EARTHQUAKE RESISTANT PROPERTIES OF CORE STEEL COMPOSITE COLUMNS

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SUMMARY

A large number of reinforced concrete (RC) buildings collapsed with story failures by 1995 Hyogoken-Nanbu earthquake in Japan. Especially RC columns at the corner post on the failure story collapsed in shear brittly under large compressive axial forces generated by large horizontal and vertical accelerations. In order to prevent happening brittle shear failure of RC columns and occurring the story failure of building structures, it is necessary to make the ductility of columns larger.

It can be thought using core steel composite columns is useful as one of the reinforcing RC columns. The beam columns are monolithic concrete encased small depth steel member with longitudinal reinforcing bars. It is thought that the encased core steel are useful for resisting large axial compressive force and little bending moment, and for preventing columns from shear failure.

This paper presents the results of an experimental work carried out in order to study elastic-plastic behaviors of the core steel composite columns under a constant axial compressive load and cyclic horizontal load. The experimental parameters were as follows; 1) Cross section (core steel composite column, RC column and usual steel reinforced concrete column), 2) Ratio ***n*** of axial compressive load against ultimate compressive strength of cross section, 3) Area ratio of steel section against total concrete cross section (1.5%, 3.0% and 5.0%), and 4) Shape of core steel section (H shaped section and compact circle section). Total 14 specimens were tested. It is the purpose of this study to describe the elastic-plastic behavior of the core steel composite columns under large compressive axial load and cyclic horizontal load, and to show the composite columns have large better earthquake resistant performance than RC columns.

INTRODUCTION

A large number of reinforced concrete (RC) buildings collapsed with story failures by 1995 Hyogoken-Nanbu earthquake in Japan. Especially RC columns at the corner post on the failure story collapsed in shear brittly under large compressive axial forces generated by large horizontal and vertical accelerations.

In order to prevent the story failure, it is necessary to make the ductility of columns larger. And some reinforcing methods have been reported, such as covering RC columns by steel tubes or confining RC columns by arranging transverse reinforcement such as hoop ties closely. It is thought that another reinforcing method is encasing core steel into RC columns. Buckling of main bars causes RC columns to collapse and not to be able to hold axial load. On the other hand, the core steel composite is useful to resist large axial force and to make the concrete strain less. And it is thought that the composite columns with sufficient quantities of core steel may hold large axial load even if buckling of the main bars and crushing concrete occurs.

This paper presents the results of an experimental work of the core steel composite columns under earthquake loading and discussion of earthquake resistant properties of the composite columns.

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EXPERIMENTAL WORK

In order to study the elastic-plastic behavior of the core steel composite columns under earthquake loading, 14 specimens were tested. Specimens are shown in Fig. 1. Encased H-shaped steel was placed as to make it subjected to strong axis bending. Loading apparatus is shown in Fig. 2. The cyclic horizontal load ***H*** was applied under constant compressive load ***N*** on the column top.

In this paper, the encased core steel is considered as resisting large axial force and little bending moment, and so the depth of the core steel is defined as less than 1/3 of that of concrete. The following experimental parameters were selected, in order to make clear that the effect of encasing core steel, axial force, quantities of core steel and shape of core steel on the behavior of composite columns under earthquake loading.

a) Cross section: 1) Core steel composite section, 2) Reinforced concrete section and 3) usual steel reinforced concrete (SRC) section.
b) Axial compressive load ***N***: 1) Axial load ratio ***n*** equals to 0.3, which is the ratio of axial force ***N*** against the ultimate compressive strength of the cross section ***Nu***, 2) Limiting axial force N_l prescribed by AIJ standard [1].

$$N_l = 1/3 \cdot {}_cA \cdot {}_c\sigma_b + 2/3 \cdot {}_sA \cdot {}_s\sigma_y \qquad 1)$$

Where ${}_cA$, ${}_sA$: Area of concrete section and steel section, respectively, ${}_c\sigma_b$: Compressive strength of concrete, ${}_s\sigma_y$: Yield stress of steel.

c) Ratio of steel area to the total area of the concrete :1)1.5%, 2) 3.0% and 3) 5%.
d) Shape of core steel section: 1) H shaped section and 2) Compact circular section.

The mechanical properties of steel and compressive strength of concrete cylinder are shown in Table 2 and in Table 3, respectively.

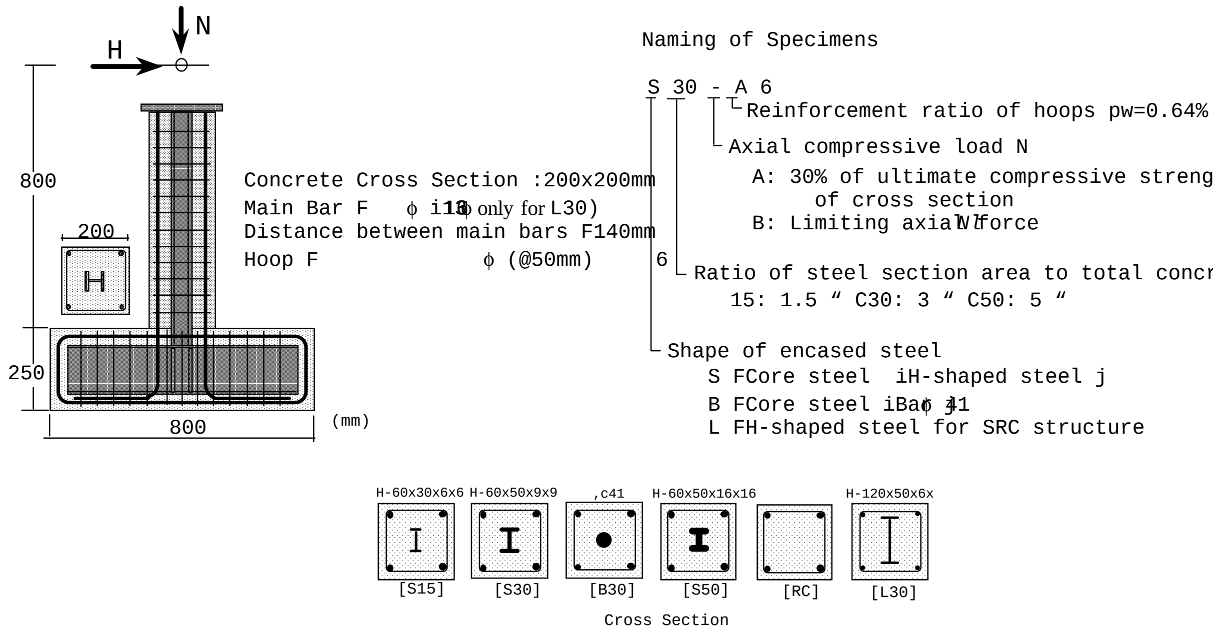


Fig. 1 Specimen

Table 1 Test program

Name of Specimen	Column	Size of encased steel	Steel ratio	Applying Axial Load	Axial Load N (kN)	Compressive Strength (kN) Nu	sNu	cNu	Axial load ratio n	ns	nc
S15-A6	Core steel Composite Column	H-60x30x6x6	1.62%	n = 0.3	418	1,732	235	1,252	0.24	1.78	0.33
S30-A6		H-60x50x9x9	3.20%		616	2,053	505	1,324	0.30	1.22	0.47
B30-A6		Bar 41φ	3.35%		601	2,003	489	1,560	0.30	1.23	0.39
S50-A6		H-60x50x16x16	5.12%		664	2,212	722	1,296	0.30	0.92	0.51
S15-B6		H-60x30x6x6	1.62%	Limiting axial force *Nl*	574	1,749	235	1,268	0.33	2.44	0.45
S30-B6		H-60x50x9x9	3.20%		750	2,007	507	1,276	0.37	1.48	0.59
B30-B6		Bar 41φ	3.35%		747	2,013	491	1,304	0.37	1.52	0.57
S50-B6		H-60x50x16x16	5.12%		891	2,220	719	1,304	0.40	1.24	0.68
RC-A6	RC Column	-	-	n = 0.3	467	1,557	-	1,296	0.30	-	0.36
RC-B6				*Nl*	663	1,789	-	1,524	0.37	-	0.44
L30-A6	SRC Column	H-120x50x6x6	3.12%	n = 0.3	640	2,133	471	1,532	0.30	1.36	0.42
L30-B6				*Nl*	807	2,126	472	1,524	0.38	1.71	0.53

@1) Nu, sNu and cNu are compressive strength of total cross section, steel section and concrete section, respectively
@2)Axial load ratio Fn=N/Nu Cns=N/sNu Cnc=N/cNu

Table 2 Mechanical properties of steel

Steel	Yield Stress (N/mm²)	Tensile Stress (N/mm²)	Yield ratio	Remarks
PL-6mm	392	539	0.73	Flange and Web for S15 series
PL-9mm	421	557	0.76	Flange and Web for S30 series
PL-16mm	359	527	0.68	Flange and Web for S50 series
Bar 41φ	366	554	0.66	Core steel for B30 series
Bar 16φ	330	487	0.68	Main Bar for specimens except
Bar 13φ	343	510	0.67	Main bar for L30
Bar 6φ	408	538	0.76	Hoop for all specimens

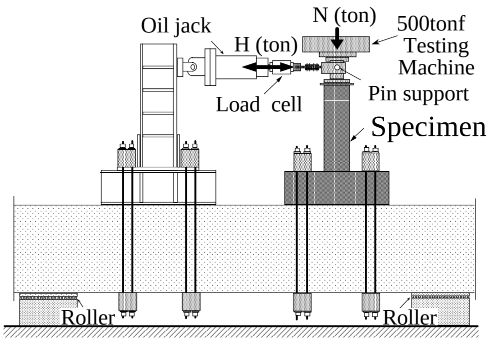


Fig.2 Loading apparatus

TEST RESULTS

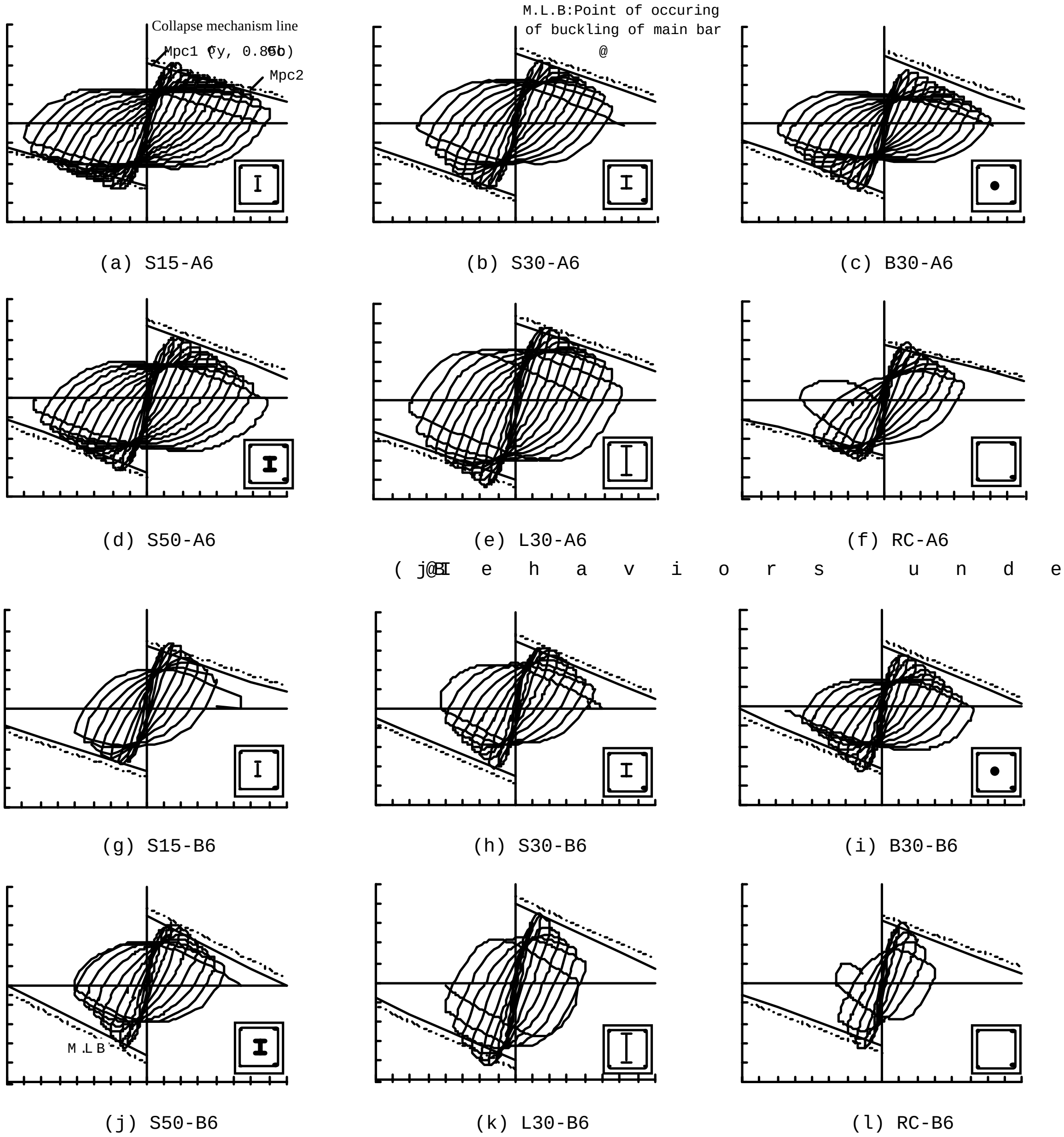
Relationship of Horizontal Load and Rotation Angle:

The experimental relations of horizontal load ***H*** and the rotation angle of the column ***R*** are shown in Fig. 3. In these figures slight solid lines and dotted lines indicate the plastic collapse mechanism lines which are obtained by assuming plastic hinge forming at the column base. The full plastic moment for calculating mechanism lines agree with the superposed strength, and they are obtained from the fully plastic stress distribution as shown in Fig. 4. The superposed strength Mpc1 was calculated by using steel yield strength $_s\sigma_y$ and 0.85 times of concrete strength $_c\sigma_b$ (slight solid lines in Fig. 3), and Mpc2 was calculated by $_s\sigma_y$ and $_c\sigma_b$ (dotted lines in Fig.3).

RC columns collapsed after buckling of main bars occurred and could not hold compressive axial load. On the other hand the core steel composite columns and SRC columns were able to hold axial load up to the maximum displacement. The displacement was the point where horizontal force ***H*** decreased to zero, caused by the effect of the overturning moment produced by the vertical load and the horizontal deflection, and by occurring of crush of concrete and buckling of main bars. The incipient crushes of covering concrete happened at ***R*** equal to 1/100 or 1.5/100 rad. The points of occurring of main bars buckling were indicated in Fig. 3.

DISCUSSIONS

Elastic-pPlastic Behavior:



(II) Behaviors under limiting axial force Nl prescribed by AIJ standard [1]

Fig. 3 Relationship of horizontal load and rotation angle of column

Figure 5 shows the relationship of the column base moment and loading cycles on unloaded points. In this figure, vertical axis shows the column base moment divided by the maximum moment, and horizontal axis shows the number of loading cycle. Figure 6 shows the relationship of the shrinkage of the column and loading

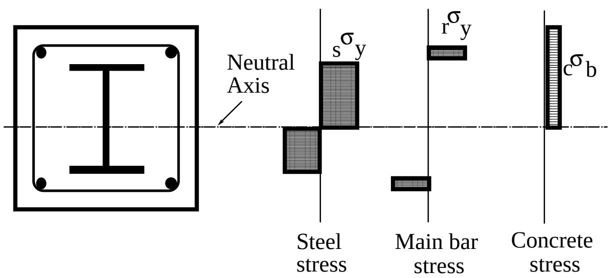


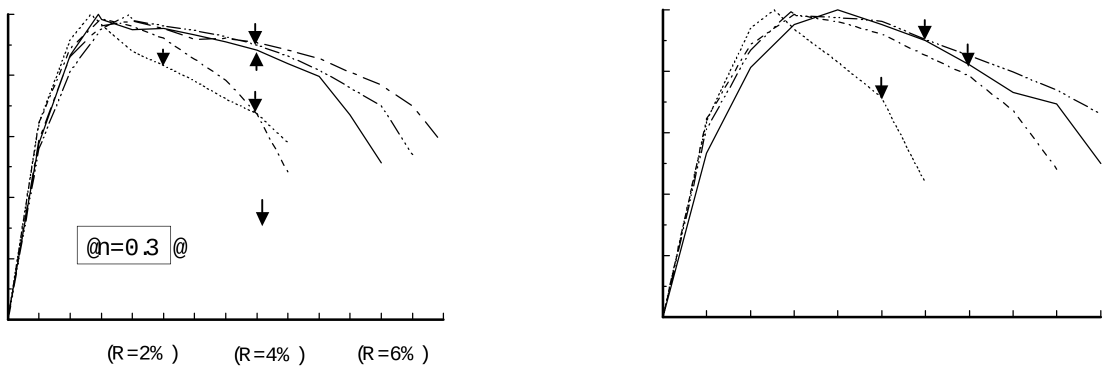
Fig. 4 Stress distribution at the ultimate state

cycle.

Table 3 Test results

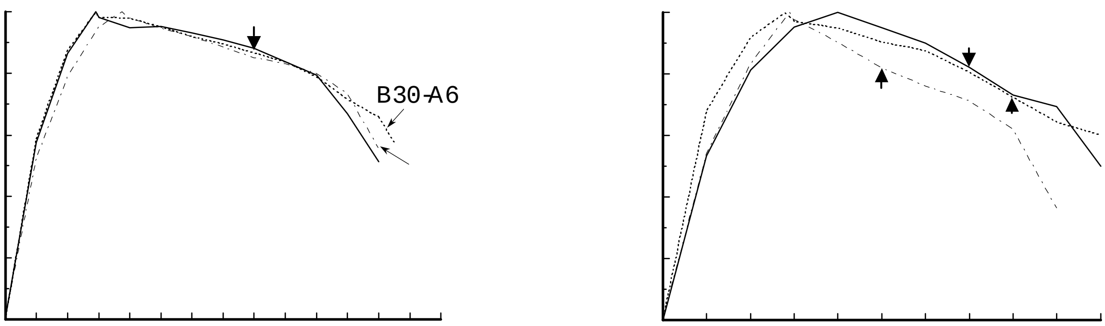
Name of Specimen	cσb (Mpa)	Bending Strengt Mmax (kN m)		Calculated Strength				Deformation capac	
		positive	negative	Mpc1 (KN m)	Mpc2 (KN m)	Mmax / Mpc1	Mmax / Mpc2	μm90	μd1
S15-A6	31.3	54.5	-59.2	49.0	53.0	1.16	1.07	29.7	122.9
S30-A6	33.1	59.0	-61.4	57.9	61.6	1.03	0.98	17.5	58.1
B30-A6	39.0	51.1	-61.1	50.5	55.5	1.11	1.01	14.0	91.3
S50-A6	32.4	58.3	-67.2	59.8	64.6	1.05	0.97	23.4	75.3
S15-B6	31.7	60.3	-51.0	51.3	55.2	1.08	1.01	10.6	38.3
S30-B6	31.9	61.2	-56.3	54.9	61.1	1.07	0.96	14.0	41.1
B30-B6	32.6	50.9	-60.2	51.0	55.2	1.10	1.01	9.8	62.3
S50-B6	32.6	61.7	-60.0	57.9	62.9	1.05	0.97	10.4	-
RC-A6	32.4	51.2	-53.8	45.2	48.1	1.16	1.10	3.9	36.8
RC-B6	38.1	55.3	-59.2	50.8	55.9	1.13	1.02	3.5	15.9
L30-A6	38.3	70.1	-78.6	65.1	70.2	1.14	1.06	11.0	48.8
L30-B6	38.1	66.1	-78.1	63.8	70.6	1.13	1.03	4.5	22.8

1 cσb Compressive strength of concrete cylinder 2 Calculated Strength Mpc1:Superposed Strength calculated by using steel yield stress (sσy) and 0.85 times of value of cσb, Mpc2: Superposed Strength calculated by using sσy and cσb 3) μm90: Ductility factor corresponding to the criterion that flexural strength of column base moment deteriorated to 0.9 times of the



(a) under axial load ratio n = 0.3 (b) under limiting axial force prescribed by AIJ standard [1]

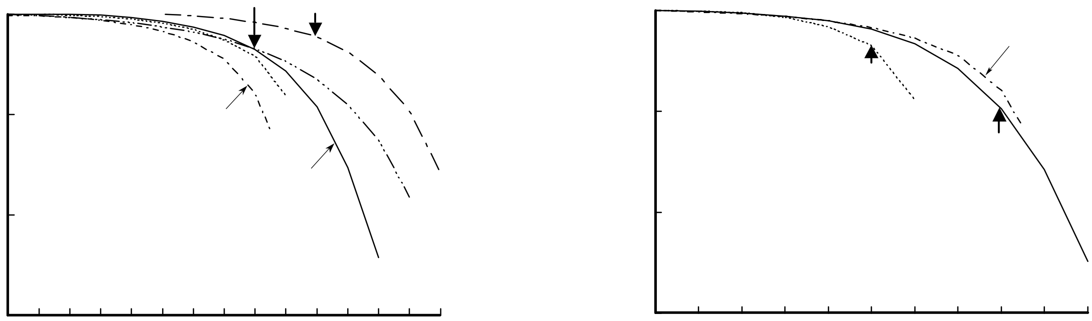
(i) Effect of area of steel section on behaviors of columns



(a) under axial load ratio n = 0.3 (b) under limiting axial force prescribed by AIJ standard [1]

(ii) Effect of shape of steel section on behaviors of columns

Fig. 5 Behaviors of column base moment



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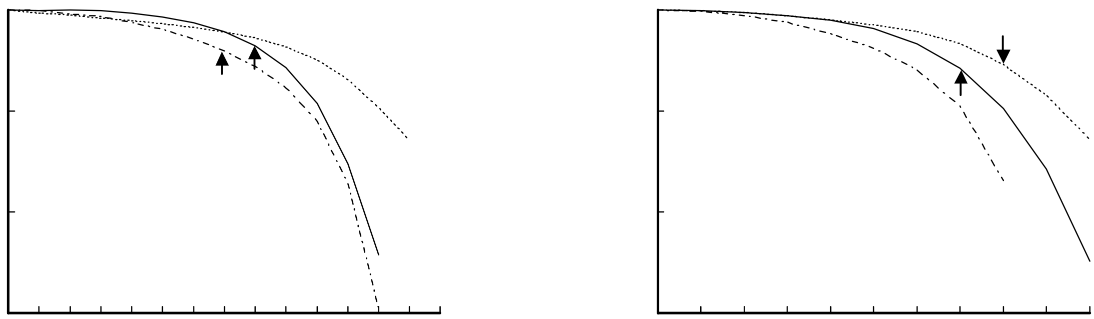


(a) under axial load ratio n = 0.3 (b) under limiting axial force prescribed by AIJ standard [1]

(ii) Effect of shape of steel section on behaviors of columns

Fig. 6 Behaviors of shrinkage of column length

Effect of encased steel on Behavior of Columns:

Behavior of the core steel composite columns and that of RC columns are shown in Fig 5 (a) and (b). The behavior of the core steel composite columns after displacement exceeds the value corresponding to maximum strength is much better than that of RC columns. The reasons are as follows. Crushing concrete and buckling of main bars of the composite columns occurred much later than that of RC columns. And the encased core steel was useful for shrinkage of the column as shown in Fig. 6 (a) and (b), so rigidity for shrinkage of column length of the composite columns is much larger than that of RC columns.

The composite columns with sufficient steel for axial compressive load were able to hold axial load up to the large deformation, so it can be thought that the core steel composite columns are useful for preventing collapse of columns and story failure.

Effect of steel section area on Behavior of Composite Columns:

The axial load ratio ***n*** of the specimen *S15-B6* was 0.33 and the behaviors of this specimen can be compared to those of specimens ***n*** equal to 0.3. The value of ***ns***, which is the ratio of compressive strength of the steel section to the axial load, was 2.4 for specimen *S15-B6*. The value of ***ns*** for this specimen is much large, so the behavior of this specimen was similar to the specimen *RC-A6* (see Fig. 5 (a) and Fig. 6 (a)). The specimens with ***ns*** equal to 1.5 show large earthquake resistant properties (see Fig. 5 (a) *S15-A6*, *S30-A6*, and (b) *S30-B6*). But the effect of making area of steel section increasing and making the value of ***ns*** less than 1.5 on the behavior of columns is small (see Fig. 5 (a) and Fig. 6 (b) *S30-A6* and *S50-A6*, and Fig. 5 (b) *S30-B6* and *S50-B6*). The reason is that the compressive stress of covering concrete becomes large and covering and covered concrete crush before steel with large area of cross section yield. From the test results, core steel encased with ***ns*** equal to 1.5 is useful for composite columns to make large earthquake resistant properties.

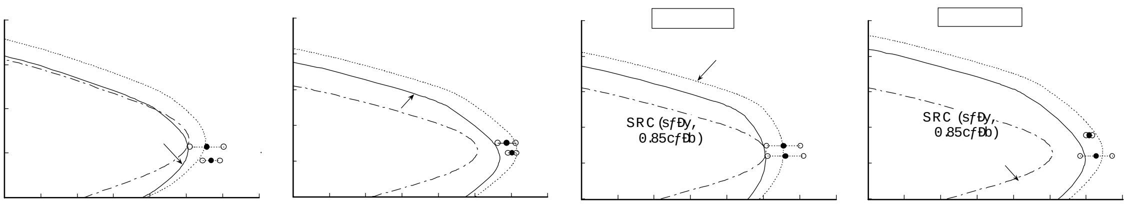
Effect of encased steel shape on Behavior of Composite Columns:

There is no difference in behaviors of column base moment for the core steel composite columns with H shaped section and that with compact circular section (see Fig. 5 (c) *S30-A6* and *B30-A6*, and (d) *S30-B6* and *B30-B6*). But the shrinkage of column length with compact circular steel was much less than that with H shaped steel (Fig. 6 (c) *S30-A6* and *B30-A6*, and (d) *S30-B6* and *B30-A6*). In order to make the core steel useful to be less shrinkage of column length, encasing of the compact steel is more useful than that of H - shaped steel, because H shaped steel resists axial load and also bending moment, even if small depth steel was used.

There is no difference in behaviors for specimen with the core steel composite columns and that with usual SRC columns under the axial force 0.3***Nu*** (see Fig. 5 (c) and Fig. 6 (c) *S30-A6* and *L30-A6*). But the columns encased core steel show larger earthquake resistant properties than those with usual SRC columns under limiting axial force prescribed by AIJ standard (see Fig. 5 (d) and Fig. 6 (d) *S30-B6* and *L30-B6*). The reason is thought as follows. Tensile stress of steel flange occurred by bending moment for SRC columns is larger than that of core steel composite columns, so axial force of concrete for SRC columns is larger than that for the columns with core steel.

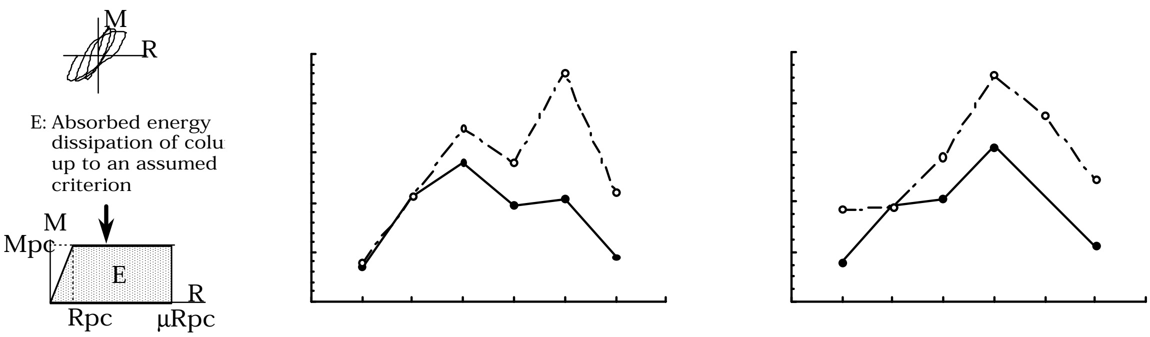
Flexural Strength

Relations of axial load ***N*** and the superposed strength Mpc of the cross sections are shown in Fig. 7. Dotted points show the experimental maximum moment at the column base. The flexural strength of the core steel composite columns can be estimated conservatively by the superposed strength ***Mpc1*** calculated with yield stress of steel and 0.85 times of compressive strength of concrete. As a result of investigating data taken measurements by strain gauge pasted on flange and web at the 50mm upper point from the column base, the flange and the web yielded when the column base moment reached the maximum. And then a part of the covering concrete already crushed. So calculating with decreasing of concrete compressive strength is necessary in order to estimate the flexural strength of composite columns conservatively by the superposed strength. As a result of investigating the decreasing value, it should be taken to 0.85. The flexural strength of RC columns and that of SRC columns



(a) S15 series (b) S30 series (c) B30 series (d) S50 series

Fig. 7 Comparison of experimental strength and superposed strength



(a) Plastic deformation capacity (b) Ductility factor corresponding to criterion of flexural strength (c) Ductility factor corresponding to criterion of shrinkage of column length

Fig. 8 Deformation capacity

are estimated conservatively by the superposed strength Mpc2 calculating with the steel yield stress and concrete compressive strength.

Deformation Capacity

Equating the absorbed energy dissipation of columns up to an assumed criterion, with the area enclosed with the idealized elastic-plastic model as shown in Fig. 8 (a), derives the equivalent ductility factor □. In Figure 8 (a), ***Mpc*** is the superposed strength calculated in 4.2 and ***Rpc*** is the elastic rotation angle when the columns base moment reaches ***Mpc*** on assumed cantilever model.

Ductility Factor Corresponding to Criterion of Flexural Strength:

Criterion of Flexural Strength is placed as flexural strength of column base moment deteriorated to 0.9 times of the maximum strength. Ductility factor $\square_{m90}$ corresponding to this criterion is shown in Fig. 8 (b). The ductility factors $\square_{m90}$ of the core steel composite columns were from three to four times as much as those of RC column, under axial load ratio ***n*** equal to 0.3. And the value of the ductility factors $\square_{m90}$ of the composite columns was twice or third times as much as those of RC column under limiting axial force prescribed by AIJ standard. And the most suitable quantities of encased steel for the composite cross section exists under large axial load such as limiting axial force.

Ductility Factor Corresponding to Criterion to Shrinkage:

Criterion of the shrinkage is placed as the maximum shrinkage reached 1/100 of the column length. RC columns collapsed and could not hold axial load on this criterion, but the core steel composite columns and SRC columns were able to hold axial load. The ductility factor $\square_{d1}$ corresponding to this criterion is shown in Fig. 8 (c). The ductility factors $\square_{d1}$ of the composite columns corresponding to this criterion were from 1.5 to two times as much as those of RC columns. And from two to three times under limiting axial force prescribed by AIJ standard.

The ductility factor of the core steel composite columns with the compact circular steel is larger than that with the H shaped steel.

CONCLUSIONS

It has become clear from the test results that:

1. Encasing core steel makes the deformation capacity of RC columns to be large.
2. Effect of the ratio ***ns*** of acting axial force to compressive strength of encased core steel, on the behavior of the core steel composite columns is large. And the columns on ***ns*** being under 1.5 show large earthquake resistant properties.
3. Flexural strength of the composite columns with core steel is estimated conservatively by superposed strength using yield steel stress and 0.85 times of concrete compressive strength
4. Encased core steel with ***ns*** equal to 1.5 was the best suitable for composite columns to make large earthquake resistant properties.
5. Using compact core steel is more suitable than H shaped steel, on the point of controlling shrinkage little.

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