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SHEAR-FLEXURAL BEHAVIORS OF STAINLESS STEEL REINFORCED CONCRETE COLUMNS

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ABSTRACT

An experimental investigation was performed to examine the shear-flexural behaviors of stainless steel reinforced concrete columns under cyclic lateral loading. The specimens had a cross section of 300mm×300mm, 900mm or 1200mm clear span with stubs at both ends, 1.07 or 2.09% flexural reinforcing ratio and 0.477% lateral reinforcing ratio. The normal stress applied to the specimens was 1/4 of the concrete strength. The shear-span ratios were 1.5 and 2.0. The shear strength of the specimens was calculated by the Ohno-Arakawa equations and the specimens were designed to reach their shear strength before the ultimate flexural strength. The experimental results showed as follows: The shear strength of the stainless steel reinforced concrete columns can be calculated by the Ohno-Arakawa equations except for the bond failure mode. The ratios of the experimental shear strength to the calculated values are from 0.93 to 1.07 using the proof stainless steel yield strengths determined as the stress at which plastic strain equals 0.2% strain. The crack patterns and deformation behaviors are different from those of carbon steel reinforced concrete columns.

Introduction

The use of carbon steel does not always guarantee sufficient resistance to corrosion in severe environments. Although many experimental investigations of corrosion of carbon steel have been done (Li 2002, Paulson 2002), perfect protection methods against steel corrosion has not yet been confirmed. To avoid corrosion, stainless steels can be used. In recent decades stainless steel reinforcements have been used in new construction and in repair work on existing structures to provide sufficient resistance to corrosion. Technical reports and standards on stainless steel have also been compiled (The Concrete Society 1998, ASTM 2001, BS 2001). However, the information for designing and constructing stainless steel reinforced concrete structures is not sufficient. It is necessary to know the structural performance of stainless steel reinforced concrete members. An experimental investigation was performed to examine the shear-flexural behaviors of stainless steel reinforced concrete columns.

Specimens

The specimens are shown in Table 1 and Fig.1 (Shear-Span Ratio M/QD=1.5). The specimens had a

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Table 1. Specimen details.

Specimen	Bar arrangement				bxD	L	M/QD	N
	Longitudinal		Lateral					
		p_t(%)		p_w(%)	(mm)	(mm)		(MPa)
No.1	12-D16	1.07	D10@100	0.477	300	900	1.5	80
No.2	12-D22	2.09	D10@100	0.477	300	900	1.5	80
No.3	12-D16	1.07	D10@100	0.477	300	1200	2.0	80
No.4	12-D22	2.09	D10@100	0.477	300	1200	2.0	80

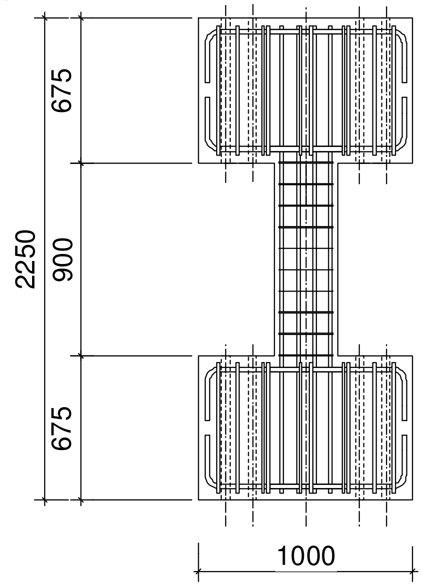

Figure 1. Specimen (**M** / **QD** = 1.5).

cross section of 300mm × 300mm, 900mm or 1200mm clear span, 1.07 or 2.09% flexural reinforcing ratio and 0.477% lateral reinforcing ratio. The normal stress applied to the specimens was 1/4 of the concrete strength. The shear-span ratios were 1.5 and 2.0. The shear strength of the specimens was calculated by the Eq. 1 (AIJ 1999) and the specimens were designed to reach their shear strength before the ultimate flexural strength. The calculated strengths of the specimens are shown in Table 2. For reference, the bond strength of the specimens was calculated by the empirical equation (Tabata 1995).

$$\mathbf{Q_u} = \left\{ \frac{0.12\mathbf{k_u} \cdot \mathbf{k_p}(180+\mathbf{F_c})}{\mathbf{M}/\mathbf{Qd}+0.12} + 2.7\sqrt{\mathbf{p_w}\boldsymbol{\sigma}_{\mathbf{wy}}} + 0.1\boldsymbol{\sigma}_{\mathbf{o}} \right\} \mathbf{b} \cdot \mathbf{j} \tag{1}$$

where $\mathbf{Q_u}$ =shear strength (kgf); $\mathbf{k_u}$ =coefficient of effective depth; $\mathbf{k_p} = \mathbf{p_t}^{0.23}$; $\mathbf{F_c}$ = concrete strength (kgf/cm^2); **M**/**Qd** =shear span ratio; $\mathbf{p_w}$ = lateral reinforcing ratio; $\boldsymbol{\sigma}_{\mathbf{wy}}$ =yield point of lateral reinforcing steel (kgf/cm^2); $\boldsymbol{\sigma}_{\mathbf{0}}$ = axial stress (kgf/cm^2)

Table 2. Calculated strength of specimens.

Specimen	M/QD	N (MPa)	Qsu (kN)	Qbu (kN)	Qmu (kN)	Qsu/Qmu	Qsu/Qmu
No.1	1.5	80	300	176	375	0.80	0.47
No.2	1.5	80	328	181	473	0.70	0.38
No.3	2.0	80	259	176	280	0.92	0.63
No.4	2.0	80	280	181	354	0.79	0.51

Table 3. Mix proportions and mechanical properties of concrete.

W/C (%)	Cement (kg/m^3)	Water (kg/m^3)	Slump (mm)	Age (weeks)	Fc (MPa)	Ec (GPa)
55	324	178	180	7	33.4	30.0

Table 4. Mechanical properties of stainless steel.

Type	Use	Shape	Fy** (MPa)	Ft (MPa)	Elongation (%)
SUS304*	Longitudinal	D22	364	682	45.0
		D16	412	654	52.0
	Lateral	D10	395	655	40.6

* Nickel 8.11%, Chrome 18.4%, Carbon 0.05% **0.2% Proof strength

Materials

The mix proportions and mechanical properties of the concrete are shown in Table 3. The concrete strength of the specimens was assumed to be the same as the test pieces taken at the time of casting and cured in the sealed condition. The strength of the concrete was 33.4MPa at the experiment. The mechanical properties of stainless steels are shown in Table 4. Main longitudinal and lateral stainless steel reinforcement were D16 or D22 and D10, respectively. The yield strength of the stainless steel was determined as the stress at which plastic strain equals 0.2% strain.

Test procedures

The loading and measuring systems are shown in Fig. 2. The lateral load was applied to the specimens

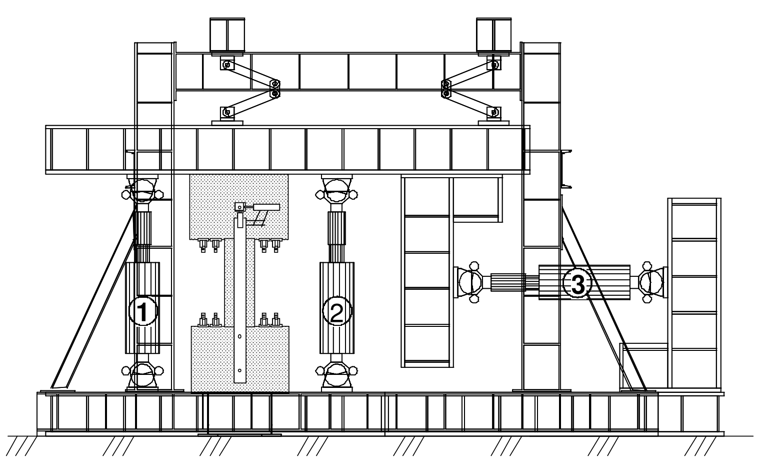

Figure 2. Loading system (①, ②:axial load, ③:lateral shear load).

under a control of relative displacement angles (R) between two end stubs: single cycle at R=1/400rad, two cycles at R=1/200rad, 1/100rad and 1/50rad, single cycle at 1/25rad and monotonically loaded to 1/20rad as a general rule. Wire strain gauges were put on the three sides of the longitudinal bars at six points in the test span and four points in the stubs. Wire strain gauges were also put on lateral reinforcement at six points. The zero initial readings of the strains were made just before the axial stress loading with no lateral load.

Results of experiment

Progress of experiment and cracking

Photo 1 shows No.4 Specimen loaded at the relative displacement angle **R** = 1/25. The crack patterns of the specimens at the final stage are shown in Fig. 3. In every specimen flexural cracks occurred at first and then flexural-shear, shear cracks occurred. In shear-span ratio **M/QD** = 1.5, diagonal shear cracks connecting two end stabs were observed. In **M/QD** 2.0, many shear cracks were observed and vertical side split cracks caused by bond failure were distinguished in No.4 Specimen. All the specimens

Photo 1. No.4 Specimen loaded at the relative displacement angle **R** = 1/25

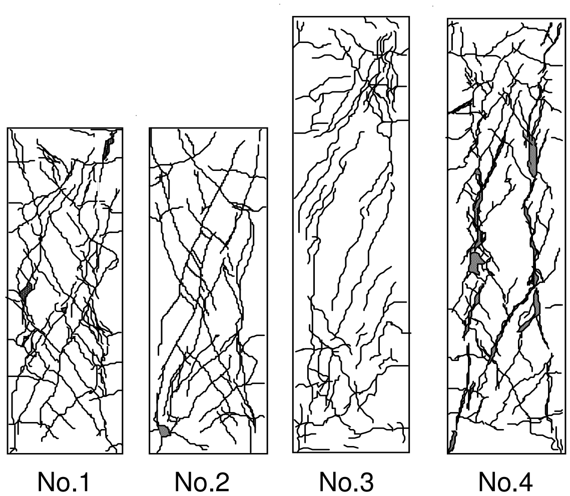

Figure 3. Final crack patterns of column specimens

supported the axial load up to final stage of the lateral loading except for No.1 Specimen which failed at **R** = 1/50. The results of experiment are shown in Tables 5 and 6.

Load-displacement relationship and failure mode

The relationships between shear load and the relative displacement of the specimens are shown in Fig. 4. No.1 specimen failed in shear at **R** = 1/100 just after the diagonal shear cracking enlarged. The side

Table 5. Cracking and maximum strength.

Specimen	L	Cracking		Qe : Maximum strength			
		Flexural	Shear	Strength	Displacement	Bar strain ($\times10^{-6}$)	
	mm	(kN)	(kN)	(kN)	(rad)	Main	Lateral
No.1	900	98.1	252	322	+1/100	2097	4348*
No.2	900	98.1	254	305	+1/200	1152	2110
No.3	1200	99.1	234	264	+1/100	3073	1644
No.4	1200	78.5	232	300	+1/100	1401	3983

Table 6. Results of experiment.

Specimen	Limit disp.** (rad)	Maximum bar strain ($\times10^{-6}$)				Qe/Qsu	Qe/Qbu	Qe/Qmu
		Main	(rad)	Lateral	(rad)			
No.1	1/100	2097	1/100	11200*	1/50	1.07	1.82	0.859
No.2	1/100	1152	1/200	4479*	1/50	0.93	1.68	0.645
No.3	1/50	3073	1/100	5063*	1/50	1.02	1.49	0.941
No.4	1/100	1401	1/100	4786*	1/50	1.07	1.65	0.848

Exceeded yield strain, ** Displacement at 0.8 of the maximum strength in descending part

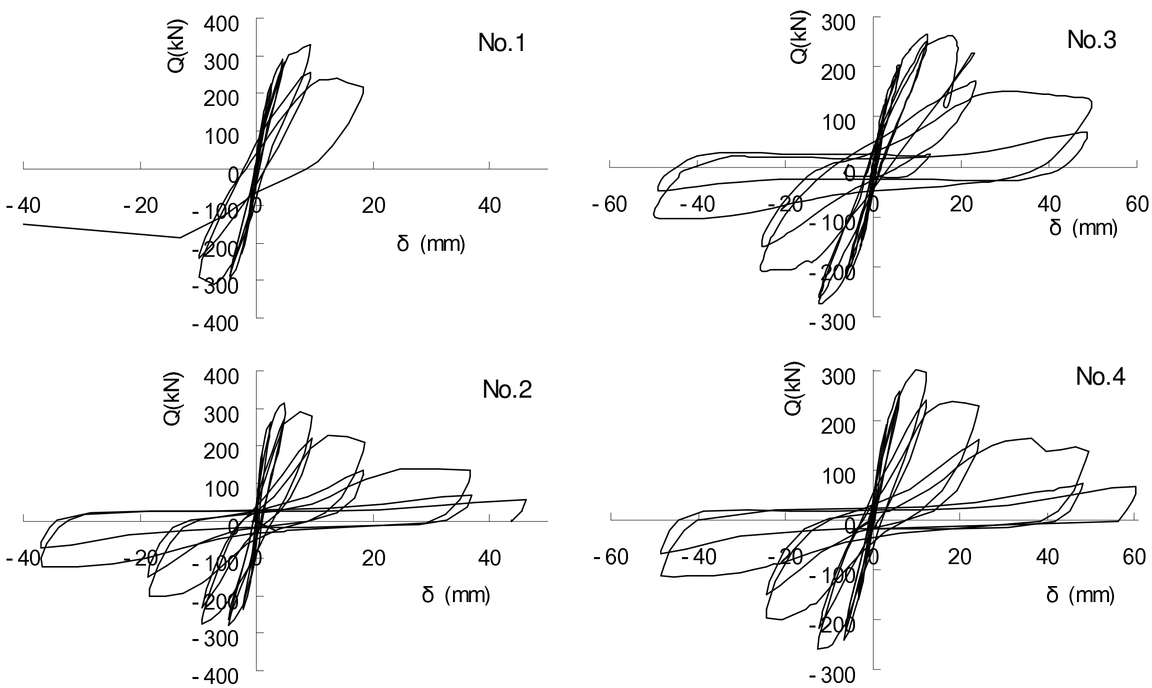

Figure 4. Load-displacement relationship.

split cracking caused by bond failure appeared at this stage. At the final stage of the experiment the cover concrete fell out. The maximum strength of No.2 Specimen developed at **R** = 1/200 and the diagonal shear cracking occurred at **R** = 1/100 and the side split cracking appeared remarkably. It is supposed that the gradual decrease of the strength compared with No.1 Specimen is due to the complex failure modes of shear and bond. The maximum strength of No.3 Specimens developed at **R** = 1/100. It is assumed that the limit displacement is the displacement at 0.8 of the maximum strength in descending part of the load-displacement skeletons. The limit displacement of No.3 Specimen was **R** = 1/50 and showed somewhat ductile behaviors due to large tensile strain of the longitudinal bars. The maximum strength of No.4 Specimen developed at **R** = 1/100. After this displacement the side split cracking appeared remarkably.

Load-strain of stainless steel reinforcement relationship

Fig. 5 shows the load-strain relationships of the longitudinal bars at lower and upper column portion of No.3 Specimen up to about maximum strength. The tensile strains of main longitudinal bars of all the specimens did not exceed the proof yield strains. Because the strain of No.3 Specimen exceeded 3000×10^{-6}, the displacement at maximum strength was **R** = 1/100. Fig. 6 shows the load-strain relationships of the lateral bars at lower and upper column portion of No.3 Specimen. The strains of lateral reinforcement of all the specimens at maximum strength did not exceed the proof yield strains except for No.1 Specimen which failed in shear.

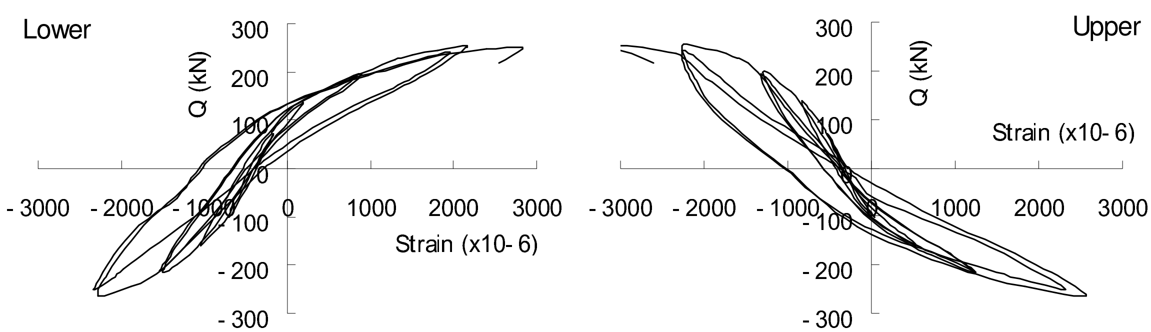

Figure 5. Load- strain relationships (No.3 Specimen longitudinal bar).

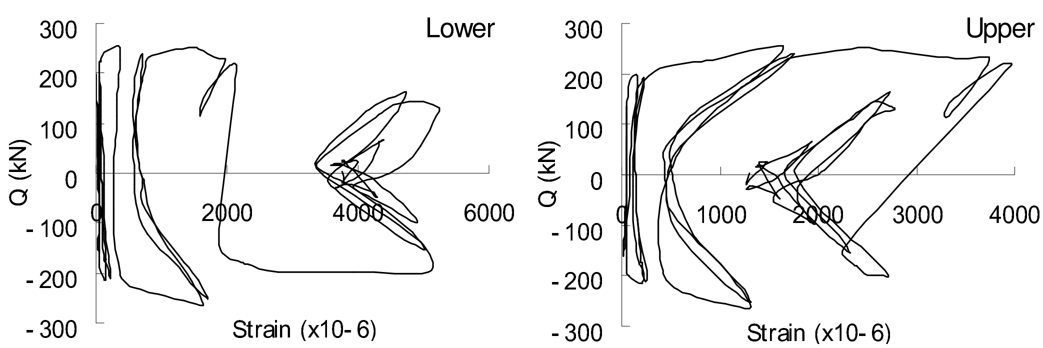

Figure 6. Load- strain relationships (No.3 Specimen lateral bar).

Conclusions

The results of experiment are summarized as follows.

1. The shear strength of the stainless steel reinforced concrete columns can be calculated by the Ohno-Arakawa equations. The ratios of the experimental shear strength to the calculated values are from 0.93 to 1.07 using the proof stainless steel yield strengths determined as the stress at which plastic strain equals 0.2% strain.
2. It was suggested that the bond failure mode should be considered to predict shear-flexural behaviors of stainless steel reinforced concrete columns.
3. The crack patterns and deformation behaviors were somewhat different from those of carbon steel reinforced concrete columns.
4. The deformation behaviors of columns were affected by the mechanical properties of stainless steel reinforcement.

Appendix

Flexural strength of stainless steel reinforced concrete beam

It is important to know the basic flexural behaviors of stainless steel reinforced concrete members. In the appendix an experiment of stainless steel reinforced concrete beam test is shown (Yamamoto 2005).

Specimen and materials

The specimen is shown in Fig. 7. The specimens had a cross section of 180mm × 300mm, 1750mm length, 1.74% flexural reinforcing ratio and 0.53% lateral reinforcing ratio. Stainless steel reinforcement was used for specimen S-1. Carbon steel reinforcement was used for specimen N-1. The strength of the concrete was 24.4MPa at the time of the loading test. Austenitic stainless steel SUS304 (Japanese Industrial Standard) was used. The yield points of D22 longitudinal reinforcing steels were 365MPa and 370MPa for stainless steel and carbon steel, respectively. Because stainless steels do not exhibit a well-defined yield point, the proof strengths were determined as the stress at which the plastic strain equals 0.2% strain. The yield strengths of stainless steels and carbon steels are almost the same.

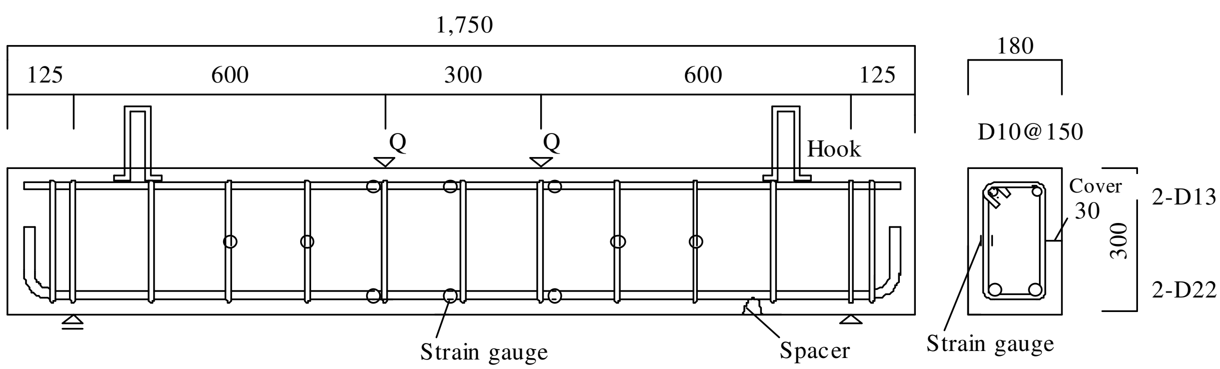

Figure 7. Beam specimen.

Flexural strength and displacement

The specimens failed in strength after the longitudinal bars yielded. The experimental strength of the specimens exceeded the calculated values, and the ratios between the experimental and calculated strengths were 1.10 and 1.03 for stainless steel and carbon steel reinforced beam specimens, respectively. The ratios of the stainless steel reinforced specimens were higher than that of the carbon steel. Fig. 8 shows the load-displacement relationship and Fig. 9 shows the load-strain relationship. The displacement curves represent the mechanical properties of the stress-strain relationships of stainless steel and car-bon steel. Stainless steel reinforced specimen did not exhibit clear yielding unlike the carbon steel reinforced concrete specimen. The relative deflection angle (R) at yielding of stainless steel reinforced specimen was R=1/120, and that of carbon steel reinforced specimen was R=1/170.

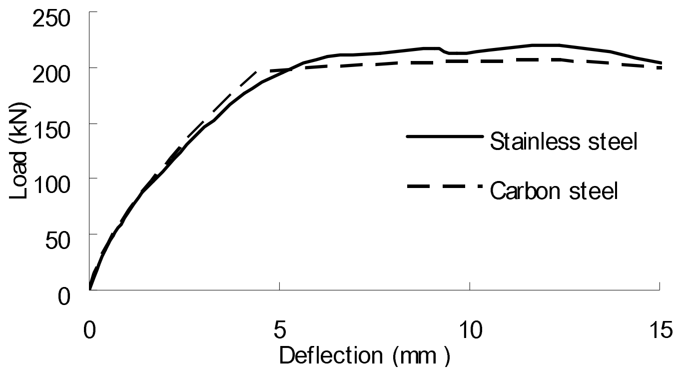

Figure 8. Load-displacement relationship.

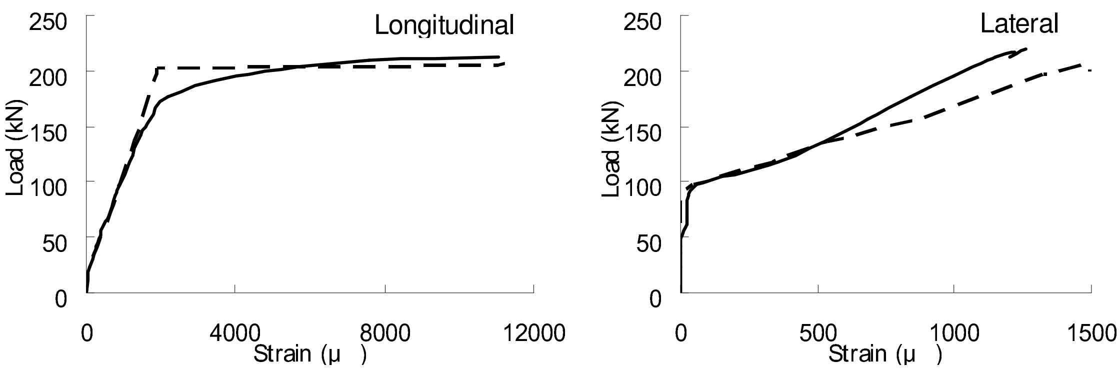

Figure 9. Load-strain of reinforcing bars relationship.

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