EXPERIMENTAL STUDY ON SEISMIC CAPACITY OF COMPOSITE COLUMN FORMED SRC AND RC PART

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SUMMARY

In a structurally non-uniform building comprising a lower part by SRC construction and an upper part by RC construction, generally columns of a floor where the structure type changes from SRC construction to RC construction are subject to a large fluctuating axial force and a large shear force. Therefore, structural performance under a high axial force (compression, tension) is important. However, no structural test has been made under these conditions so far. Thus, we confirmed the following through two series of tests under a fluctuating axial force:

- A sufficient anchorage can be secured when the maximum strain of special lateral reinforcement far exceeds the yield strain, the special lateral reinforcement pierces web plates, and an overlap length of 10d is secured.

- Under a compressive axial force, special lateral reinforcement is effective for binding concrete and useful for preventing the buckling of main reinforcement. And, under a tensile axial force, it increases the anchorage of cut-off bars.

- Crest plates are effective for displaying the maximum strength and for controlling a degradation in strength after the maximum strength.

- Under a compressive axial force, the flexural yield strength of SRC-RC switching columns reaches the generalized cumulative strength of SRC columns.

- Under a tensile axial force, too, the flexural yield strength of SRC-RC switching columns reaches the generalized superposed strength of SRC columns by making the length of the projected steel frame close to the inflection point height.

From the above, the yield strength deformation performance of SRC-RC switching columns under high compression and high tension can be secured by a reinforcement method using special lateral reinforcement and crest plate.

INTRODUCTION

In constructing a super high-rise collective residential building, a structure type of super high-rise RC construction, effective in terms of both the cost and construction period, has become popular in Japan. In redevelopment areas, particularly station fronts, the construction period and the construction yard to be required during construction are the problems. To solve these problems, a method of simultaneously executing SRC and RC floors using the lower SRC floors as a construction yard for the upper RC floors, as shown in Figure 1, can be considered. Normally the floor of structure type change is an upper floor where stress is small, and columns of the floor are composite columns with steel frames extending up to a half of the floor height, the lower part by SRC construction and the upper part by RC construction. However, in terms of the construction period it is not

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so advantageous when the structure type change is made at an upper floor with small stress, but changing the structure type at a lower floor, if possible, leads to a reduction in the construction period. In this study, structural problems [3, 4, 5] in changing from SRC to RC construction at a lower floor were extracted and solved.

The problems are generally classified into two. One is that columns that are subject to a large axial force inevitably have large sectional areas, because of a structural rule not to limit the axial force below 0.4 Nu (Nu: maximum yield strength against compressive axial force) as a minimum condition to allow ductility in Japanese SRC regulations. The other is that no design method has been established for changing structurally different SRC and RC members in the middle of columns. Thus, focusing on the second problem, this paper deals with experimental studies about the strength deformation performance of columns changing SRC to RC under a fluctuating axial force, discusses the results and give considerations based on such results.

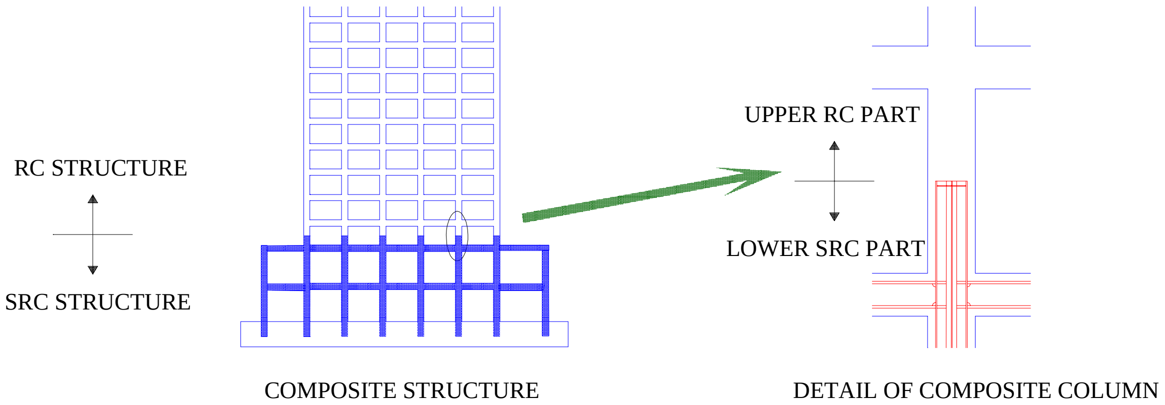


Figure 1

OBJECT OF EXPERIMENT

As experimental studies of members for changing SRC to RC construction, experiments on beam members [1, 2] were made and design methods were proposed in the past. According to these, a strength deformation performance equal to that of SRC members is obtained when the flexural strength of RC at the changing part is 1.4 times or more that of SRC at the beam end, and the strength of SRC members can be assured when it is 1.1 times or more. Also, for transmission of stress from RC part to steel frame, mechanically the force of compression flux of concrete is received by the bearing force of steel frame and the reaction force is with drawn into the members by the lateral reinforcement of RC, presenting an assumed condition allowing the calculation of shear reinforcement quantity. The experiments we made is to confirm the effectiveness of reinforcing method through repeated alternate positive-negative loading under a fluctuating axial force condition with high compression and high tension based on the results of these experiments.

SPECIMEN OVERVIEW

To secure the deformation performance, preliminary experiments with parameters of the reinforcement methods of reinforced concrete part and steel frame [Series I] and experiments with parameters of member strength, steel frame shape, etc. in addition to these reinforcement methods [Series II] were made. To secure the same inflection point height under a fluctuating axial force, specimens assuming side columns are designed in shape as top-bottom symmetrical models with SRC column bases and column heads and with RC column in the center part of specimen, and specimens for constant axial loading assuming center columns are designed as a SRC column base, RC column head model.

3.1 Series I

In this series, two specimens assuming side columns with large axial force fluctuation and one specimen assuming center columns with a constant axial force were planned. The basic specimen (HH1) assumes side columns, and the bar arrangement of central RC section is 24-D10 (including 4 core bars). The section of end SRC is a cross-shaped steel frame; a crest plate is arranged at its top end, bar arrangement done by anchoring 12

pieces of four corners in the stub and other 12 pieces are cut off at the column end. For the other side column specimen (HH2) and the center column specimen (HH3), the following two points are reinforced:
(1) To increase the binding effect of concrete, special lateral reinforcement is used by inserting a U-shaped bar into a hole provided in the steel frame web.
(2) A rib plate (PL-4.5) having two roles, to prevent the buckling of compress ion flange of steel frame and to receive the compression strut of concrete, is provided.

3.2 Series II

In this series, one employing reinforcement method (1) of Series I is used as a basic specimen (HH4), with the following eight items as parameters: Length of projected steel frame (HH5), flange width (HH6), quantity of anchoring main reinforcement (HH7), with/without crest plate (HH8), thickness of crest plate (HH9), quantity of lateral reinforcement (HH10), concrete strength (HH11), and shape of steel frame (HH12).

These specimens assuming side columns and one assuming center columns (HH13), 10 specimens in all, were planned. Specimens are listed in Table 1; the strength and specification of materials used are shown in Table 2: and sectional shapes in Figure 2.

Table 1: Specimen List

(Unit: mm)

SPECIMEN	M/QD	RC part		SRC part			
		Main bar	Hoop	Main bar	Hoop	Steel	Add. Device
Series Ⅰ							
HH1	1.93	24-D10	4-D6@50	12-D10	2-D6@50	2H-200*75*5.5*8 Ls=300	Crest PL-6
HH2							Crest PL-6 Special Hoop @50 rib PL-4.5@100
HH3	1.00	12-D10		4-D10			
Series Ⅱ							
HH4	2.00	24-D10	4-D5@40	12-D10	2-D5@40	2H-200*75*5.5*8 Ls=300	Crest PL-6 Special Hoop@40
HH5						2H-200*75*5.5*8 Ls=450	
HH6						2H-200*45*5.5*8 Ls=300	
HH7				20-D10		2H-200*75*5.5*8 Ls=300	
HH8				12-D10			Special Hoop@40
HH9							Crest PL-3 Special Hoop@40
HH10					2-D5@60		Crest PL-6 Special Hoop@60
HH11					2-D5@40		Crest PL-6 Special Hoop@40
HH12						CT-127*75*5.5*8 Ls=300	
HH13	1.00					2H-200*75*5.5*8 Ls=300	

Hoop : Lateral Reinforcement

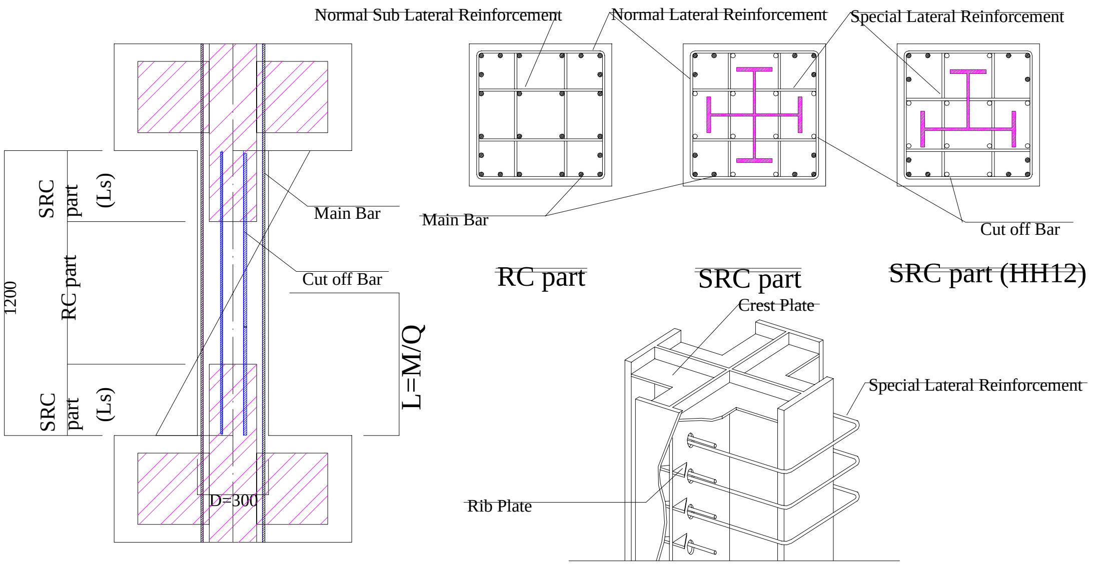


Figure 2.

Table 2: Material Strength

a) concrete (Unit : N/mm^2)

Design strength	Specimen	Compressive strength	Tensile strength	Young's Modulus
Fc60	HH1～HH3	60.2	3.16	3.33×10^4
Fc60	HH4～HH10,HH13	55.6	4.11	3.20×10^4
Fc36	HH11,HH12	39.0	2.84	$2,74\times10^4$

b) steel & bar (Unit : N/mm^2)

Position	Kind of Material	Specimen	Yield Strength	Young's Modulus
Flange(PL-8)	SM490A	HH1～HH13	414	1.97×10^5
Web(PL-5.5)	SM490A	HH1～HH13	450	1.93×10^5
Crest Plate (PL-3 or 6)	SS400	HH1～HH7 HH9～HH13	381	1.91×10^5
Rib Plate(PL-4.5)	SS400	HH2,HH3	365	1.89×10^5
Main Bar(D10)	SD490	HH1～HH13	660	1.84×10^5
Hoop(D6)	SD785	HH1～HH3	1,178	1.99×10^5
Hoop(D5)	SD590	HH4～HH13	557	1.88×10^5

LOADING METHOD

Repeated alternate positive-negative loading is applied. For specimens given a fluctuating axial force, the axial force is 0.5 cNu (cNu: maximum yield strength of RC member against compressive axial force). The tensile axial force is 1.0tNu for specimen HH7 (tNu: maximum yield strength of RC member against tensile axial force), and 0.7 tNu for others. The fluctuating axial force, by switching from compressive force to tensile force, is applied at a stroke when the shear force is zero. In addition, for specimens given a constant axial force, a compressive force of 0.2 cNu is given to specimen HH3, and 0.3 cNu to specimen HH13.

The loading system is outlined in Figure 3, and the standard loading history is shown in Figure 4.

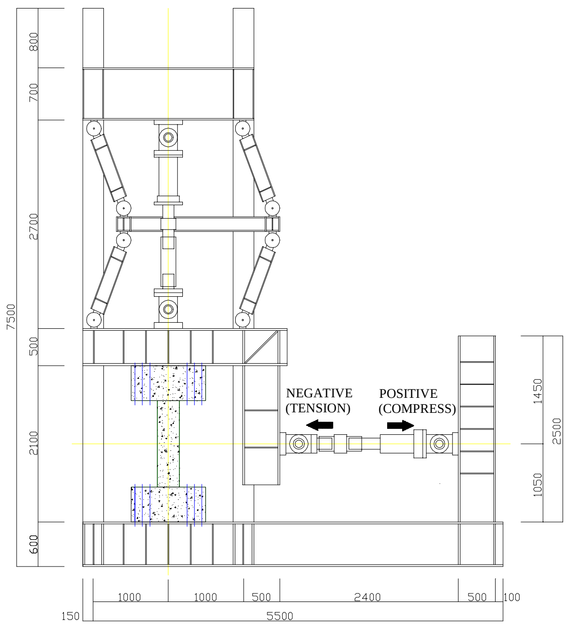


Figure 3: LOAD DEVICE

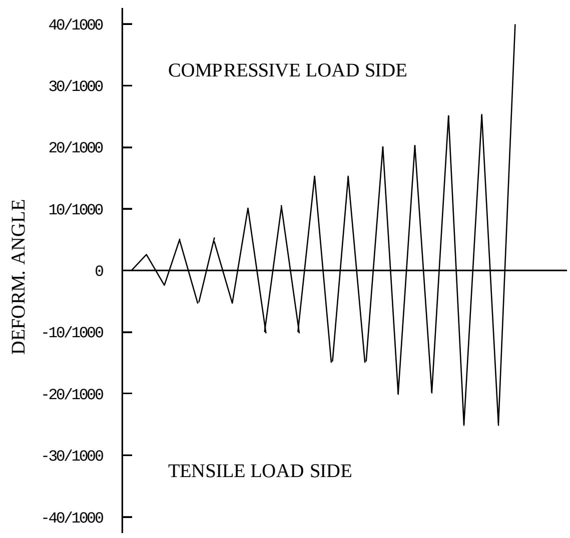


Figure 4: LOAD CYCLE

TEST RESULTS

5.1 Series I

Table 3 shows the test results of specimens HH1 to HH3. When subjected to a compressive axial force, all the specimens HH1-HH3 show test values above the maximum shear force by the generalized superposed strength system. For the tensile axial force, however, specimen HH1 displayed strength of 82% only with regard to a yield strength at which RC is assumed to cause flexural yield at switching in the experiment. Specimen HH2 did not reach the yield strength of SRC ends either, but it displayed strength 20% above the yield strength at which RC is assumed to cause flexural yield at switching in the experiment. As a reason it can be assumed that the anchorage of cut-off bars of specimen HH1 was insufficient at the section of SRC-RC switching, while the anchorage of cut-off bars of specimen HH2 by special lateral reinforcement was increased. Figure 5 shows the Q-R relationship of specimens HH1 and HH2. It shows that both specimens present the strength larger than calculation values under a compressive axial force, but for a degradation in strength after the maximum strength, it is more noticeable in specimen HH1, indicating a big difference in deformation performance between both specimens. The favorable deformation performance of specimen HH2 is considered a great contribution of special lateral reinforcement. Figure 6 shows the Q-R relationship of specimen HH3. Under a constant axial force (0.2cNu), it retained a sufficient yield strength even with R=30/1000rad.

Table 3: Maximum Strength (Series I)

(Unit : kN)

Specimen	Axial force at maximum strength	Maximum shear force			Remark
		Test value	Calculation value	Experiment/ Calculation	
HH1	3314.6	538.4	503.1	1.07	*1)
	-541.3	-196.1	-239.3	0.82	*2)
			-340.3	0.58	*1)
HH2	3347.0	625.7	502.1	1.25	*1)
	-540.3	-286.4	-239.3	1.20	*2)
			-340.3	0.84	*1)
HH3	1373.9	814.9	769.8	1.06	*3)

*1) Calculation values are evaluated by the generalized superposed strength of SRC ends.
*2) Calculation values are evaluated by extending the RC yield strength at the RC-SRC switching part to the end part.
*3) Calculation values are evaluated by recognizing that RC and SRC ends respectively reached the flexural yield strength.

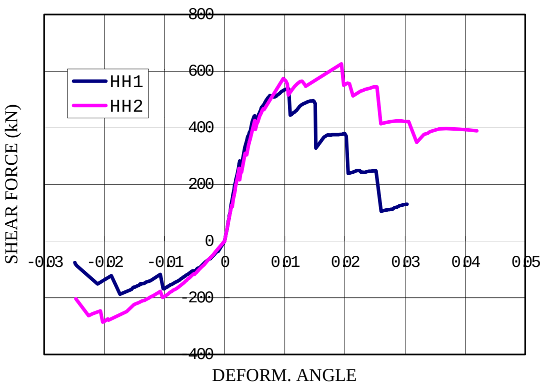


Figure 5: Q-R Relationship of HH1 and HH2

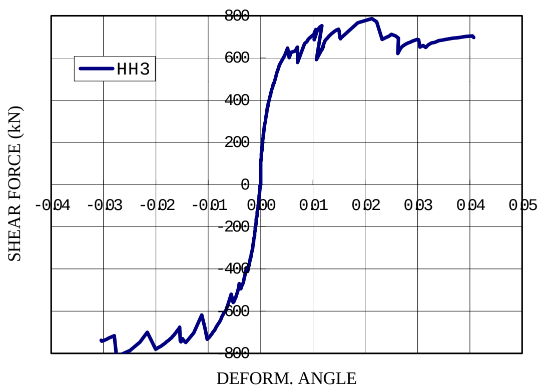


Figure 6: Q-R Relationship of HH3

5.2 Series II

1) Flexural strength of compression side

Except for HH10 and HH13, specimens' maximum flexural strength reaches the generalized superposed strength. Specimen HH8 also reaches the generalized superposed strength, but it being slightly lower than other specimens indicates a large degradation in strength after the maximum strength. This shows the effectiveness of crest plates for securing the strength deformation performance. Specimen HH10 does not reach the generalized superposed strength, but buckling of main bars in the early stage can be considered due to the wide spacing of lateral reinforcement at column bases. Specimen HH13 caused shear breakdown.

2) Flexural strength of tension side

Under a tensile axial force, all specimens except HH5 show strengths between the simple superposed strength and the generalized superposed strength. This indicates that the position of flexural yield at tensile axial force is not the end part but the SRC-RC switching part, so, to display the generalized superposed strength of the section

of SRC end, the increase in yield strength referred to in the reference [1] is required at the SRC-RC switching part. However, since specimen HH5 of which the projection of steel frame is 2/3 in length of the inflection point height reaches the generalized superposed strength at the section of SRC end, it does not require the reinforcement referred to in the reference [1]. Specimen HH8 having no crest plate, compared with other specimens, is low in strength even under tensile axial loading. Specimen HH10 shows a degradation in strength at and after 20/1000rad. This indicates an influence of spacing of lateral reinforcement on the anchorage of main bars at the section of SRC-RC switching, in addition to prevention of main bar buckling.
Table 4 shows maximum values of specimens, Figure 7 shows the N-M yield strength curves and maximum values of major specimens, and Figure 8 shows the Q-R curves of major specimens.

Table 4: Maximum Strength (Series II)

(Unit: kN)

Specimen	Axial force at maximum strength	Maximum strength			Remark
		Test value	Calculation value	Experiment/ Calculation	
HH4	3158.2	547.2	490.9	1.11	*1)
	-635.0	-255.5	-197.5	1.29	*2)
			-326.6	0.78	*1)
HH5	3194.8	528.7	489.7	1.08	*1)
	-695.1	-333.9	-340.9	0.98	*2)
			-317.0	1.05	*1)
HH6	3173.9	556.1	450.1	1.24	*1)
	-661.2	-249.6	-185.6	1.34	*2)
			-258.7	0.96	*1)
HH7	3142.6	579.6	541.9	1.07	*1)
	-960.4	-194.8	-51.1	3.81	*2)
			-324.8	0.60	*1)
HH8	3136.0	502.3	493.2	1.02	*1)
	-686.0	-224.2	-198.2	1.13	*2)
			-318.4	0.70	*1)
HH9	3142.6	537.5	491.4	1.09	*1)
	-633.8	-252.5	-215.2	1.17	*2)
			-326.8	0.77	*1)
HH10	3134.7	485.6	491.6	0.99	*1)
	-595.8	-237.0	-233.5	1.01	*2)
			-332.8	0.71	*1)
HH11	2643.4	490.5	385.2	1.27	*1)
	-619.4	-231.1	-189.9	1.22	*2)
			-313.1	0.74	*1)
HH12	2653.8	483.6	384.9	1.26	*1)
	-653.4	-222.3	-179.0	1.24	*2)
			-305.5	0.73	*1)
HH13	1673.8	787.1	667.5	1.18	*3)
	1770.5	826.3	655.2	1.26	*3)

*1) Calculation values are evaluated by the generalized superposed strength of SRC ends.
*2) Calculation values are evaluated by extending the RC yield strength at the RC-SRC switching part to the end part.
*3) Calculation values are evaluated by the ultimate shear strength of RC part..

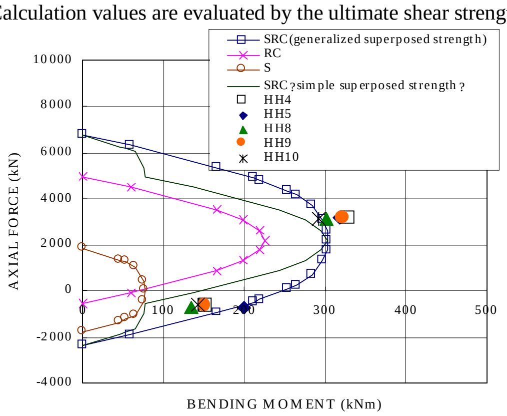


Figure 7: N-M Yield Strength Curves and Maximum Values

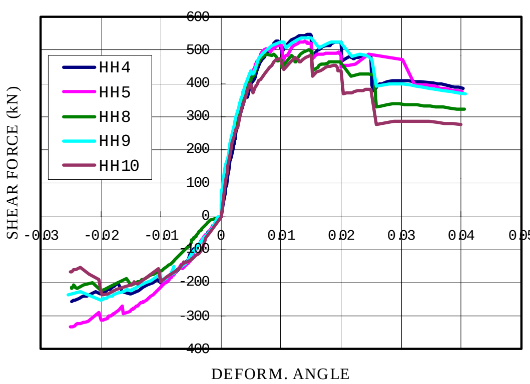


Figure 8: Q-R Relationship

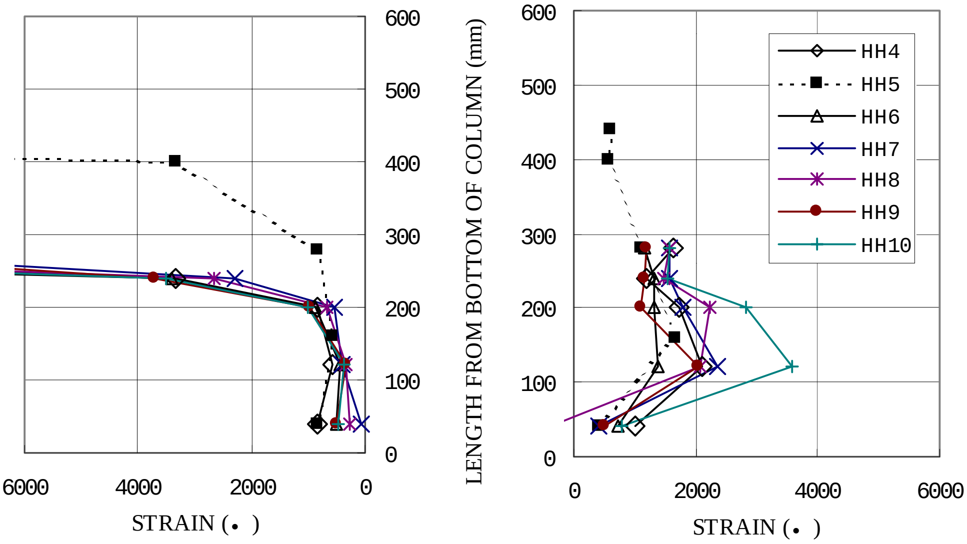
3) Shear force

For specimen HH13 that showed shear breakdown, the maximum shear force is above the ultimate shear strength of RC part and no degradation due to inserting steel frames into RC column bases is seen in maximum shear strength of RC columns. Figure 9 shows the strain distribution of special lateral reinforcement at maximum shear force. Under a tensile axial force, the strain distribution of special lateral reinforcement grows in a range of about 10 cm from the top end of steel frame toward the column base. In this instance, strain exceeds the yield strain, indicating that special lateral reinforcement is thoroughly anchored. The shear force borne by ordinary lateral reinforcement and special lateral reinforcement is calculated by the following equation and the result is in Figure 10.

$$Qw = b\, j_t\, p_w\, E\, e$$

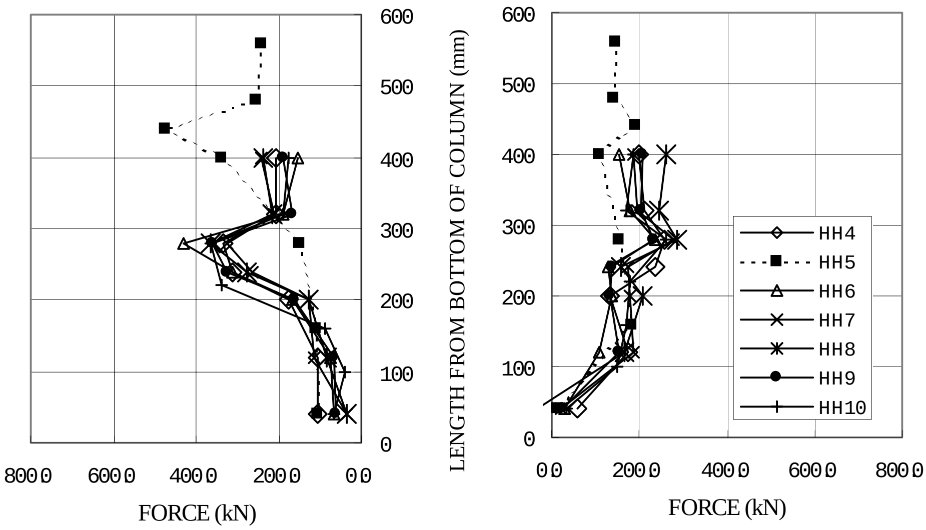
Where, b: column width, j_t: main bar spacing, p_w: lateral reinforcement ratio, E: young's modulus, and e: strain of lateral reinforcement

The shear force borne by lateral reinforcement shows nearly constant values along the column member axis under a compressive axial force, while the shear force that exists under a tensile axial force and is borne by the central RC section shows an increase in a range of 1/2 the steel frame height from top to end of the steel frame and a decrease in the column base part below the former.



a) TENSILE LOAD SIDE b) COMPRESSIVE LOAD SIDE

Figure 9: STRAIN OF SPECIAL LATERAL REINFORCEMENT



a) TENSILE LOAD SIDE b) COMPRESSIVE LOAD SIDE

Figure 10: SHEAR FORCE OF LATERAL REINFORCEMENT

CONCLUTIONS

Through the two series of experiments conducted in this study, the following can be assumed:

- A sufficient anchorage can be secured when the maximum strain of special lateral reinforcement far exceeds the yield strain, the special lateral reinforcement pierces web plates, and an overlap length of 10d is secured.
- Under a compressive axial force, special lateral reinforcement is effective for binding concrete and useful for preventing the buckling of main reinforcement. And, under a tensile axial force, it increases the anchorage of cut-off bars.
- Crest plates are effective for displaying the maximum strength and for controlling a degradation in strength after the maximum strength.
- Under a compressive axial force, the flexural yield strength of SRC-RC switching columns reaches the generalized superposed strength of SRC columns.
- Under a tensile axial force, too, the flexural yield strength of SRC-RC switching columns reaches the generalized cumulative strength of SRC columns by making the length of projected steel frame close to the inflection point height.

From the above, the yield strength deformation performance of SRC-RC switching columns under high compression and high tension can be secured by a reinforcement method using special lateral reinforcement and crest plate.

REFERENCE

[1] Fukuchi, Tokari. (1988), “Experimental Study on SRC-RC Beam Joints.”, *Proceedings of the Japan Concrete Institute,* 10-3.

[2] Suzuki, Nishihara, Matsumoto, Tabata. (1997), “Flexural Shear Experiment of SRC-End, RC-center Composite Beams.”, *Technology Institute Report Vol.3,* Ando Construction.

[3] Konno, Imaizumi, et al. (1998a), “Experimental Studies on High-Rise Buildings with Lowerfloor Composed of SRC Structure. Part 1 – Outline of the Tests about Deformation Capacity of SRC Columns.”, *Summaries of technical papers of annual meeting Architectural Institute of Japan*, September.

[4] Konno, Imaizumi, et al. (1998b), “Experimental Studies on High-Rise Buildings with Lowerfloor Composed of SRC Structure. Part 2 – Results and Examination of Experimentation.”, *Summaries of technical papers of annual meeting Architectural Institute of Japan,* September.

[5] Sugiyama, Konno, et al. (1998c), “Experimental Studies on High-Rise Buildings with Lowerfloor Composed of SRC Structure. Part 3 – Outline of the Tests about Structural Performance of Column that is Composed of SRC and RC, and Test Results of Base Experimentation.”, *Summaries of technical papers of annual meeting Architectural Institute of Japan*, September.