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EXPERIMENTAL STUDY ON DEFORMATION CAPACITY OF WALL COLUMNS AFTER FLEXURAL YIELDING

Hisahiro HIRAISHI[1], Masaomi TESHIGAWARA[1], Toshikazu KAWASHIMA[1]

Tadaharu NAKACHI[2], Hiroyuki TUBOSAKI[3], and Masao OGUMA[4]

[1]Building Research Institute, Ministry of Construction, JAPAN
[2]Technical Research Institute, Hazama-Gumi, Ltd., JAPAN
[3]Technical Research Institute, Penta-Ocean Construction Company, JAPAN
[4]Technical Research Institute, Ohmoto-Gumi Co. ,Ltd. ,JAPAN

SUMMARY

An experimental study on the deformation capacity after flexural yielding of columns with the large depth(D)-to-width(b) ratio(D/b), referred to as wall columns in this paper, was performed. Thirteen wall columns were tested under constant axial load and cyclic lateral displacements. The main variables were as follows:

1) Confinement in compressive zone
2) The depth-to-width ratio
3) The presence of transverse wall in wall columns
4) Axial stress level

The test data indicated that confinement in compressive zone depending on the axial stress level and the depth-to-width ratio were very important to maintain load carrying capacity in the large deflections range.

INTRODUCTION

Research and development on a structural system so called as "High-Rise Frame Structure with Wall Columns", for the 10-15 storied apartment houses, have been performed (Ref. 1). This structural system has frames consisting of wall columns and beams in the ridge direction and shear walls in the span direction (see Fig. 1). In the ridge direction, the frames are designed to absorb and dissipate the seismic energy in large plastic deformation of the wall columns on the first story as well as beams of each story. In this study, to evaluate the deformation capacity of such wall columns, nine wall columns with a rectangular section and four wall columns with transverse walls representing the panel wall of the shear wall in the span direction were tested. This study intends to obtain basic data for the guideline in order to ensure that wall columns behave in a ductile manner after flexural yielding and have the required deformation capacity.

OUTLINE OF TEST PROGRAM

All of the specimens were designed to represent the wall columns in the first story of a fifteen storied apartment house, and tested under constant axial load, and under the cyclic lateral displacements at the height corresponding to the center of beam in the second floor(see Fig. 2). The shear span to depth ratio of wall columns is 2.5   3.0. This test program consisted of the following three series;

Series 1 : Study on Effects of Confinement in Ccompressive Zone Four specimens were designed to be a half-scale representations (Ref. 2). Specimen No.1 is without special confining reinforcement in edge zone. The shape of special confining reinforcement is □ -type reinforcement for Specimen No.2, ⊞ -type reinforcement for Specimen No.3, and ⊡⊡ -type reinforcement for Specimen No.4. Details of the specimens are listed in Table 1. According to preliminary compressive prism tests, ⊞ -type reinforcement is more effective on improving decrease in the load carrying capacity in large strain range than □ -type reinforcement (see. Fig. 3). The axial stress is 0.35 Fc for all specimens, where Fc is 240 kg/cm, and the specified concrete strength for design. The depth-to-width ratio is 5 for all. The measured material properties of steel and concrete are listed in Table 2. Figure 4 shows the reinforcement arrangement of Specimen No.2.

The typical observed behavior of specimens are as follows; The first horizontal crack was observed near the base when the drift angle(R) of the first story was about 1/1300 radian. As the lateral displacement increased, inclined shear cracks occurred successively and extended toward the compressive zone at the base. At the displacement when R was about 1/400 radian, longitudinal cracks occurred along the longitudinal compression bars near the base. And when R was about 1/200 radian, all of the tensile bars at the base yielded. Then the compressive failure of concrete at the base developed from the edge to inner side, and finally specimens could not sustain the axial load. The typical crack patterns at the maximum load and at the final are shown in Fig. 5. Test results indicated that the deformation capacity of specimens was almost in proportion to the confining reinforcement.

Series 2 : Study on Effects of the Depth-to-Width Ratio The depth-to-width ratio was designed to be 3 for Specimen No.5, 5 for Specimens No.6 and No.9, and 8 for Specimens No.7 and No.8 (Ref. 3). The axial stress is 0.35 Fc for Specimens No.5, No.6, No.7, and No.8, and 0.60 Fc for Specimen No.9. Specimen No.8 has the transverse ties to confine concrete in not only edge zone, but also central zone. The scale of representation is 1/3.87 for the specimens of which the depth-to-width ratio is 3, 1/3.0 for the specimens of which the ratio is 5, and 1/2.37 for the specimens of which the ratio is 8 (see Fig. 6 and Tables 3 and 4).

The behavior of the specimens in this series was almost same as that of the specimens in series 1. However, in the specimens subjected to the high axial stress, the number of the observed crack was less than that of the specimens subjected to the low axial stress. At the final stage, the specimens except Specimen No.5 could not sustain the axial load due to compressive failure of concrete at the base, and reached the deformation capacity just after this.

Series 3 : Study on Effects of the Presence of Transverse Walls Specimen P1 with +-shape cross section is identical to Specimen No.6 in series 2 except the presence of transverse walls at the middle, and also specimen P2 is identical to Specimen No.9 in series 2. Specimen P3 has the sub-hoop to confine, in addition to the reinforcement of Specimen P2. Specimen T1 with T-shape cross section was designed to be 1/3-scale representation of corner wall columns in the ridge direction (Ref. 3). The axial stress is 0.60 Fc when the transverse wall is subjected to compression, and 0 when the transverse wall is subjected to tension. The depth-to width ratio of Specimen T1 is 2.5. The length of transverse wall in all of the specimens is 6.0t, where t is the thickness of transverse wall. (See Fig. 7 and Table 5 and 6. The used steel in series 3 is the same one in series 2.)

Specimens behaved in the almost same manner as specimens in series 1 and 2 before the maximum load. But then because of the presence of transverse walls, the lateral load carrying capacity of specimens decreased gradually, and they could hold the axial load even at the end of test. Especially, Specimen T1 behaved in a excellent ductile manner until the end of the loading (see Fig. 8).

COMPARISON OF TEST RESULTS

According to the test results, it was common to all of the specimens in series 1, 2, and 3 that the vertical displacement measured at the second floor along the tensile bars was almost in proportion to the lateral displacements at the height while the specimens behaved in the stable manner without a great decrease in the load carrying capacity. The critical point in this proportional relation is referred to as stable limit in this paper (Ref. 4).

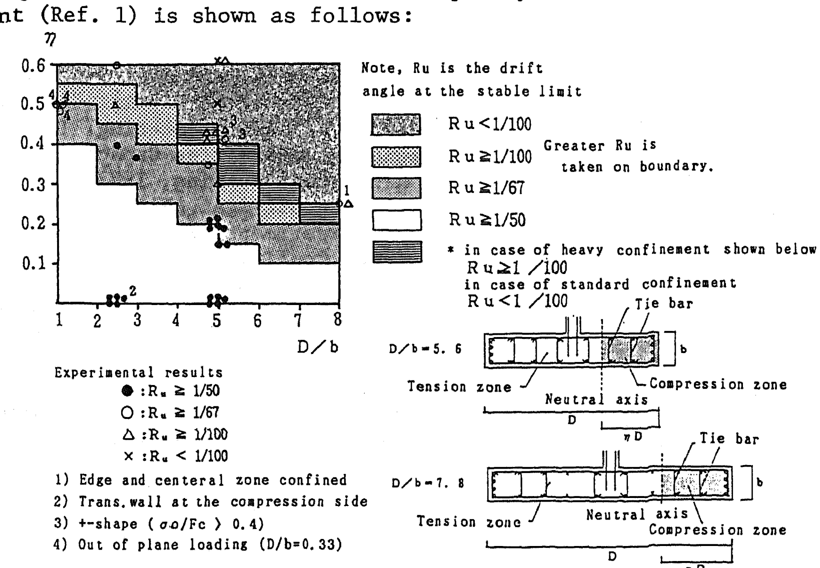
The envelope of the average shear stress (τ) versus the drift angle of the first story (R) relation obtained from tests except Specimen T1 are shown in Figs. 9 , 12.

Concerning the drift angle (Ru) at this stable limit, the following results were obtained by the comparisons of test results:

1) Confinement of concrete in the compressive edge zone is very effective to increase the value of Ru (see Fig. 9). Extending of confined zone is also effective, especially in the wall columns with large depth-to-width ratio and the wall columns subjected to high axial load (see Fig. 10).
2) The value of Ru decreases according to increase in the depth-to-width ratio (see Fig. 11). The reason for it is mostly that the ratio of the area of the unconfined zone to the gross sectional area tends to be large in the wall columns with the large depth-to-width ratio.
3) The value of Ru of the wall column with transverse walls is larger than that of the wall column with a rectangular section when the transverse walls are in the compression side, but transverse walls in the tension side are not effective to increase the value of Ru (see Fig. 12).
4) Arrangement of transverse ties or sub-ties in the central zone and the presence of transverse walls at the center of section are both effective to prevent wall columns from a sudden and great decrease in the load carrying capacity after the stable limit (see Figs. 10 and 12).

A PROROSED DESIGN CHART FOR THE DEFORMATION CAPACITY OF WALL COLUMNS WITH STANDARD CONFINEMENT

A design chart for the deformation capacity of wall columns with standard confinement (Ref. 1) is shown as follows:



Design Chart on Deformation Capacity of Wall Columns with standard confinement where, $\eta = \sigma_0/Fc$ (σ_0=N/(bD));

N=axial load

Fc=the specified concrete strength for design

Comments:

1) For +-shape wall columns, η is taken as 0.4 in the cart when the real exceeds 0.4.
2) For T-shape wall columns, σ_0 =N/(bD) when the transverse wall is in compression side, and σ_0=N/(BD) when the transverse wall is in the tension side (B=effective flange width).
3) For out of plane loading, D/b=1 in the chart for any real value of D/b.

The experimental values obtained by authors and others (Ref. 5, 6, 7, and 8) agree well with the proposed ductility rank.

CONCLUSIONS

Following findings were described from this study:

1) In wall columns of "High-Rise Frame Structure with Wall Columns", the amount of confinement in compressive zone, the extent of the confined zone, the depth-to-width ratio, the axial stress level, and the presence of transverse walls significantly affect the deformation capacity of such wall columns.
2) For wall columns with standard confinement, the deformation capacity of the wall columns can be evaluated by the combination of the depth-to-width ratio and the axial stress level. And the proposed design chart is in good agreement with test results, including the effect of the transverse walls.

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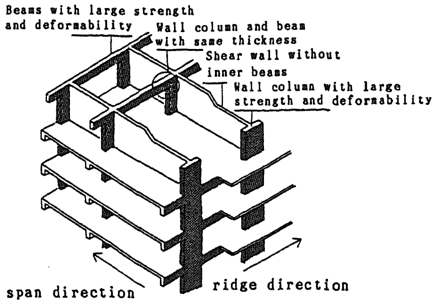
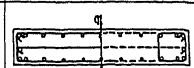
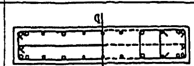
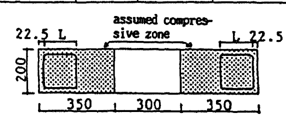


Fig.1 Outline of Structual System of the R/C Frame Structure with Wall Column

Table 1 Specimens in Series 1

	section of specimens		lateral reinf. as confinement	area of confine-ment L	reinf.	common factor
	lateral reinf.	corner lateral reinf. as con-finement				
NO.1			———	———	main reinf. 14-D13 10-D8 P_g=1.14% lateral reinf. 3-D6 @50 P_w=0.96%	section b=20 cm D=100 cm D/b=5 M/QD≒2.5 σ_0/F_c≒0.35
NO.2			D5@50	155 mm		
NO.3			5φ@50 welded wire mesh	155 mm		
NO.4			D5@50 D5@50	285 mm		



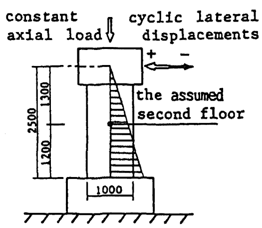


Fig.2 loading system

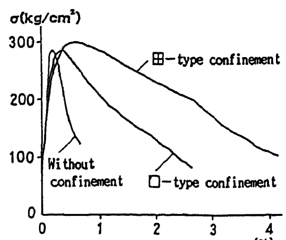


Fig. 3
Stress versus Strain Relationship
(Centrally Loaded)

Table 2
Measured Material Property

reinforcement

	σ_y (t/cm²)	σ_{max} (t/cm²)	ε_{max} (%)
5φ	5.12	5.63	13.1
D5	3.98	4.53	16.4
D6	4.30	5.71	22.3
D8	3.63	5.19	24.7
D10	3.89	5.66	26.2
D13	3.83	5.62	25.2

concrete

fc' (kg/cm²)	E_c (kg/cm²)	ε_m (%)
235	1.94×10⁵	0.24

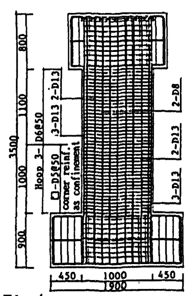


Fig.4
Reinforcement Arrangement
No.2

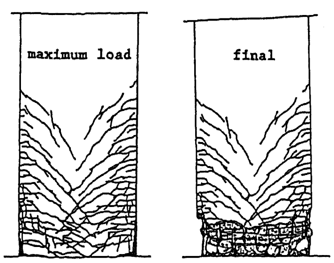


Fig. 5 Crack Patterns No.2

Tables 3 Specimens in Series 2

Specimen	D/b (cm)	b (cm)	D (cm)	H (cm)	h (cm)	M/QD	N (t)	σ_0/f_c'	Pg (%)	Pw (%)
No.5	3.0	13.5	40.5	115.0	62.0	2.84	46.19	0.37	1.41	0.87
No.6	5.0		67.5	183.6	80.0	2.72	77.21	0.37	1.17	
No.7	8.0		108.0	285.1	101.3	2.64	122.87	0.29	1.01	
No.8							123.08	0.29		
No.9	5.0		67.5	183.6	80.0	2.72	131.77	0.61	1.17	

Index

b:Width D:Depth H:Shear span h:Height of measuring point M/Qd:Ratio of shear span to depth N:Axial load fc':Mesured concrete strength of cylinder σ_0 :Axial stress 1000×N/(b×D) (kg/cm²) Pg:Ratio of longitudinal reinforcement Pw:Ratio of lateral shear reinforcement Ec:The modulus of elasticity of concrete(secant modulus at 1/3 of fc') ε_m :Strain at the maximum stress of concrete σ_y :Yield stress of steel σ_{max} :Maximum strength of steel ε_y :Yield strain of steel Es:The modulus of elasticity of steel ε_{max} :Maximum strain of steel

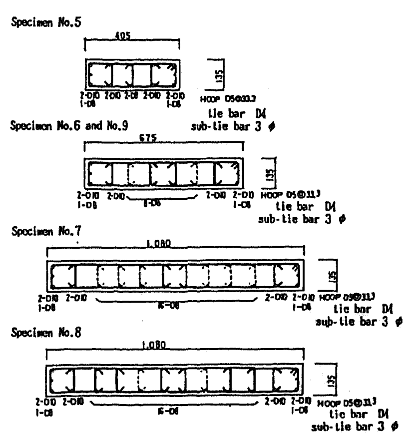


Fig. 6 Specimens in Series 2

Table 4 Measured Material Property

reinforcement

Nominal diameter	σ_y (kg/cm²)	σ_{max} (kg/cm²)	ε_y (%)	E_s (×10⁶ kg/cm²)
D10	3793	5343	0.22	1.78
D8	4207	6110	0.21	2.10
D5	2867	4402	0.35	1.95
D4	2800	4431	0.35	1.94
3φ	2547	5936	0.26	1.97

concrete

Specimen	fc' (kg/cm²)	ε_m (%)	E_c (×10⁵ kg/cm²)
No.5	228	0.213	1.95
No.6	231	0.206	1.99
No.7	290	0.212	2.26
No.8	285	0.214	2.25
No.9	236	0.204	2.04

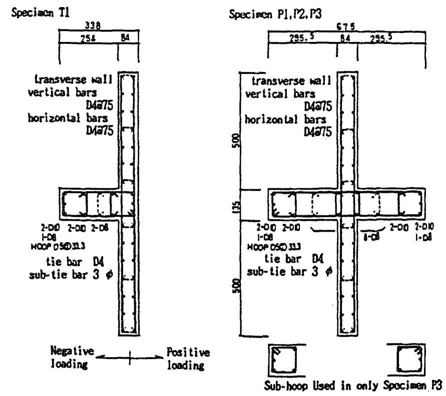


Fig. 7 Specimens in Series 3

Table 5 Specimens in Series 3

Specimen		D/b (cm)	b (cm)	D (cm)	H (cm)	h (cm)	M/QD	N (t)	σ_0/f_c'	Pg (%)	Pw (%)
P1								77.55	0.33		
P2		5.0		67.5			2.72	131.82	0.56	1.17	
P3			13.5		183.6	80.0		131.87	0.57		0.87
T1	pos.	2.5		33.8			5.43	66.35	0.57	1.69	
	neg.							0.75	0.01		

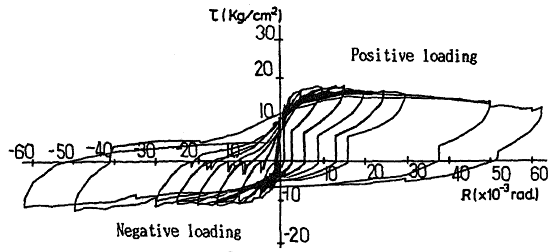


Fig.8 τ versus R
(Specimen T1)

Table 6 Measured Material Property

concrete

Specimen	fc' (kg/cm²)	ε_m (%)	E_c (×10⁵ kg/cm²)
P1	256	0.215	2.08
P2	257	0.213	2.18
P3	256	0.215	2.11
T1	257	0.210	2.35

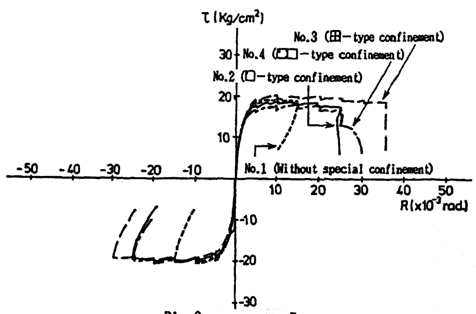


Fig.9 τ versus R
(Effect of confinement at edge zone)

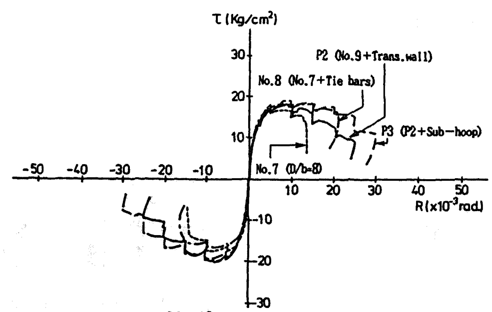


Fig.10 τ versus R
(Effect of tie bars and sub tie bars)

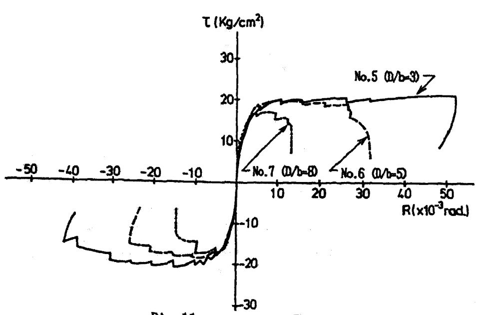


Fig.11 τ versus R
(Effect of ratio of depth to width)

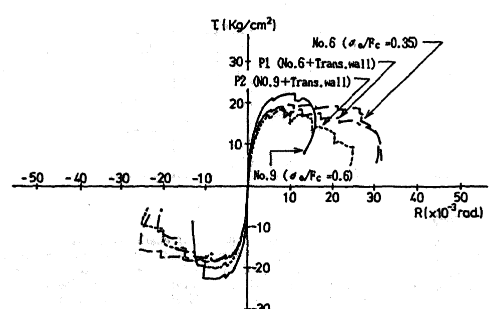


Fig.12 τ versus R
(Effect of transverse wall and axial stress level)