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STRENGTH AND DEFORMATION CHARACTERISTICS OF SRC FRAMES HAVING WEAK AXIS BENDING COLUMNS

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SUMMARY

An experimental work of SRC (Steel encased Reinforced Concrete) frames with weak axis bending columns was performed. Total four specimens were subjected to constant vertical load and cyclic horizontal load. As the experimental parameters, the column length to section depth ratio and axial load ratio were selected. From the test result, the experimental strength was compared with AIJ (Architectural Institute of Japan) design formula. In addition, the collapse mechanism of specimens, the hysteresis characteristics and the deformation capacity were investigated and discussed.

INTRODUCTION

Steel-concrete composite structures are being widely used for the building in Japan now. And by the development of the industrial technology in recent years, high-strength concrete and steel are used as a building material. Therefore, a column member should become slender.

In AIJ design formula, the strength of slender steel-concrete composite column is calculated by superposed method considered additional bending moment. However, the design method takes no consideration about deformation capacity. To evaluate the deformation capacity, it is necessary to establish proper hysteresis model for steel-concrete composite member.

From the above, an experimental study on SRC frame was performed. The objectives of the study are to examine the AIJ design formula and to evaluate the deformation capability of the composite column.

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TEST PROGRAM

1. Specimen

Total four SRC specimens were tested to investigate the elastic-plastic behavior of composite frame. The specimens were subjected to constant vertical load and cyclic horizontal load. The encased H-section column was weak-axis bending.

When performing the experimental study, two parameters were chosen. One was L/D (column Length to section Depth ratio) of 12and 6, the other was axial load ratio (N/BDF_c=0.1, 0.5 (N: axial load, B: width of a column, F_c: compressive strength of concrete)).

The dimensions and properties, and the details of specimens are shown in Table 1 and Fig.1. The clear-height of columns is 1800mm or 900mm.

The columns, with square section of 150x150mm, are reinforced with 4-D6 (D: deformed bar, SD295A) longitudinal bars and 4ϕ hoops (@100mm). The encased H-section designed as H-100x50x5x7 is mild steel and is placed so as to be subjected to weak axis bending. The area ratio of the steel section to the section of the column is about 4.6%.

The beams, with section of 150x160mm, are reinforced with 8-D10 (SD295A) bars and 4ϕ stirrups (@50mm). The encased H-section is placed so as to be subjected to strong axis bending.

The specimens were designed so that the columns were collapsed earlier than the beams. Therefore, the number of stirrups was arranged in the beams. The main reinforcements were welded to the end plate.

All specimens are named such as SRC-06-01. In this case, 06 means L/D (the column Length to section Depth ratio) and 01 means vertical load ratio.

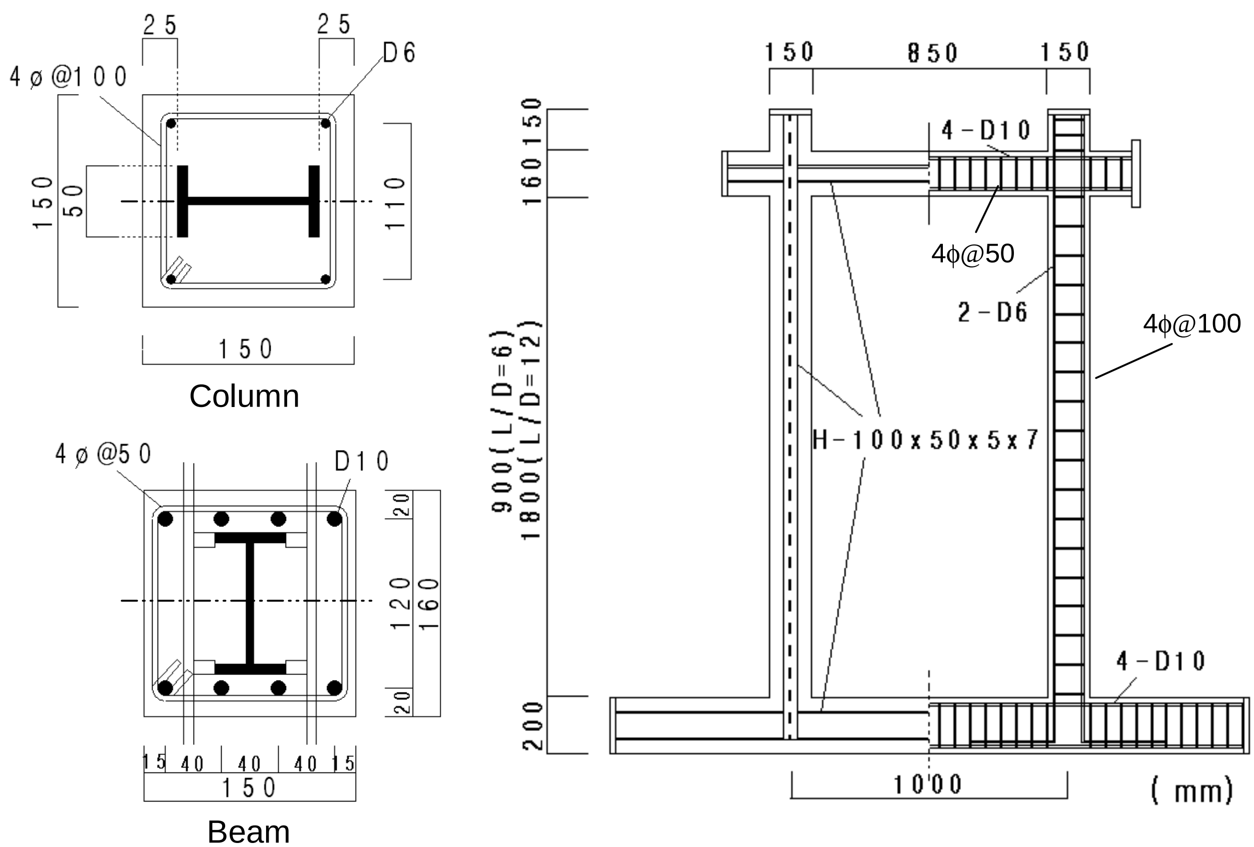


Fig. 1 Details of specimens

Table 1. Properties and Size of specimens

			b(mm)	D(mm)	$_mp$(%)	$_sp$(%)	$_cE$(Gpa)	F_c(Mpa)	N(kN)	n	n'	batch
SRC-06-01	L/D=6	Column	152	150	0.49	4.65	26.1	40.5	182	0.07	0.1	A
		Beam	148	160	2.20	4.48						
SRC-06-05		Column	152	149	0.49	4.68	23.2	24.9	564	0.29	0.5	B
		Beam	153	164	1.96	4.22						
SRC-12-01	L/D=12	Column	157	150	0.47	4.50	24.1	35.4	182	0.07	0.1	A
		Beam	149	159	2.19	4.45						
SRC-12-05		Column	153	151	0.48	4.60	21.8	24.5	564	0.29	0.5	B
		Beam	152	163	1.98	4.26						

b: width of cross section, D: depth of cross section, $_mp$: reinforcement ratio ($_mA$/bD, $_mA$: gross area of rebars)
$_sp$: steel ratio ($_sA$/bD, $_sA$: area of H-section), $_cE$: modulus of elasticity of concrete, F_c: compressive strength of concrete,
N: applied axial load, n: axial load ratio, n': N / 2F_cbD

Specimen: SRC-L/D-n’

2. Mechanical properties

The properties of concrete such as the modulus of elasticity and the compressive strength are shown in Table 1. The mechanical properties of steel obtained from the coupon test are shown in Table 2 and the size of H-section is summarized in Table 3.

Table 4 shows the mix proportion of concrete. The design strength of concrete was 27MPa. The strength of concrete shown in Table 1 was the result of compressive test using cylinders (100ϕx200mm) at test age of the specimen. The maximum aggregate diameter was 15mm.

Table 2. Mechanical properties of steel

		diameter ϕ(mm)	Young's Modulus $_sE$(GPa)	yield point $_s\sigma_Y$(MPa)	tensile strength $_s\sigma_u$(MPa)	yield strain ε_Y($_s\sigma_Y$/$_sE$)	yield ratio $_s\sigma_Y$/$_s\sigma_u$	elongation (%)
flange			206	345	471	0.00168	0.733	38.2
web			206	340	468	0.00165	0.726	34.6
D10	A	9.10	204	396	544	0.00194	0.727	19.1
	B	8.84	189	472	620	0.00250	0.761	20.3
D6		5.93	214	473	582	0.00221	0.813	18.4
4ϕ	A	3.95	205	592	608	0.00289	0.974	15.4
	B	3.97	203	589	597	0.00291	0.987	13.4

Table 3. Size of H-section

height H (mm)	width B (mm)	flange thickness t_f (mm)	web thickness t_w (mm)
99	49.4	6.57	4.77

Table 4. Mix proportion of concrete

	weight per unit volume (kg/m^3)						W/C	S/a	F	G_{MAX}	Slump
	C	W	S1(sea)	S2(crushed)	G	AE agency	(%)	(%)	(MPa)	(mm)	(cm)
A	430	224	356	356	827	0.86	52.1	46.6	27	15	18
B	400	200	364	364	900	0.8	50.0	45.1			

F: design strength of concrete, G_{MAX}: maximum diameter of coarse aggregate, S: S1+S2

3. Loading Apparatus

The loading program is shown in Fig.2. The amplitude of story drift angle of the frame was increased in increment of 0.005rad.

The experimental apparatus is shown in Fig.3. Through the Loading beam, the constant vertical load was applied on the specimen by hydraulic jack 1, and kept constant during horizontal loading process. The cyclic horizontal load was applied to the specimen by hydraulic jack 2. And the applied forces were measured by load cells.

Fig.4 shows the position of measurement. Horizontal displacements were measured using displacement transducers at the center of the beam. Axis displacements were measured by displacement transducers at the top of the both columns and 1.5D (D: depth of column) below. The strain of concrete, encased H-section, and main reinforcements were measured by wire strain gauges placed at D apart from the face of column and beam.

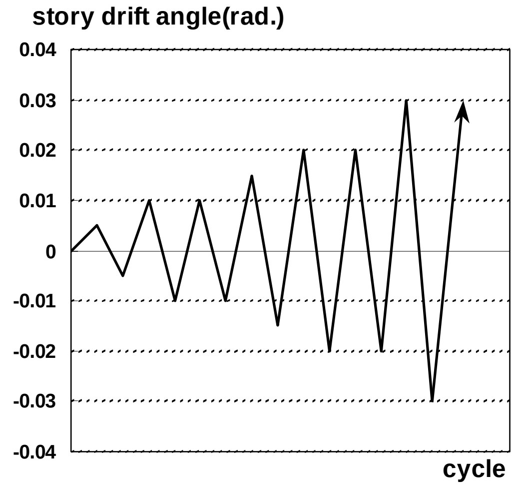


Fig. 2 Loading program

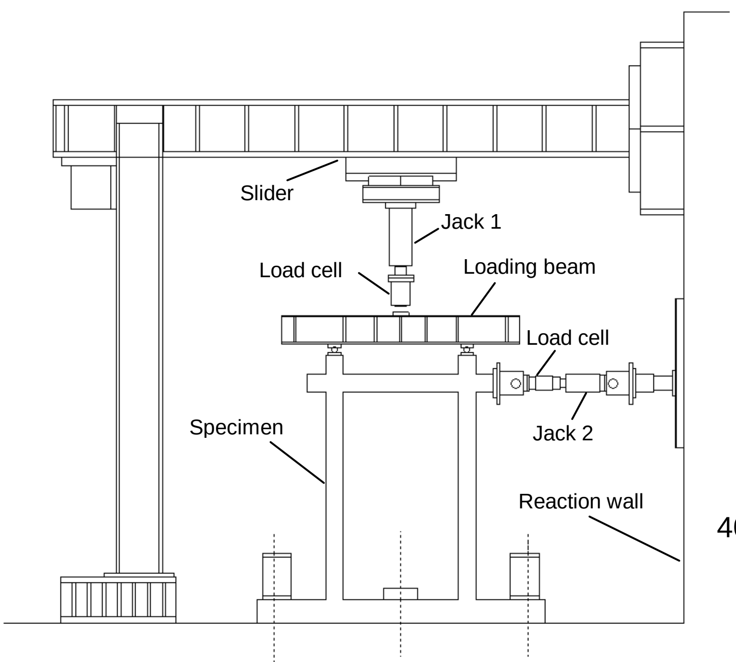


Fig. 3 Loading apparatus

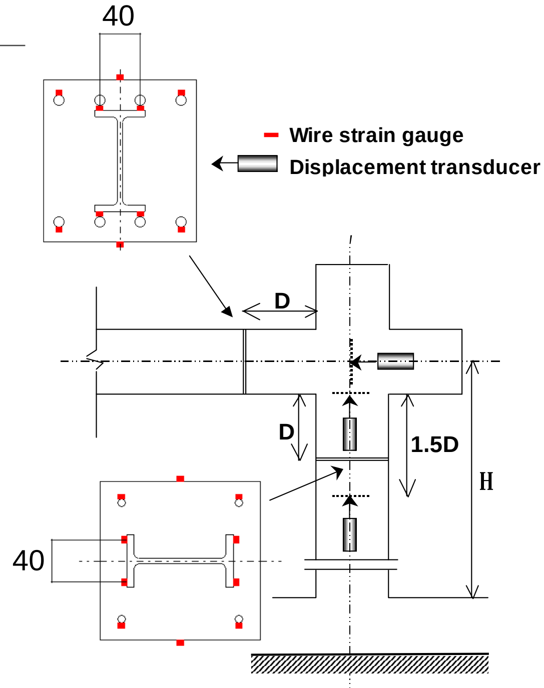


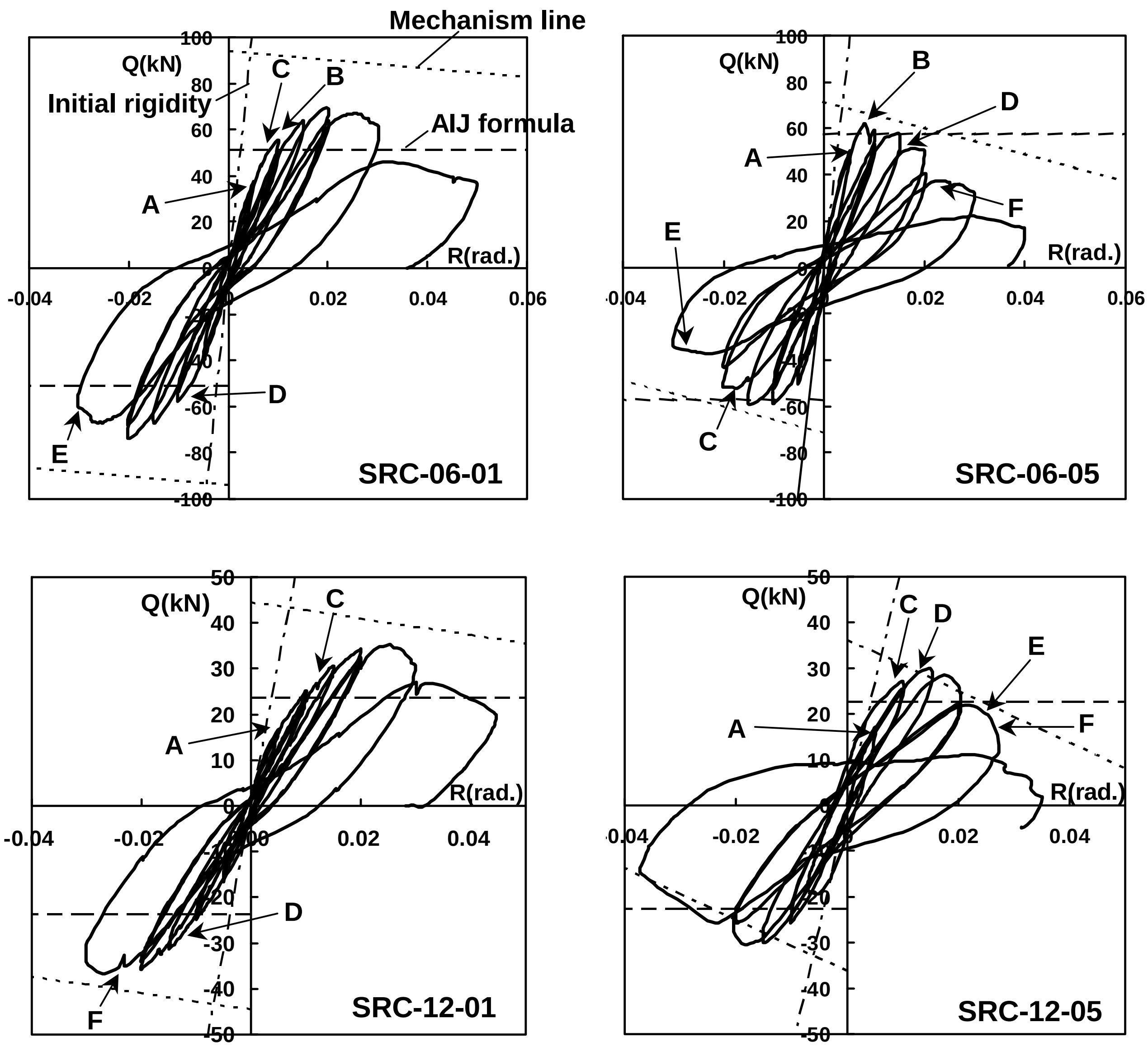
Fig. 4 Position of measurement

RESULTS AND DISCCUTION

1. Load-displacement relations

Fig.5 shows the Q-R relations of all specimens, where Q means the lateral force applied to the specimen and R means the story drift angle defined as the relative lateral displacement between top and bottom of the column divided by height of the column H (See Fig.4). Additionally, the stages observed flexural cracks and bond cracks are shown by marks A and B, respectively. And the stages that main reinforcements and H-section steel reached their yield strain are shown by C, D, E and F. In that case, the occurrence of the cracks was judged by eye observation and yielding of the steel materials by strain gauges.

In Fig.5, the solid lines show the experimental behavior. The chain lines show initial rigidity of the frame, which is calculated by disregarding the rigid zone and using linear-frame model. The dotted lines show the rigid plastic collapse mechanism line based on the fully plastic moment of the column cross section, which is calculated by assuming that a plastic hinges are formed at the surface of the beams (the



A : flexural cracks were observed **B** : bond cracks were observed
C : occurrence of main reinforcements at top of column yielding
D : occurrence of main reinforcements at bottom of column yielding
E : occurrence of steel at top of column yielding
F : occurrence of steel at bottom of column yielding

Fig. 5 Horizontal load - story drift angle relations

top and bottom of the columns). The dashed lines represent the strength that is calculated by AIJ formula for slender column.

In all of the specimens, the stable hysteresis properties were observed until their maximum strength was observed. And the yielding of H-section was observed in the final stage of all specimens.

The experimental strength of all specimens exceeds the strength calculated by AIJ design formula. Neither specimens excluding SRC-12-05 reached the rigid plastic collapse mechanism line. Two specimens whose axial load ratio is 0.1 showed their maximum strength at R=0.02rad. On the other hand, two specimens whose axial load ratio is 0.5 showed their maximum strength (at about R=0.01). After peak, the strength of all specimens deteriorated larger than the mechanism lines.

2. Process of collapse

The final crack patterns of the test specimens are illustrated in Fig.6.

The L/D=12 specimens collapsed flexural failure. On the other hand, the L/D=6 specimens collapsed shear bond failure that was different from that assumed in AIJ design formula.

In case of the L/D=12 specimens, flexural cracks were observed at the top and bottom of the columns (R=0.005 rad.). Subsequently, the number of cracks increased as the deformation was increased. The SRC-12-01 specimen reached maximum strength at R=0.025rad. and the SRC-12-05 specimen has already reached at R=0.01rad. And after that, crush of concrete section was observed at R=0.03rad in both specimens.

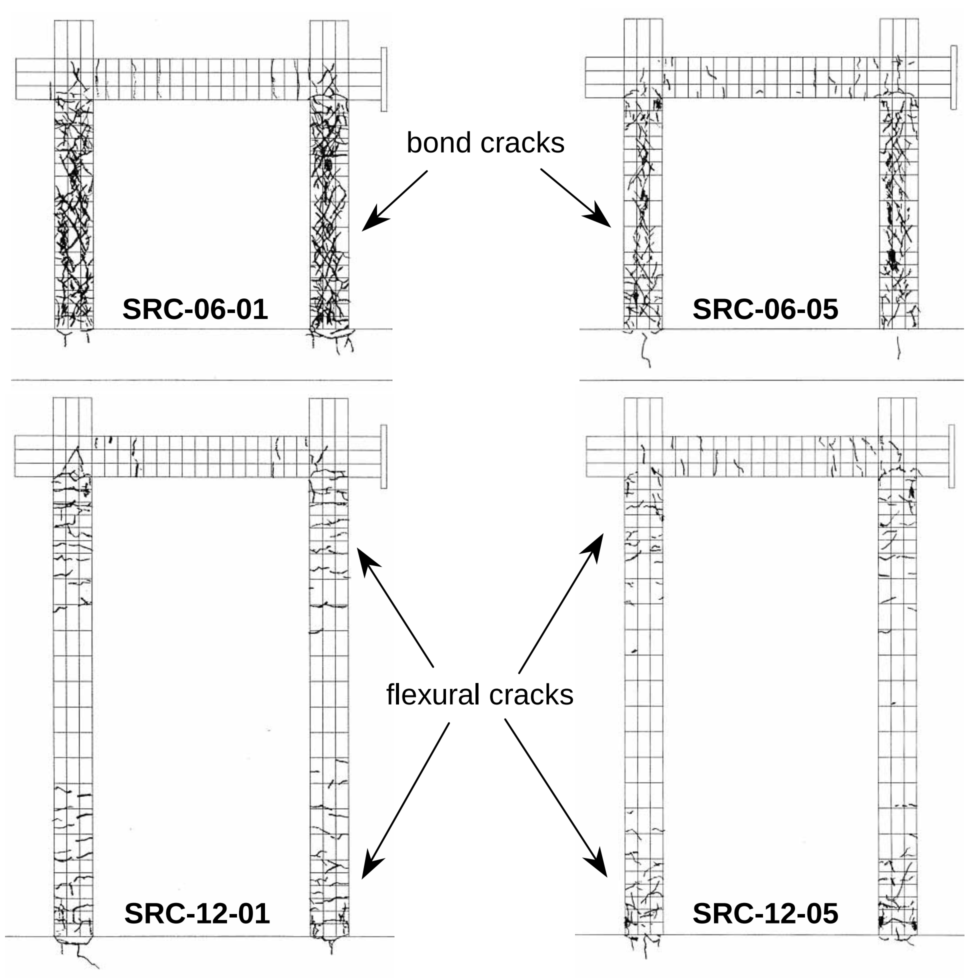


Fig. 6 Crack pattern (final stage)

In case of the L/D=6 specimens, flexural cracks observed at R=0.005rad. At R=0.01rad, bond cracks between steel and concrete was observed. After that, the cracks have progressed fast. Finally, the cracked cover concrete fell off gradually.

3. Comparison of strength between Test and AIJ formula

In AIJ design formula, the strength of slender composite column is calculated by Equation 1, where a conventional equation of simple superposition is modified. The strength of slender composite column is obtained as larger value of the two superposed strength. One is preferable for bending about the strong axis of steel member, the other one is preferable for bending about weak axis.

preferable for bending about the strong axis

When $N_U \leq {}_rN_{cU}$ or $M_U \geq {}_sM_{U0}(1 - {}_rN_{cU} / N_k)$

$$N_U = {}_rN_U \,,\quad M_U = {}_sM_{U0}\left(1 - \frac{{}_rN_{cU}}{N_k}\right) + {}_rM_U$$

When $N_U \geq {}_rN_{cU}$ or $M_U \leq {}_sM_{U0}(1 - {}_rN_{cU} / N_k)$

$$N_U = {}_rN_{cU} + {}_sN_U \,,\quad M_U = {}_sM_U\left(1 - \frac{{}_rN_{cU}}{N_k}\right) \qquad \textbf{(1-1)}$$

preferable for bending about the weak axis

When $N_U \leq {}_sN_{cU}$ or $M_U \geq {}_rM_{U0}(1 - {}_sN_{cU} / N_k)$

$$N_U = {}_sN_U \,,\quad M_U = {}_rM_{U0}\left(1 - \frac{{}_sN_U}{N_k}\right) + {}_sM_U$$

When $N_U \geq {}_sN_{cU}$ or $M_U \leq {}_rM_{U0}(1 - {}_sN_{cU} / N_k)$

$$N_U = {}_sN_{cU} + {}_rN_U \,,\quad M_U = {}_rM_U\left(1 - \frac{{}_sN_{cU}}{N_k}\right) \qquad \textbf{(1-2)}$$

where

N_U: ultimate compressive strength of member,

M_U: ultimate flexural strength of member,

${}_rN_U$: ultimate compressive strength of RC portion,

${}_rM_U$: ultimate flexural strength of RC portion,

${}_rN_{cU}$: ultimate compressive strength of RC portion subjected to compression alone,

${}_rM_{U0}$: ultimate flexural strength of RC portion subjected to bending alone,

${}_sN_{cU}$: ultimate compressive strength of steel portion subjected to compression alone,

${}_sN_U$: ultimate compressive strength of steel portion,

${}_sM_U$: ultimate flexural strength of steel portion,

${}_sM_{U0}$: ultimate flexural strength of steel portion subjected to bending alone,

N_k: elastic buckling strength of column defined by Equation 2

$$N_k = \frac{\pi^2}{{l_k}^2}\left(\frac{{}_cE \cdot {}_cI}{5} + {}_sE \cdot {}_sI\right) \qquad \textbf{(2)}$$

where

${}_cE$ and ${}_sE$: modulus of elasticity of concrete and steel, respectively,

${}_cI$ and ${}_sI$: moment inertia of concrete portion and steel, respectively,

l_k: buckling length of column

The experimental strength are compared with the strength calculated by AIJ design formula in terms of moment M – axial load N relations in Fig.7.

In Fig.7, thick solid lines are the strength of AIJ SRC Standard design formula. The thin solid lines are based on fully plastic moment. The strength of materials composing a cross section is used values shown in Table 1 and 2. And the circles show the experimental maximum strength.

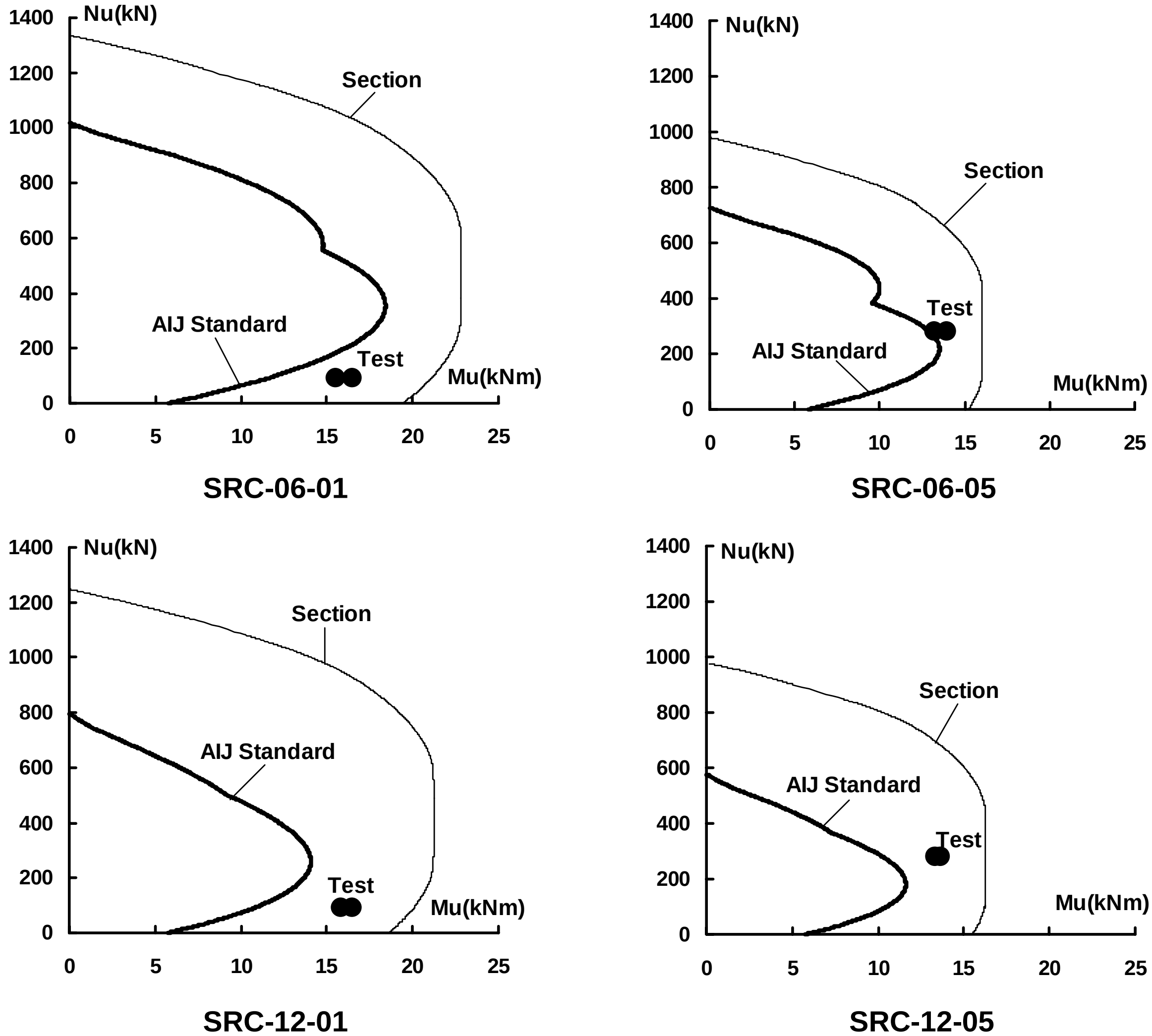


Fig. 7 Moment-axial load relations

The experimental ultimate flexural moments obtained by test results are compared with the strength calculated by AIJ design formula (See Table 5). AIJ design formula estimate the test results conservatively and reasonably. Because the ratio of test results to AIJ strength is 1.04-1.45 (L/D=6) and 1.29-1.54 (L/D=12).

Table 5. Comparison between measured strength and AIJ strength

		N (kN)	$_{ex}M_{MAX}$ (kNm)		M_{AIJ} (kNm)	M_p (kNm)	$_{ex}M_{MAX}/M_{AIJ}$	
L/D=6	SRC-06-01	91	15.5	-16.5	11.4	21.1	1.36	1.45
	SRC-06-05	282	14.0	-13.3	12.8	16.1	1.09	1.04
L/D=12	SRC-12-01	91	15.9	-16.5	10.7	20.1	1.49	1.54
	SRC-12-05	282	13.4	-13.7	10.4	16.2	1.29	1.32

N: applied axial load, M_{AIJ}: calculated ultimate flexural strength, M_p: fully plastic moment

$_{ex}M_{MAX}$: maximum experimental strength ($_{ex}Q_{MAX}$ L/4, $_{ex}Q_{MAX}$: maximum measured strength)

4. Limit story drift angle

Some of the limit story drift angle (R_{MAX},R_{95}, R_{90}) are shown in Fig.9 and Table 6.

They are defined as shown in Fig.8. i.e. R_{MAX} is the story drift angle at the maximum strength. R_{95} and R_{90} are ones at 95% and 90% strength after peak strength, respectively.

Drastic decrease of the strength is observed after peak strength in all specimens. Therefore, there are few differences between R_{MAX} and R_{90} (See Fig.9). In all the specimens, the strength decreases more than 10% of the maximum strength at the next cycle after reaching maximum strength.

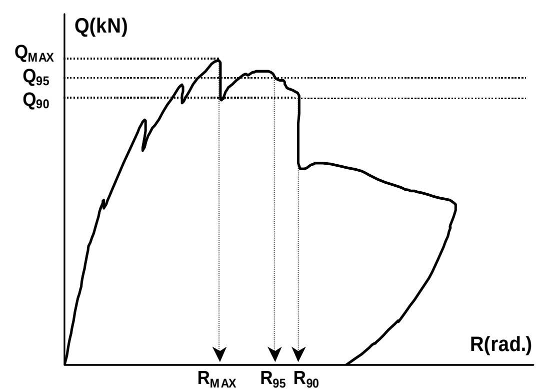


Fig. 8
Definition of limit story drift angle

Table 6. Limit story drift angle

	$_{ex}Q_{MAX}$ (kN)	R_{MAX} (rad.)	R_{95} (rad.)	R_{90} (rad.)
SRC-06-01	69.0	0.020	0.020	0.020
SRC-06-05	62.2	0.008	0.008	0.009
SRC-12-01	35.2	0.025	0.028	0.029
SRC-12-05	29.7	0.014	0.015	0.015

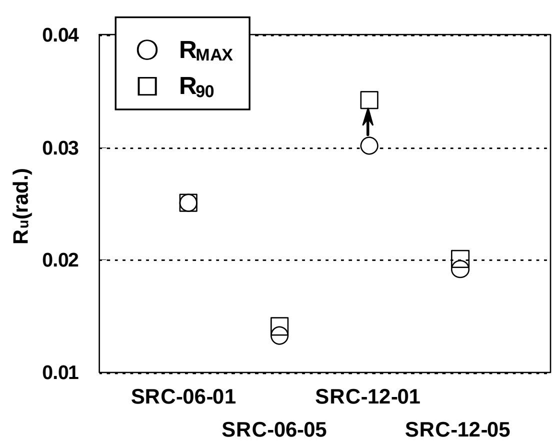


Fig. 9 Limit story drift angle

5. Equivalent viscous damping ratio

The loops drawn by the hysteresis characteristics of structures bears important role of the damping performance.

The equivalent viscous damping ratio h_{eq} is defined by Equation 3 and represents the damping performance at each loop (See Fig.10).

$$h_{eq} = \frac{1}{4\pi}\left(\frac{\Delta W}{W_e}\right) \qquad \textbf{(3)}$$

where

ΔW: area of one cycle

W_e: equivalent potential energy $= \frac{1}{2} k_e a^2$

Fig.11 shows the transitions of the equivalent viscous damping ratio at R=0.01 to 0.02. The equivalent viscous damping ratio of all the specimens is between about 0.05-0.1. The H-section of all the specimens yielded at final stage (See Fig.5), therefore it is thought that the damping performance doesn't increased greatly.

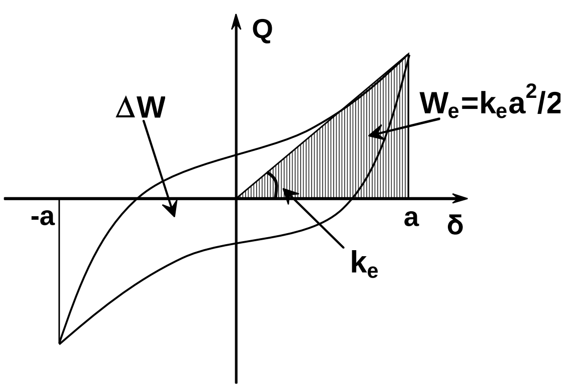


Fig.10 Definition of equivalent viscous damping ratio

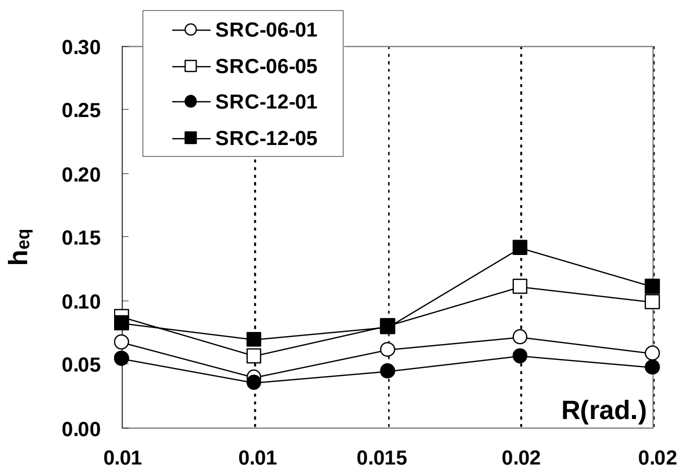


Fig.11 Transition of equivalent viscous damping ratio

CONCLUSIONS

To examine the AIJ design formula and to evaluate the deformation capacity of the composite column, an experimental work of SRC frame with weak-axis bending columns was performed. From results, it has become clear that:

1) In all of the specimens, the stable hysteresis properties were observed until their strength reached their maximum strength. And after that, the strength of all specimens deteriorated larger than that of the mechanism line.
2) The ratio of the measured strength to the value by AIJ design formula was 1.04-1.45 (L/D=6) and 1.29-1.54 (L/D=12). The AIJ SRC Standard design formula was conservative compared to the test results.

3) In all of the specimens, the equivalent viscous damping ratio (R=0.01-0.02) was between about 0.005-0.01 and damping performance didn’t increased greatly.
4) The L/D=12 specimens collapsed flexural failure. The collapse mechanism of L/D=6 was shear bond failure that was different from that assumed in AIJ design formula.

REFERENCES

1. Architectural Institute of Japan. “Standard for Structural Calculation of Steel Reinforced Concrete Structures”, 2001.