Experimental Study on Shear Resistance of SRC Column and Steel Beams Frames Constructed by Simplified Method

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ABSTRACT:

In this research, experimental studies were carried out in order to develop the new structure system using steel and concrete with execution method that is easier than the SRC structures and with earthquake resistant performance that is equivalent to the SRC structures. Valuables of the experiment were steel cross-sectional shape of column, confining method of the cover concrete and the ratio of column shear reinforcement. Following considerations were obtained from the experimental results.
1) Execution method using the wire mesh for the column was easier than that of conventional SRC structure.
2) All specimens except standard specimen of conventional SRC and the specimen using the wire mesh, wide steel flange width and wide steel web depth were failed in column shear. The standard specimen of conventional SRC was failed in beam-column joint shear. The specimen using the wire mesh, wide steel flange width and wide steel web depth was column flexural failure.
3) By increasing the column steel web depth, shear resistance behavior of the column and confining effect of the concrete were approximately equal to the standard specimen of conventional SRC structures. In addition, the maximum shear strength of SC specimens ware improved further than conventional SRC specimen, and even the specimen without shear reinforcement had the almost equal shear strength.
4) The shear strength of column in the column-beam frame specimen was dependent on the degree of the destruction of the joint.

KEYWORDS: SRC structure, SC column, shear resistance, shear strength, execution method, shear reinforcement, beam-column joints

1. INTRODUCTION

Recently, construction number of steel encased reinforced concrete (SRC) structures declines in Japan. Because, the structure design method of SRC is complicated and the processes of execution works are abounding for its construction. However, SRC structures are more excellent for ductility capacity than the RC structures, and the damages of the SRC structures ware slight in Hyogo-ken Nanbu Earthquake. In this research, experimental studies were carried out in order to develop the new structure system using steel and concrete with execution method that is easier than the SRC structures and with earthquake resistant performance that is equivalent to the SRC structures.

2. OUTLINE OF THE EXPERIMENT

2.1. Test Specimens and Materials Used

A total of seven specimens ware tested. The dimensions and details of the specimens are shown Fig.1 and Table 1.The specimen configuration represented upper and lower beams and column segments between inflection points in a frame subjected to lateral loading. The contraction scale of the test specimens was about 1/2 of an SRC structure on the assumption of column that included beams and beam-column joints of a middle floor in a multistory multi-span. To make sure that the column shear failure occurs prior to any other failure; the column shear strength was designed smaller than the flexural and the shear strengths of the beam and be-m-column joints. Experimental variables were method of shear reinforcement, cross section of the steel column and shear reinforcement ratio. All specimens had columns with 2,900mm height and 300mm square section, and beams with 2,900mm length and single H-section steel. Specimen SRC/S-1-30 was used as a standard for all specimens. Specimen SRC/S-1-27 was prepared in order to fail in column shear surely. Specimens of SC/S-_-W series were used welded wire mesh instead of shear reinforcement bar. Specimen SC/S-30 had no reinforcement bar. Specimens of SC/S-4 series had an H-shape steel section of column. Specimen SC/S-4-WC-27 was less than 0.1% in shear reinforcement ratio. The column steel section of SC/S-2-W-30 was designed that the flexural performance was improved by increasing the distance between flanges in as of cross section equal to standard specimen SRC/S-1-30, and that of SC/S-3-W-30 was designed the flange width was widened in addition to it.

Table 1. Outline of the specimens

Specimen		Column				Beam	Joint	
		p_w *2	Reinforcement Bar		Steel*1	Steel*1	p_w *2	Reinforcement Bar
		(%)	axial	shear			(%)	
A	SRC/S-1-27	0.19	12-D10	6ϕ@50	①	⑤	0.37	6ϕ@50
B	SC/S-4-W-27	0.11	WM3.2ϕ@50	WM3.2ϕ@50	②	⑥	0.11	WM3.2ϕ@50
C	SC/S-4-WC-27	0.04	4-6ϕ	WM3.2ϕ@150	②	⑥	0.04	WM3.2ϕ@150
D	SRC/S-1-30	0.37	12-D10	6ϕ@50	①	⑦	0.19	6ϕ@50
E	SC/S-2-W-30	0.11	WM3.2ϕ@50	WM3.2ϕ@50	③	⑦	0.11	WM3.2ϕ@50
F	SC/S-3-W-30	0.11	WM3.2ϕ@50	WM3.2ϕ@50	④	⑦	0.11	WM3.2ϕ@50
G	SC/S-3-30	0.00	-	-	④	⑦	-	-

*1 steel configration
①2H-200×100×5.5×8
②2H-240×160×4.5×12
③2H-240×100×4.5×9
④2H-240×160×4.5×6
⑤H-400×150× 9×12
⑥H-400×150×12×12
⑦H-400×150×6×12
*2 symbol
p_w : shear rainforcement ratio

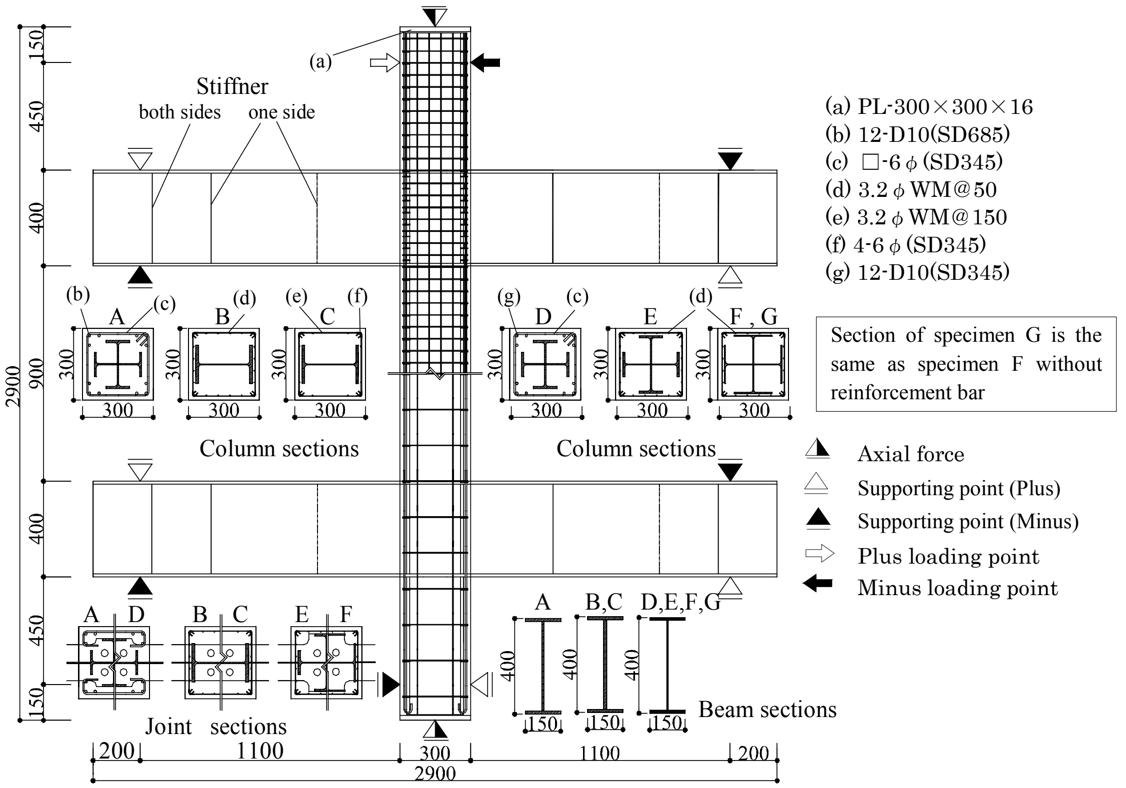

Figure 1. Outline of the specimens

Table 2-a. Mechanical properties of concrete

Specimen	compressive stress σ_B (N/mm^2)	tensile stress σ_t (N/mm^2)	compressive strain ε_u (μ)	Young's modulus $E_{1/3}$ (kN/mm^2)	Young's modulus $E_{2/3}$ (kN/mm^2)
A SRC/S-1-27	28.5	2.03	2650	24.2	20.8
B SRC/S-1-30	31.7	2.00	2590	24.8	21.4
C SC/S-2-W-30	39.0	2.20	2650	26.2	23.0
D SC/S-3-W-30	39.0	2.90	2450	27.7	24.0
E SC/S-3-30	37.7	2.90	2860	26.7	22.8
F SC/S-4-W-27	28.9	2.04	2450	25.3	21.0
G SC/S-4-WC-27	31.4	3.14	2620	26.5	22.6

Table 2-b. Mechanical properties of steel bar

Specimen	steel	yield stress σ_y (N/mm^2)	yield strain ε_y (N/mm^2)	Young's modulus E (kN/mm^2)
SRC/S-1-30 SC/S-2-W-30 SC/S-3-W-30 SC/S-3-30	3.2ϕ (W.M)	627	3120	201
	6ϕ	365	1770	207
	D10(SD345)	391	1960	200
SRC/S-1-27 SC/S-4-W-27 SC/S-4-WC-27	3.2ϕ@50 (W.M.)	644	4190	204
	3.2ϕ@150(W.M.)	559	3870	199
	6ϕ	368	1990	203
	D10(SD685)	702	5630	201

* W.M.: welded wire mesh

Table 2-c. Mechanical properties of steel plate

Specimen	steel plate	yield stress σ_y (N/mm^2)	yield strain ε_y (N/mm^2)	Young's modulus E (kN/mm^2)
SRC/S-1-30 SC/S-2-W-30 SC/S-3-W-30 SC/S-3-30	PL4.5	321	1680	192
	PL5.5	354	1810	196
	PL6	349	1691	207
	PL8	312	1500	208
	PL9	301	1641	187
	PL12	287	1560	184
SRC/S-1-27 SC/S-4-W-27 SC/S-4-WC-27	PL4.5	329	1710	195
	PL5.5	331	1880	197
	PL8	314	1540	199
	PL9	309	1650	195
	PL12	276	1580	189

The design strength of the concrete was set at 30N/mm^2 and 27N/mm^2. The arrangements of the main column reinforcement bars were 12-D10 for the SRC/S-series, and 4-6ϕ for the SC/S-4-WC-27. Other specimens had no main column reinforcement bars. The shear reinforcement of SRC/S-series used 6φ and other specimens used welded wire mesh of 3.2ϕ. Material properties are shown in Table 2-a, 2-b and 2-c.

2.2. Loading and Instrumentation

The specimens were loaded lateral cyclic shear force by horizontal actuator at the top of the column while a constant axial compression of $\sigma_0/6$ was applied by vertical actuator, as shown in Fig.2. And upper beam and lower beam were always kept parallel by four jacks installed in the end of the beams.
The incremental loading cycles were controlled by story drift angles, R, defined as the ratio of lateral displacements to the column height, δ/h. The lateral load sequence consisted of two cycles to each story drift angle, R of 0.002, 0.005, 0.010, 0.017, 0.026, 0.037 and 0.05 radians.
During the tests, the forces, displacements and reinforcement strains were measured.

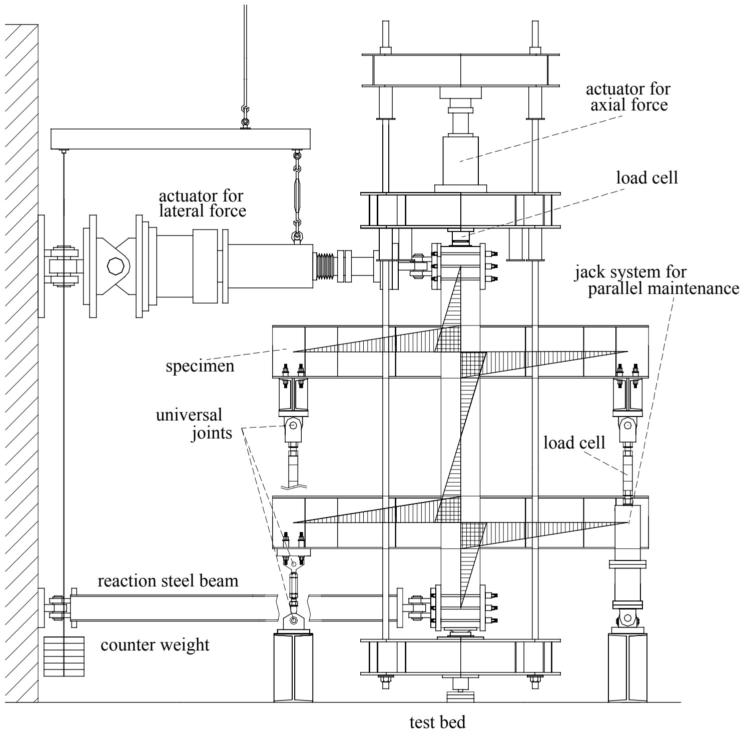

Figure 2. Outline of the loading equipment

3. EXPERIMENTAL RESULTS AND DISCUSSIONS

3.1. Circumstances of Failure

The circumstances of the failures for all specimens after testing are shown in Photo 1. Specimen SRC/S-1-30 failed upper and lower joints shear. An initial flexural crack appeared in the column end and beam-column joints during the first cycle loading and an initial shear crack appeared in the column and joints during the second cycle loading. After that, shear cracks markedly appeared in most parts in the joint by the fifth cycle. The widening of the shear crack of the joint was intensified after the seventh cycle, and the concrete cover came off in places. The shear crack of the column, however, was not widened. On the other hand, specimen SRC/S-1-27 failed shear in the end of column. An initial shear crack appeared in the column and joints at R (story drift angles) =0.002 to 0.005 radian. After that, the widening of the shear crack and crushing in the end of column were intensified at R=0.01 to 0.017 radian, and the shear reinforcement bars of column yielded. After R=0.037 radians, cover concrete of column was flaked off intensively. Specimen SC/S-2-W-30 failed column shear. An initial shear crack appeared in the column at R=0.005 radian. At R=0.01 radian, the welded wire mesh of joints and column were yielded and the flexural crack appeared in the end of column and the widening shear crack in the column were intensified. After maximum strength, the flaking of the cover concrete occurred in the whole column, the steel flange of column was bared, and the cutting off of the wire mesh was also observed. For SC/S-3-W-30, an initial shear crack appeared in the column at R=0.005 radian. At R=0.01 radian, the welded wire mesh of joints and column were yielded and the flexural crack appeared in the end of column and the widening shear crack in the column were remarkable. However, the ability of the jack reached it limitative at R=0.026 radians, and the experiment were finished. Specimen SC/S-3-30 failed shear in the column. An initial shear crack appeared in column at R=0.005 radian. Many shear cracks appeared and widened in the whole column. After that, the cover concrete was flaked in the whole column and the buckling of steel flange was occurred in the end of column. For SC/S-4-W-27, in the appearance, it was the flexure fracture of the end of column. An initial flexural crack and an initial shear crack of joints and column appeared at R=0.005 radian.

Mostly, the flexural cracks had grown and widened in the end of column. After that, however, the widening of shear crack of the joints was remarkable. After maximum strength, the welded wire mesh was bared in the end of column and the buckling and cutting off of welded wire mesh were occurred, and the concrete cover came off in the joints. Specimen SC/S-4-WC-27 failed shear in the column. An initial flexural crack and shear crack appeared in the column and joints at R=0.005 radian. After that, joint shear cracks appeared and had grown remarkable, and the corner bars in the column and welded wire mesh in the joints were yield. After maximum strength, the widening of the shear crack of the joints was intensified, and the welded wire meshes in the column were bared, and the buckling and the cutting off of the wire mesh was also observed.

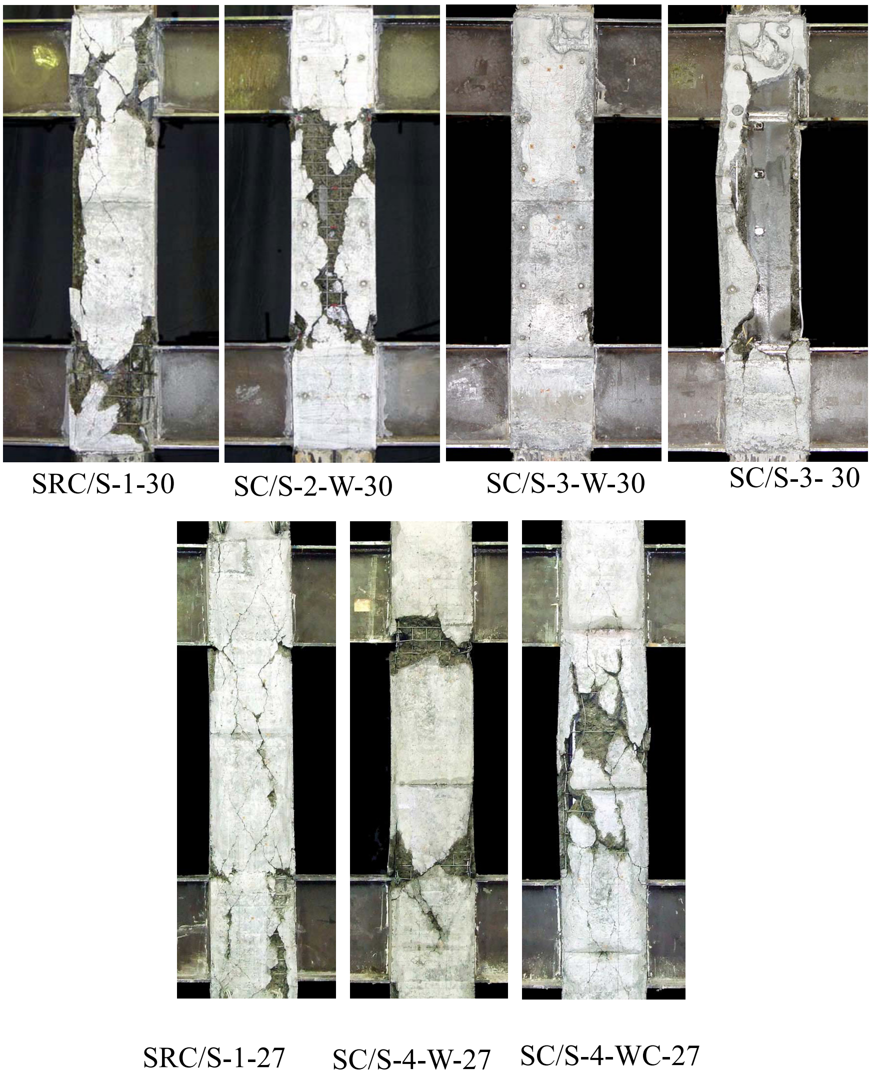

Photo 1. The circumstances of the failures

3.2. Load vs. Displacement Relationship

Skelton curves of all specimens, which were obtained from the interaction curves of the column shear force '${}_{C}Q$' and story deformation angle '${}_{C}R$', are shown for comparison in Figures 3 to 5.

3.2.1 Effect of the shape of the column steel section by cross H-section (Figure 3)

The effect of the shape of the column steel section can be observed by comparing SRC/S-1-30, SC/S-2-W-30, SC/S-3-W-30 and SC/S-3-30. The initial stiffness was almost same on four specimens, and the maximum shear strength was also almost same except for SC/S-3-W-30. In spite of the difference of the member, which failed SC/S-2-W-30 and SRC/S-1-30, the similar behavior was proved in ${}_{C}Q$-${}_{C}R$ relationship. Therefore, it was proven that to obtain performance of structure equal to usual SRC structure was possible using the wire mesh by enlarging the steel cross section of column. For SC/S-3-W-30, the shear stress kept rising after stiffness reduction, and it reached pressurization limit of the actuator for lateral force at R= 0.026 radian, and the experiment was finished. However, the rise of the shear stress was a flat almost, and the shear stress at R= 0.026 radian was considered maximum strength. By increasing the column steel web depth, shear resistance behavior of the column and confining effect of the concrete were approximately equal to the standard specimen, SRC/S-1-30. In addition, the maximum shear strength of SC specimens ware improved further than conventional SRC specimen, and even the specimen without shear reinforcement had the almost equal shear strength.

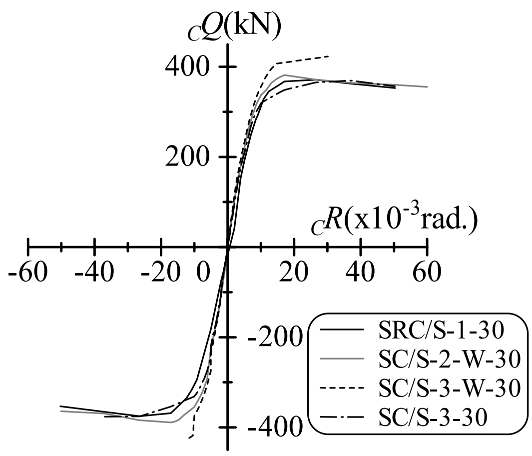

Figure 3. Skelton curves (Effect of the shape of the column steel section by cross H-section)

3.2.2 Effect of the shape of the column steel section between cross H-section and single H-section (Figure 4)

The effect of the shape of the column steel section between cross H-section and single H-section can be observed by comparing SC/S-4-W-27, SC/S-2-W-30 and SC/S-3-W-30. The initial stiffness was almost same on three specimens. It was observed a difference of the shear stress after around ${}_{C}R$=0.02 radian, and for the maximum shear strength of SC/S-3-W-30, SC/S-4-W-27 and SC/S-2-W-30 were respectively reduction of 30kN and 40kN. The reason why the difference occurred at the maximum strength was not the existence of the transverse flanges, because the concrete strength of SC/S-3-W-30 was bigger than SC/S-4-W-27 about 10N/mm^2. And, it seemed to hold the shear stress lowering by the transverse flanges restricting the core concrete, because in comparison with the looped form of SC/S-2-W-30 and its of SC/S-4-W-27 after ${}_{C}R$=0.026 radian, SC/S-4-W-27 gradually changed in the pinching form, therefore SC/S-4-W-27 was fusiform. However, SC/S-2-W-30 and SC/S-4-W-27 was same behavior in the large displacement. It seems to be the reason why steel burden shear stress and core concrete burden shear stress surrounded in the steel were almost equal.

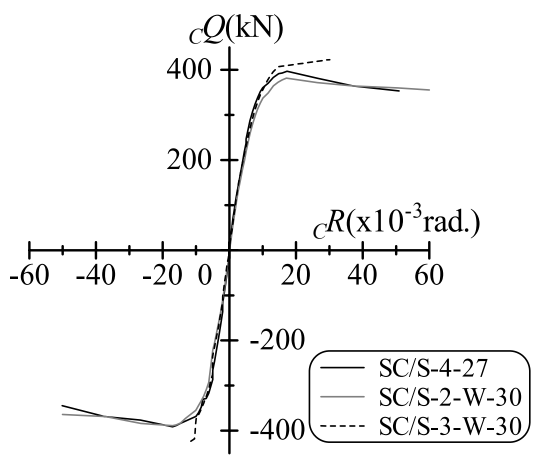

Figure 4. Skelton curves (Effect of the shape of the column steel section between cross H-section and single H-section)

3.2.3 Effect of the shear reinforcement ratio of the welded wire mesh (Figure 5)

The effect of the shear reinforcement ratio of the welded wire mesh can be observed by comparing SC/S-4-W-27 and SC/S-4-WC-27. Stiffness reduction also appeared both specimen near ${}_{C}R$=0.007 radian. SC/S-4-W-27 became ${}_{C}R$=0.017 radian with the maximum strength, and SC/S-4-WC-27 became ${}_{C}R$ =0.01 radian with the maximum strength. As a factor of the maximum strength reduction, SC/S-4-W-27 was mainly the progress of flexure fracture and collapse in the column edge, and SC/S-4-WC-27 was the flexure failure over column full face. From this fact, though there was a function of the shear reinforcement on the welded wire mesh, it was proven to be insufficient for retaining the ductility capacity in p_w=0.1% or less that the quantity is regulated in the SRC standard.

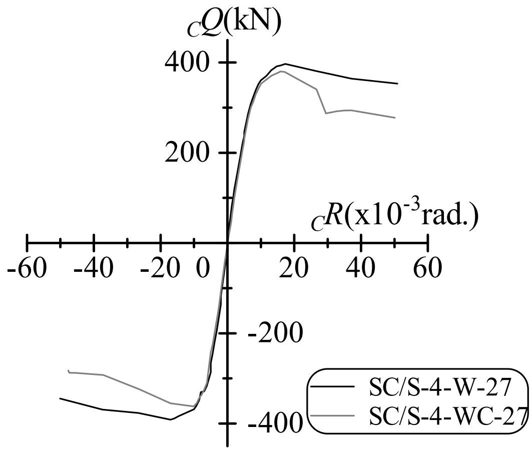

Figure 5. Skelton curves (Effect of the shear reinforcement ratio of the welded wire mesh)

3.3. Ultimate Shear Strength of the Column

The calculated values of the strengths of the column and beam-column joints for all specimens were calculated from the AIJ-SRC Standard equations, the experimental and the calculated values are shown in Table 4. The experimental and calculated values were compared for the column shear force. The AIJ-SRC standard equations are shown in Table 3.

Table 3. The AIJ-SRC standard equations

Ultimate shear strength of column	Ultimate shear strength of joint
$${}_{C}Q_{su} = \underbrace{{}_{s}Q_{U}}_{steel} + \underbrace{{}_{r}Q_{U}}_{RC}$$ $${}_{s}Q_{U} = \min({}_{s}Q_{sU}, {}_{s}Q_{bU})$$ $${}_{s}Q_{sU} = {}_{c}t_{w} \cdot 2({}_{C}D_{s} - 2{}_{c}t_{f}){}_{Cw}\sigma_{y} / \sqrt{3}$$ $${}_{s}Q_{bU} = \Sigma({}_{s}M_{U} / h_{e}) = 2{}_{s}M_{U} / h_{e}$$ $${}_{r}Q_{u} = \min({}_{r}Q_{sU1}, {}_{r}Q_{sU2}, {}_{r}Q_{bU})$$ $${}_{r}Q_{sU1} = 7/8 \cdot {}_{c}b \cdot {}_{c}d(0.5F_{s} \cdot {}_{r}\alpha + 0.5p_{w} \cdot {}_{rw}\sigma_{y})$$ $${}_{r}Q_{sU2} = 7/8 \cdot {}_{c}b \cdot {}_{c}d(F_{s} \cdot b' / {}_{c}b + p_{w} \cdot {}_{rw}\sigma_{y})$$ $${}_{r}Q_{bU} = \Sigma({}_{r}M_{u} / h_{e}) = 2 \cdot {}_{r}M_{U} / h_{e}$$	$${}_{cal.J}Q_{su} = Q_{ju} / \left\{ \frac{(l - {}_{mC}d) \cdot h}{{}_{sB}d} - 1 \right\}$$ $$Q_{ju} = {}_{j}M_{u} / {}_{sB}d$$ $${}_{j}M_{u} = \underbrace{{}_{c}V_{e}({}_{j}F_{s} \cdot {}_{j}\delta + {}_{w}p \cdot {}_{rw}\sigma_{y})}_{RC} + \underbrace{\frac{1.2{}_{s}V \cdot {}_{s}\sigma_{y}}{\sqrt{3}}}_{steel}$$ **Ultimate flexural strength** $${}_{C}Q_{Mu} = 2{}_{C}M_{U} / h_{e}$$ $${}_{C}M_{U} = \underbrace{{}_{s}M_{U}}_{steel} + \underbrace{{}_{r}M_{U}}_{RC}$$
${}_{s}Q_{sU}$: shear strength of steel (N) ${}_{s}Q_{bU}$: flexural strength of steel (N) ${}_{r}Q_{sU1}$: shear strength(N) ${}_{r}Q_{sU2}$: bond splitting strength(N) ${}_{r}Q_{bU}$: flexural strength of R/C(N) ${}_{rw}\sigma_{y}$: yield tress of shear reinforcement(N/mm^2) F_s : shear strength of concrete(N/mm^2) b' : effective width by steel flange(mm) ${}_{c}r_{U}$: reduction coefficient *${}_{c}r_{U}$=0.85−2.5${}_{s}p_{c}$	Q_{ju}: ultimate shear strength of joint(N) ${}_{j}M_{u}$: ultimate flexural moment of joint(N･mm) ${}_{sB}d$: distance between steel flanges center of gravity (mm) ${}_{mC}d$: distance between main bars center of gravity (mm) ${}_{c}V_{e}$: volume of R/C of joint(mm^3) ${}_{j}\delta$: coefficient of shape of joint ${}_{s}V$: volume of steel of joint(mm^3)

The calculated values of column shear by SRC standard equation had overestimated the experimental value, when calculated values were compared with experimental values in all test specimen except for SRC/S-1-30. Following two points was considered as this cause. The first, the calculated value of the shear strength shared by the RC of the SRC standard was experimental equation which applied the RC standard, which was required by the column shear experiment with the conventional loading stub, and it is different from the specimen in this study. The second, there was the report that the column shear strength was reduced by the difference between the damage circumstances of the joint. Then, as a result of examining the effect of shear capacity magnification factor of the beam-column joint on column shear capacity calculation equation, it was proven that the experimental value had been overestimated, as shear capacity magnification factor of beam-column joint is smalle. Still, it was clarified that there was no effect of the joint damage on the column shear capacity, if the joint shear margin is over 1.50.

Table 4. Experimental and calculated values on ultimate strength

	Specimen	maximum strength (kN)	calculate ultimate strength by AIJ-SRC standard(kN)			${}_{exp.C}Q_u / {}_{cal.*}Q_{**}$		shear capacity magnification factor	failure position
			joint shear	column shear	column flexural	joint	column shear	$\frac{{}_{cal.J}Q_{su}}{{}_{cal.C}Q_{su}}$	
		${}_{exp.C}Q_u$	${}_{cal.J}Q_{su}$	${}_{cal.C}Q_{su}$	${}_{cal.C}Q_{bu}$	${}_{cal.J}Q_{su}$	${}_{cal.C}Q_{su}$		
A	SRC/S-1-27	347	506	361	506	0.69	0.96	1.40	column
B	SC/S-4-W-27	397	483	431	544	0.82	0.92	1.12	column
C	SC/S-4-WC-27	380	478	420	540	0.80	0.91	1.14	column
D	SRC/S-1-30	375	472	421	447	0.79	0.89	1.12	joint
E	SC/S-2-W-30	389	580	387	475	0.67	1.01	1.50	column
F	SC/S-3-W-30	429	625	469	555	0.69	0.92	1.33	column
G	SC/S-3-30	369	556	403	507	0.68	0.93	1.38	column

4. CONCLUSIONS

The following observations were obtained from this experimental study of SRC structures.
1) Execution method using the wire mesh for the column was easier than that of conventional SRC structure.
2) All specimens except standard specimen of conventional SRC and the specimen using the wire mesh, wide steel flange width and wide steel web depth were failed in column shear. The standard specimen of conventional SRC was failed in beam-column joint shear. The specimen using the wire mesh, wide steel flange width and wide steel web depth was column flexural failure.
3) By increasing the column steel web depth, shear resistance behavior of the column and confining effect of the concrete were approximately equal to the standard specimen of conventional SRC structures. In addition, the maximum shear strength of SC specimens ware improved further than conventional SRC specimen, and even the specimen without shear reinforcement had the almost equal shear strength.
4) The shear strength of column in the column-beam frame specimen was dependent on the degree of the destruction of the joint.

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