BEHAVIOR OF SQUARE CONCRETE-FILLED TUBULAR COLUMNS UNDER ECCENTRIC COMPRESSION WITH DOUBLE CURVATURE DEFLECTION

T. Fujinaga[1], H. Doi[2] and Y.P. Sun[3]

[1] *Associate Professor, Research Center for Urban Safety and Security, Kobe University, Kobe, Japan*
[2] *Graduate Student, Dept. of Architecture Security, Kobe University, Kobe, Japan*
[3] *Professor, Dept. of Architecture Security, Kobe University, Kobe, Japan*
Email: ftaka@kobe-u.ac.jp

ABSTRACT :

Objective of this paper is to present experimental information about the behavior of square CFT columns subjected to eccentric compression with double curvature deformation, to investigate validity of the current design formula for slender CFT columns, meanwhile to study its applicability to the middle-length columns.

Forty-three square CFT columns were fabricated and tested. The experimental variables among the tests are, 1) the buckling length to depth ratio of the column, 2) the concrete strength, 3) the moment gradient which deforms the column into double curvature, and 4) the eccentricity of the axial load.

Experimental results have indicated that the flexural strength of slender square CFT columns increases with the concrete strength and the moment gradient. Comparison between the test results and the theoretical ones predicted by the current design code has shown that for the slender square CFT columns with the buckling length to depth ratio of 20, the current design formula could satisfactorily well predict the flexural strength in spite of the degree of moment gradient. For the middle-length square CFT columns with the buckling length to depth ratio of 10, however, the current design formula overestimates the flexural strength when the column was under eccentric compression without moment gradient, i.e. under pure bending. On the other hand, for the middle-length columns under eccentric compression with moment gradient, the current design formula could be applied to evaluate their flexural strength with satisfactory accuracy.

KEYWORDS: Concrete-filled tubular column, Slenderness ratio, Double curvature deformation, Eccentric compression, Square column

1. INTRODUCTION

In the current design standards for steel reinforced concrete structures (hereafter refereed to as SRC standards) and the design guideline for concrete-filled tubular structures (hereafter refereed to as CFT guideline) by the Architectural Institute of Japan, the ultimate capacity of a steel-concrete composite slender column is calculated by simply superimpose the capacities of the concrete column and the steel column (AIJ, 2001; AIJ 1997). This simple superimposition method has such advantages as to keep continuity in the calculation of capacity for the short and the slender columns, and to contain the up-to-date information on each component, while the design formula becomes inevitably complicated. In addition, for the columns under concentric compression, the current design standards not only provides calculation equations for short and slender columns with slenderness ratio L_k/D less than and equal to 4 and larger than and equal to 12, respectively, but also recommendation for the calculation for the middle-length column with L_k/D varying between 4 and 12. However, for the columns under combined flexure and axial compression, the current SRC standards only shows capacity design formulae for short and slender column, and approximates the calculation for the middle-length column by applying the equation for the short column, hence resulting discontinuity in the design equation.

As to the capacity design equation for the middle-length columns with L_k/D varying between 4 and 12, Chung et al [2004] haves proposed a method to obtain the ultimate capacity via linear interpolation between the short and the slender column. This method, however, involves tedious calculation procedure, while its concept is clear and mathematical expression is simple. Kido et al [2005] have also proposed a theoretical approach for the calculation for the middle-length columns, but in their proposal there is still discontinuity in the calculation.

Fujinaga et al [2005] have recently proposed a simple method for the middle-length CFT column. In this method, the ultimate capacity for the middle-length columns is calculated by linear interpolation between the short and the slender column. Nevertheless, validity of their proposed method has not yet been verified with experimental results, neither the details of the moment modification factor C_M were mentioned in the method.

On the other hand, experimental study on effects of the moment gradient in the columns under double curvature on the ultimate capacity is scarce. While Kilpatrick et al [1997] have conducted systematic tests on the circular CFT columns to investigate the effect of the moment gradient; there are no experimental results of square CFT columns available. Furthermore, the moment amplification factor C_M due to the moment gradient for the middle-length column has not yet been investigated.

Objectives of the research is to obtain experimental information for the behavior of square CFT slender and middle-length columns under double curvature deformation, and to investigate validity of the previous calculation methods through comparing the experimental results with the theoretical predictions.

2. OUTLINES OF THE CURRENT DESIGN FORMULAE

2.1. Capacity Equations for the CFT Columns under Combined Flexure and Axial Compression

According to the current SRC standards, ultimate flexural strength of the CFT columns under combined flexural and axial load can be obtained as follows:

(1) For the column with L_k/D ratio less than and equal to 12

$$\left.\begin{aligned} N_U &= {}_cN_U + {}_sN_U \\ M_U &= {}_cM_U + {}_sM_U \end{aligned}\right\} \tag{2.1}$$

(2) For the column with L_k/D ratio larger than 12

(i) When $N_U < {}_cN_{cU}$ or $M_U > {}_sM_{U0}\,(1 - {}_cN_{cU} / N_k) / C_M$

$$\left.\begin{aligned} N_U &= {}_cN_U \\ M_U &= \left\{ {}_cM_U + {}_sM_{U0}\left(1 - \frac{{}_cN_U}{N_k}\right)\right\}\frac{1}{C_M} \end{aligned}\right\} \tag{2.2}$$

(ii) When $N_U > {}_cN_{cU}$ or $M_U < {}_sM_{U0}\,(1 - {}_cN_{cU} / N_k) / C_M$

$$\left.\begin{aligned} N_U &= {}_cN_{cU} + {}_sN_U \\ M_U &= {}_sM_U\left(1 - \frac{{}_cN_{cU}}{N_k}\right)\frac{1}{C_M} \end{aligned}\right\} \tag{2.3}$$

where ${}_cN_U$, ${}_cM_U$, ${}_sN_U$, ${}_sM_U$ are ultimate compressive strength and ultimate flexural strength of the filled concrete section, and those of the steel tube, respectively, ${}_sM_{U0}$ is ultimate flexural strength of steel tube under pure bending, N_k is Euler's buckling load of the CFT column, ${}_cN_{cU}$ is ultimate compressive strength of the filled concrete section, and C_M is the moment amplification factor.

2.2. Moment Amplification Factor C_M

In the current SRC standard, the formula recommended in the plastic design guideline by the AIJ [AIJ, 1975] is applied to calculate the moment amplification factor in the form of

$$C_M = 1 - 0.5\left(1 - \frac{M_2}{M_1}\right)\sqrt{\frac{N_U}{N_k}} \geq 0.25 \quad , \quad N_k = \frac{\pi^2}{l_k{}^2}\left(\frac{{}_cE \cdot {}_cI}{5} + {}_sE \cdot {}_sI\right) \tag{2.4}$$

where M_1, M_2 are the larger absolute end moment and smaller end moment, respectively, ${}_cE$ and ${}_sE$ are the Young's modulus of concrete and of steel, respectively, ${}_cI$, ${}_sI$ are moment inertia of concrete and of steel, respectively, l_k is buckling length of the column.

2.3. Strength of Middle-length Column Calculated by Linear Interpolation

In the AIJ design formula, the boundary of short column is L_k/D=4, and the boundary of slender column is L_k/D=12. Then the flexural strength for middle-length column (4<L_k/D<12) can be easily calculated by using linear interpolation method.

Table 1 Experimental Condition and Result

Specimen	Type	L_k/D	e_1 (mm)	e_2 (mm)	β (=e_2/e_1)	${}_c\sigma_B$ (N/mm^2)	N (kN)	$M=N\cdot e_1$ (kNm)
R20-27-C	B	20	0	0	-	29.1	835	0.0
R20-27-10-1.0(+)			10	10	1.0	31.6	585	5.8
R20-27-10-0.5(+)				5	0.5		627	6.3
R20-27-10-0.0				0	0.0	33.9	671	6.7
R20-27-10-0.333(-)				-3.33	-0.333		711	7.1
R20-27-10-0.667(-)				-6.67	-0.667	33.3	738	7.4
R20-27-10-1.0(-)				-10	-1.0		823	8.2
R20-27-30-1.0(+)			30	30	1.0	36.7	440	13.2
R20-27-30-0.5(+)				15	0.5		479	14.4
R20-27-30-0.0				0	0.0		552	16.6
R20-27-30-0.333(-)				-10	-0.333		574	17.2
R20-27-30-0.667(-)	C			-20	-0.667	31.5	620	18.6
R20-27-30-1.0(-)				-30	-1.0		665	19.9
R20-27-100-1.0(+)	B		100	100	1.0	32.1	222	22.2
R20-27-100-0.5(+)				50	0.5		265	26.5
R20-27-100-0.0				0	0.0	35.5	307	30.7
R20-27-100-0.333(-)				-33.3	-0.333		328	32.8
R20-27-100-0.667(-)	C			-66.7	-0.667	35.8	325	32.5
R20-27-100-1.0(-)				-100	-1.0		336	33.6
R20-60-C	B		0	0	-	61.3	1106	0.0
R20-60-30-1.0(+)			30	30	1.0	63.2	550	16.5
R20-60-30-1.0(-)				-30	-1.0		878	26.3
R20-60-100-1.0(+)			100	100	1.0	65.5	250	25.0
R20-60-100-0.0				0	0.0		354	35.4
R20-60-100-0.667(-)				-66.7	-0.667	66.4	372	37.2
R20-60-100-1.0(-)	C			-100	-1.0		385	38.5
R10-27-C	A	10	0	0	-	35.9	1016	0.0
R10-27-30-1.0(+)			30	30	1.0	35.4	606	18.2
R10-27-30-0.5(+)				15	0.5		650	19.5
R10-27-30-0.0				0	0.0	31.6	695	20.9
R10-27-30-0.333(-)				-10	-0.333		722	21.7
R10-27-30-0.667(-)				-20	-0.667	31.1	737	22.1
R10-27-30-1.0(-)				-30	-1.0	37.1	776	23.3
R10-27-100-1.0(+)			100	100	1.0		281	28.1
R10-27-100-0.5(+)				50	0.5		328	32.8
R10-27-100-0.0	B			0	0.0	35.9	351	35.1
R10-27-100-0.333(-)				-33.3	-0.333	34.1	372	37.2
R10-27-100-0.667(-)	C			-66.7	-0.667		372	37.2
R10-27-100-1.0(-)				-100	-1.0	34.7	385	38.5
R10-60-C	A		0	0	-	68.4	1452	0.0
R10-60-100-1.0(+)	B		100	100	1.0		324	32.4
R10-60-100-0.0				0	0.0	67.6	406	40.6
R10-60-100-1.0(-)				-100	-1.0		430	43.0

e_1, e_2: Eccentricity of upper end and bottom end, respectively,
${}_c\sigma_B$: Compressive strength of concrete,
N: Experimental maximum strength

3. ECCENTRICALLY COMPRESSIVE TESTS UNDER DOUBLE CURVATURE DEFORMATION

3.1. Outlines of Specimen

A total of forty-three columns were fabricated and tested. Figure 1 shows details of the test columns. All of the specimens were made of square steel tube with dimensions of 125x125x3.2mm (STKR400) and filled with concrete having targeted compressive strength of 27 N/mm^2 or 60 N/mm^2. The experimental variables are; 1) the L_k/D ratio (20 and 10), the concrete strength, the moment gradient expressed in term of M_2/M_1 ratio, and the initial eccentricity e_1 (100, 30, 10 mm).

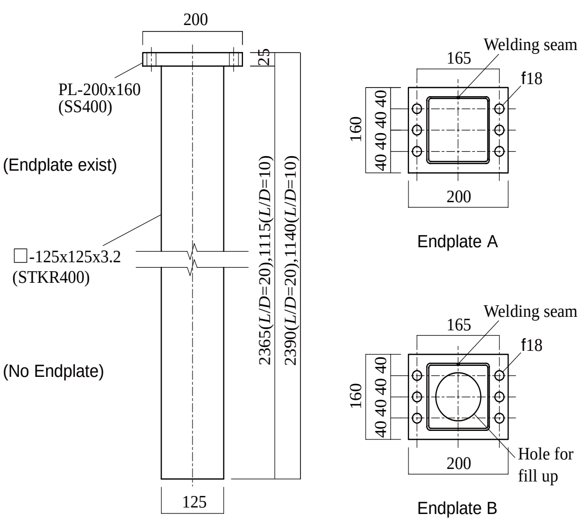


Figure 1 Specimen (mm)

Table 2 Mixture of Concrete

Depth	Thickness	Tension test		Compression test	
		Yield stress	Tensile strength	Yield stress	Compressive strength
(mm)	(mm)	(N/mm^2)	(N/mm^2)	(N/mm^2)	(N/mm^2)
125.1	2.99	358	452	-	318

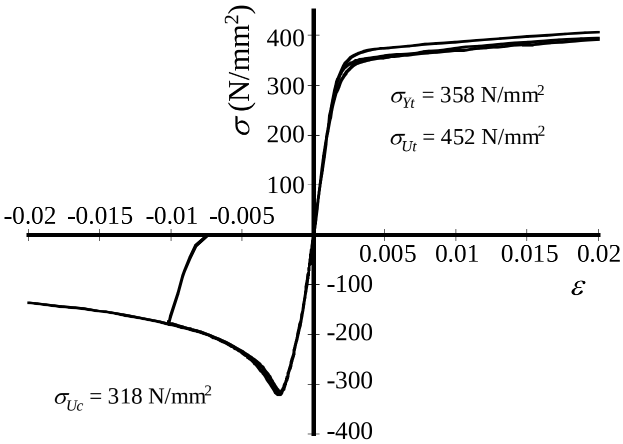


Figure 2 Stress – strain relationship

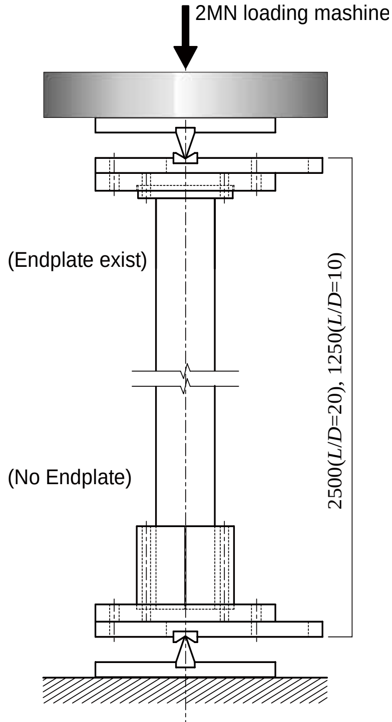


Figure 3 Loading condition

According to the initial eccentricity, the specimens were divided into three types. In Type A specimen, there are no steel plates at both ends of the column. On the other hand, in Type B specimen only one steel plate of 25mm in thickness was welded to the end of the column where the effect due to extra end confinement is significant, while each C type specimens had end plates at both ends. Table 1 shows experimental conditions along with the primary test results of all specimens.

The concrete strengths at the stage of testing are given in Table 1. Figure 2 displays the complete stress-strain curve of the steel tube. The compressive stress-strain curve shown in Figure 2 was obtained by concentric compressive test on the short steel tube, while the tensile stress-strain curve was obtained by conducting tensile test of three standard coupons. The dotted line in Figure 2 expresses result where the axial compressive strain of the steel tube was measured by strain gages.

3.2 Testing Apparatus

Figure 3 shows the test apparatus, while Photos 1 displays detail of end condition. Each specimen was at first loaded concentrically in elastic by a 2MN capacity test machine. After verifying the axial load had been applied in the center of the column via the records of the strain gages, the knife-edge (see Photo 1) at the end of the column was slid to the targeted eccentricity, and then the eccentric loading was applied till large deformation. In addition to two displacement transducers measuring the axial deformation, eight displacement transducers were used to measure the lateral displacements along the length of the column. A total of twenty two strain gages were embedded on the surface of the steel tube to measure the steel strain.

Photo 1 Loading situation

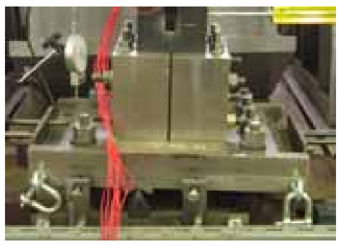
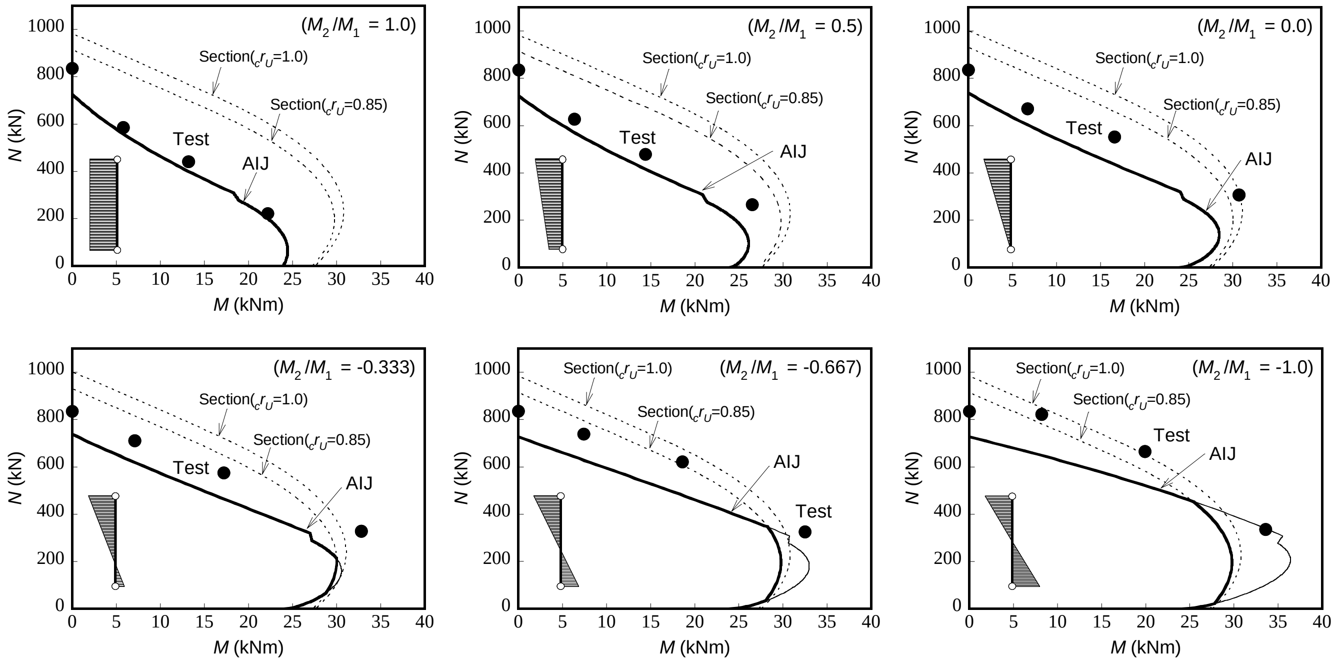
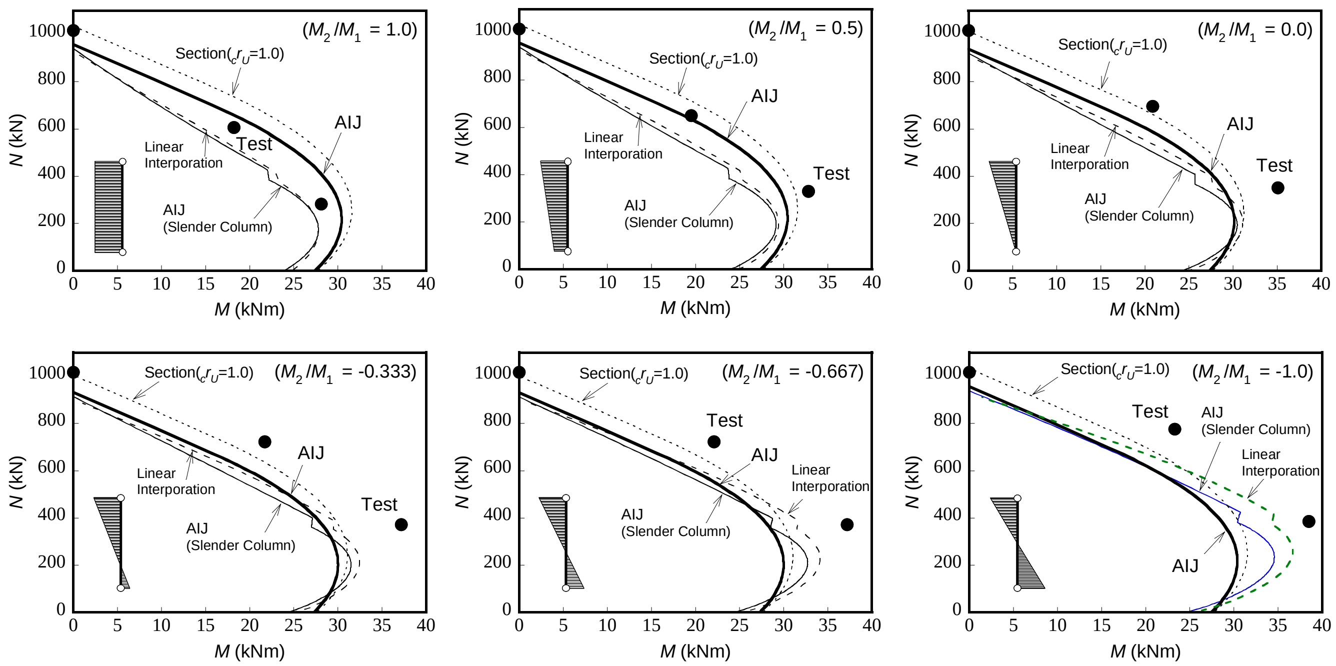


Photo 2 Details of loading equipment



(a) L_k/D=20 (Slender column), $_c\sigma_B$=27N/mm^2



(b) L_k/D=10 (Middle-length column), $_c\sigma_B$=27N/mm^2

Figure 4 Comparison of strength (continue)

4. EXPERIMENTAL RESULTS AND COMPARISON WITH THEORETICAL PREDICTIONS

4.1 Moment Versus Axial Load Relationship

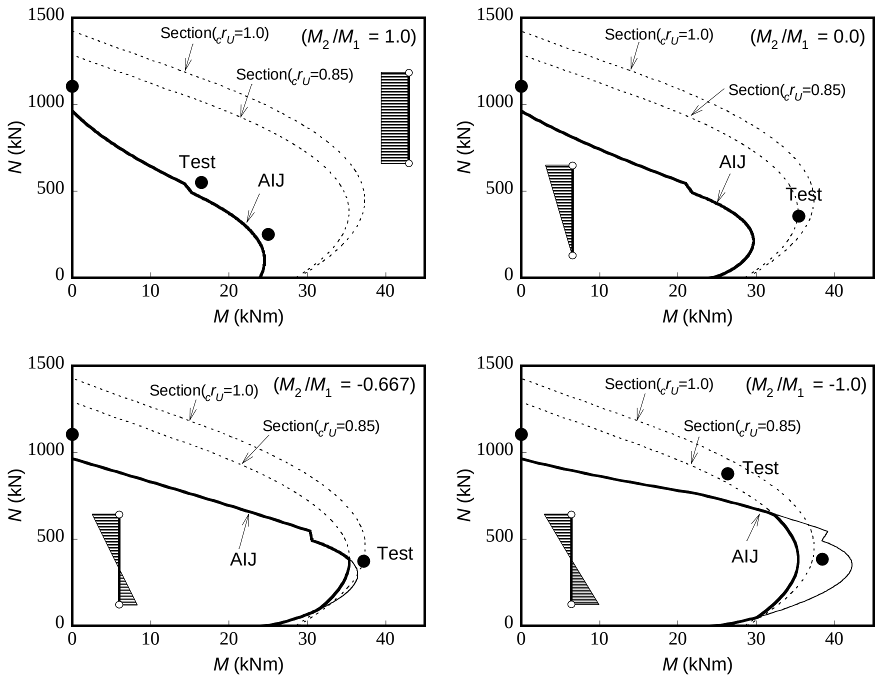
Figure 4 shows experimental flexural strengths along with several theoretical predictions. In Figure 4, the black circles express the experimental results, while the theoretical results are represented by several curves; the black solid line corresponds to the flexural strength of the CFT column calculated by the SRC standard, where effect of the slenderness has been taken into consideration, and the dotted line expresses the ultimate flexural strength of the CFT section. For the middle-length columns, a black line is further plotted as shown in Figure 4 (b) and (d) to represent the flexural strength computed by linear interpolation between the flexural strengths of short and slender columns by SRC standard.

As obvious from Figure 4 (a) and (c), the theoretical results calculated by current SRC standard agree well with the experimental results of the slender columns in spite the concrete strength, while the theoretical predictions tend to underestimate the experimental ones as the moment gradient becomes larger.

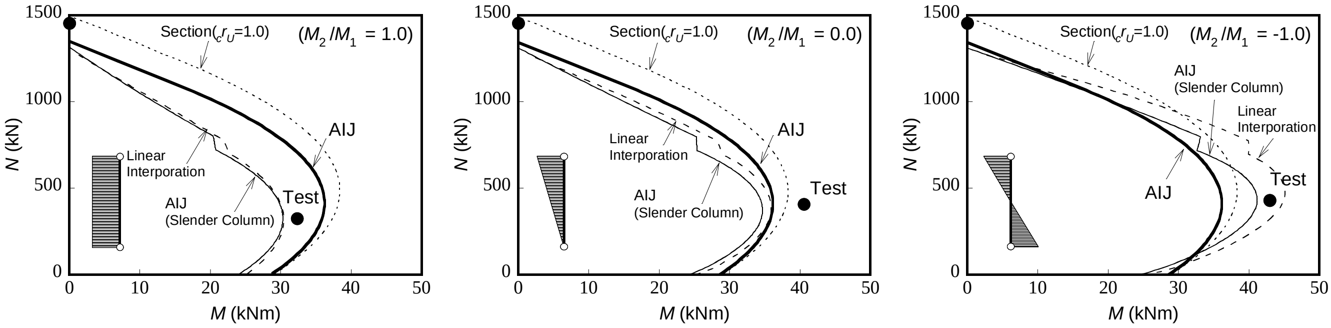
For the middle-length columns, however, the SRC standard seems to overestimate the experimental result for the columns under pure bending,, i.e. without moment gradient. This discrepancy is mainly due to that in the SRC standard, the flexural strength of the middle-length column with L_k/D ratio varying between 4 and 12 is computed by using the equation for the short column. It is apparent, on the other hand, that the theoretical results calculated by linear interpolation could conservatively predict the test results in spite of the moment gradient.

From the above-mentioned comparisons and observation, one can see that flexural strength of the middle-length square CFT column can be reasonably predicted by linear interpolation between the flexural capacities of the short and slender columns calculated by the current SRC standard.

To better understand difference between the experimental and theoretical flexural strengths, relationships between the excess ratios and the excess angles of flexural strength, which are defined in Figure 5, are plotted in Figure 6. For the slender columns with L_k/D ratio of 20, the ratio of the experimentally measured strength to the theoretical results varies between 1.02 and 1,24, having an average of 1.13. For the middle-length columns with L_k/D ratio of 10, the flexural strength ratio varies between 0.90 and 1.29 when the SRC equation for short column is directly applied to calculate the flexural strength. On the other hand, the linear interpolation method can give a more reasonable prediction to the



(c) L_k/D=20 (Slender column), $_c\sigma_B$=60N/mm^2



(d) L_k/D=10 (Middle-length column), $_c\sigma_B$=60N/mm^2

Figure 4 Comparison of strength (the rest)

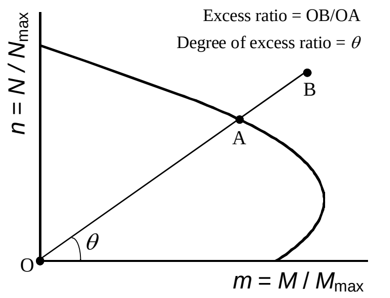
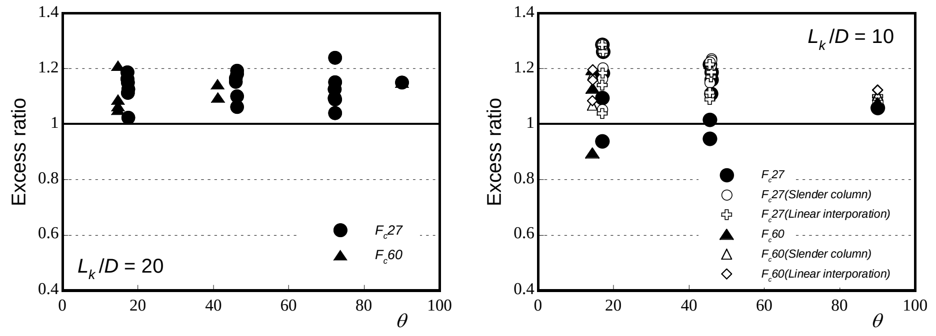


Figure 5 Definition of excess ratio



(a) L_k/D=20 (Slender column)

(b) L_k/D=10 (Middle-length column)

Figure 6 Comparison about excess ratio

flexural strength for the middle-length CFT columns, with the strength ratio varying between 1.05 and 1.29 and an average of 1.18.

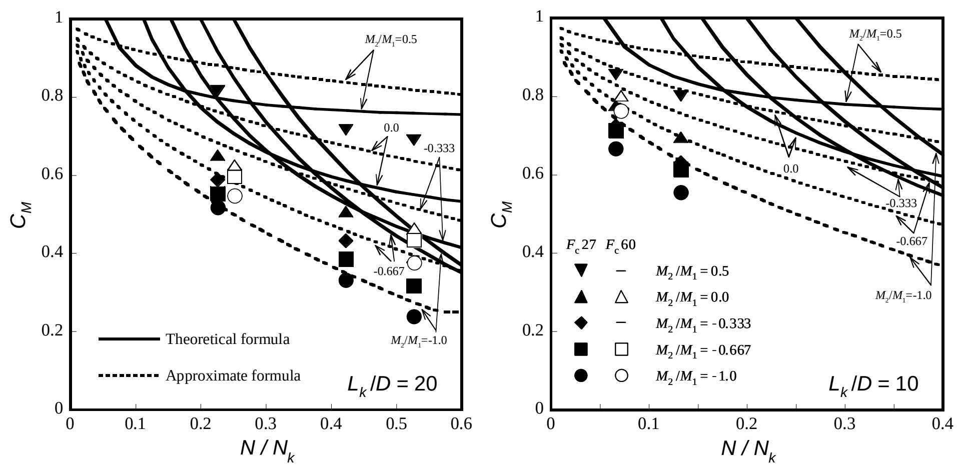
4.2 Moment Amplification Factor C_M

In order to obtain experimental results for the moment amplification factor, the experimental moment versus axial load curves were at first approximated by conducting regression analysis on the experimental flexural strengths for each moment gradient as shown in Figure 7. Then the experimental moment amplification factors were calculated as the difference between the experimentally measured strength and the strength drawn from the approximated moment-axial load curve shown in Figure 7.

Figure 8 shows experimental moment amplification factors along with the theoretical curves recommended by the AIJ plastic design guideline for steel structures [AIJ, 1975]. The bold solid lines and dotted lines in Figure 8 represent equation for the moment amplification factor based on elastic theory and the approximate equation, respectively. Axial force is normalized by Euler's load (see Eqn. (2.4)).

For the slender columns with L_k/D ratio of 20, in the case of F_c=27N/mm^2, the approximate equation agreed well with the test results as the moment ratio M_2/M_1 is equal to -1.0. As becomes larger, i.e. the moment gradient becomes smaller, the discrepancy between the experimental and the theoretical results becomes wider, but the approximate equation could still trace the tendency of the moment amplification factor along with the axial load. In the case of F_c =60N/mm^2, although the experimental data is few, the approximate formula traced the tendency of moment amplification factor C_M as well as in the case of F_c=27N/mm^2.

For the middle-length columns with L_k/D ratio of 10, the experimental moment amplification factors were less than the calculated results obtained either by the elastic theoretical equation or by the approximate equation, which means that the design formula for the slender CFT columns recommended in the current standards would underestimate the axial capacity of the middle-length CFT columns.



(a) L_k/D=20

(b) L_k/D=10

Figure 8 Comparison of moment modified factor C_M

5. CONCRUDING REMARKS

Experiments of slender and middle-length square CFT columns under eccentrically compression with doublecurvature deflection were conducted to examine the effect of moment gradient on the flexural strength of column. Comparing the experimental results with the theoretical predictions leads to the following conclusions:

1) For the slender square CFT columns with the buckling length to depth ratio of 20, the current design formula could satisfactorily well predict the flexural strength in spite of the degree of moment gradient.
2) For the middle-length square CFT columns with the buckling length to depth ratio of 10, the current design formula overestimated the flexural strength when the column was under eccentric compression without moment gradient, i.e. under pure bending. On the other hand, for the middle-length columns under eccentric compression with moment gradient, the current design formula could be applied to evaluate their flexural strength with satisfactory accuracy.
3) Moment amplification factor, which was calculated backward from experimental result, compared well with the elastic theoretical and approximate equations. Approximate equation predicts well the results for the columns with L_k/D= 20 and M_2/M_1=-1.0. However, for the columns with L_k/D= 10, accuracy is much lower regardless of the moment gradient.

REFERENCES

Architectural Institute of Japan (2001). AIJ Standards for Structural Calculation of Steel Reinforced Concrete Structures

Architectural Institute of Japan (1997). Recommendations for Concrete Filled Steel Tubular Structures

Chung, J. and Kimura, J. (2004). Strength Formula of CFT Short Beam-Column. *Summaries of Technical Papers of Annual Meeting* AIJ, 1083-1084

Kido, M. and Tsuda, K. (2005). Design Formula for Concrete Filled Steel Tubular Column Subjected to Axial Load and Bending Moment. *Journal of Structural Engineering* **Vol.51B**, 469-474

Fujinaga, T., Chang, Y.K. and Mitani, I. (2005). A Study on Strength of Slender CFT Beam-Columns. *Journal of Structural Engineering* **Vol.51B**, 475-476

Kilpatrick, A.E. and Rangan, B.V. (1997). Tests on High-Strength Composite Concrete Columns, *Research Report* **No 1/97**, School of Civil Engineering Curtin University of Technology

Architectural Institute of Japan (1975). Recommendations for the Plastic Design of Steel Structures

ACKNOWLEDGMENT

The Authors wish to express their gratitude to the staff of Department of Architecture, Kobe University for their help in experimental work. This investigation is supported by a grant-in-aid of research for steel structures JSSC.