186 kN and 314 kN, based on the standard. The stiffness degradations for both the test girders were observed at the first loading cycle. In both girders, the peaks of the shear forces were measured at drift angle R = 1/200 rad. The maximum shear forces exceeded the calculated flexural strength and did not reach the calculated shear strength. Although the shear cracks progressed, 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 concrete may have occurred because the reinforcement was a plain round bar, and the concrete strength was low. No significant degradation of the shear force *Q* was observed until drift angle R = 1/50 rad. The maximum strength of the retrofitted girder (297

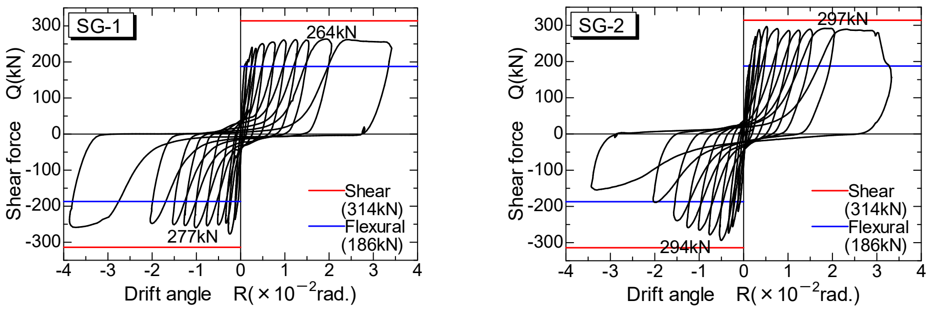

Figure 6. Shear force and drift angle response.

kN) slightly increased in comparison with those of the original test girder (277 kN). Those values were much greater than the flexural strength calculated with Eq. (1) using the recommended material strength based on the standard (JBDPA, 2001).

5 Discussions

5.1 Strength

5.1.1 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 broad experimental data on RC members. The tensile stress σ_T was recommended by Collins *et al*. (1991).

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

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

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, an average concrete strength of 19.2 N/mm^2 was used. The comparisons of the observed and calculated strengths of the shear cracks are shown in the first half of Table 3. The shear crack strength as calculated by Eq. (5) was underestimated, while that calculated by Eq. (6) was overestimated.

5.1.2 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 pieces for the tensile test were taken out from the test girders after loading. Bar arrangements in the girders were inspected by removing the concrete cover after the loadings. The stirrups (9ϕ)

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

were arranged with a 300 mm - 600 mm space, unlike the structural draft, and a 600 mm space was used in the calculation. The empirical equation for the shear strength was used in this study. Eq. (7) expresses the mean values of the test results taken from previous studies on RC members. This equation was also proposed by Arakawa (1960) and was used for the direct comparison of the observed maximum strength in shear failure. The observed and calculated values for maximum strength are summarized later in Table 3. The predicted maximum strengths, as calculated by Eq. (1), were consistent with the observed values of the original girders. The maximum strengths of the girders were 277 kN and 297 kN, respectively. Those maximum values were 7% and 15% greater than the flexural strengths calculated by Eq. (1). However, the researchers noted that the crack patterns of both girders through the loadings were mainly shear failure modes.

5.2 Effect of Retrofitting

The maximum strength of the retrofitted girder was 1.07 times that of the original girder, although the girders before repair contained some honeycombs. Researchers estimated that the epoxy resin injected into the honeycombs or around the main bars protected the progress of the flexural failure or the bond slippage. Fig.7 shows the comparisons between the observed envelope curves of the shear force responses and the shear strength calculated by Eqs. (1) and (7).

Table 3. Lists of strength.

Test Girder	Cracking strength [kN]			Maximum strength [kN]		
	Obs.	Eq.(5) V_c	Eq.(6) Q_{sc}	Obs.	Eq.(7) Q_{su}	Eq.(1) Q_{mu}
SG-1	198	193 (1.03)	257 (0.77)	277	360 (0.77)	259 (1.07)
SG-2	209	193 (1.08)	257 (0.81)	297	360 (0.83)	259 (1.15)

* ( ): Observed/Calculated

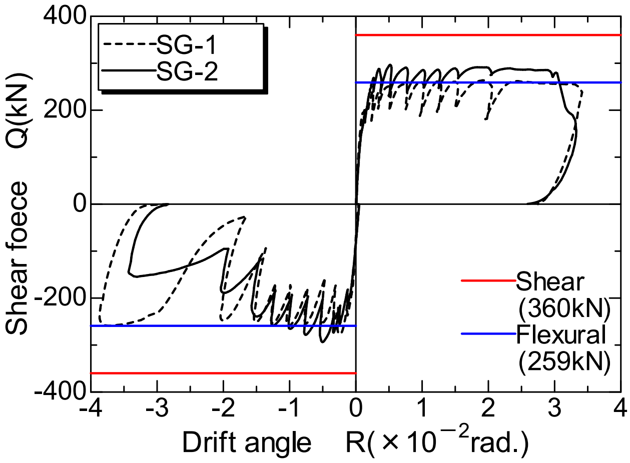

Figure 7. Envelopes of shear force and drift angle response.

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 1961:

- Material tests revealed that eight concrete cylinders obtained from each floor of the building exhibited average compressive strengths of 19.2 N/mm^2 and an average tensile splitting strength of 1.72 N/mm^2.
- The failure modes of the 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.
- The predicted flexural strengths were consistent with the observed maximum values.
- Epoxy resin injection improved the seismic performance of the RC girders.
- The researchers noted that the crack patterns of both the girders through the loadings were mainly shear failure modes.

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

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