EXPERIMENTAL STUDIES ON THE ULTIMATE MOMENT OF CONCRETE FILLED SQUARE STEEL TUBULAR BEAM-COLUMNS

By MASAHIDE TOMII* and KENJI SAKINO**
Members of A.I.J.

1. INTRODUCTION

Tubular sections, either empty or filled with concrete, are being increasingly used as structural members. When a tube is acting as a compression member, filling the tube with concrete is advantageous because it increases the load-carrying capacity without increasing the size of the column.

Considerable work has been done with regard to the behavior of concentrically loaded concrete filled steel tubular columns. In practice most structural members subjected to axial load are also subjected to combined shearing force and bending moment. Due to the large shear capacity of the concrete filled steel tubular columns, they predominantly fail in flexure. Therefore, it is important to structural design to investigate their inelastic flexural behavior.

Significant researches on circular section beam-columns have been carried out by Furlong¹⁾, Knowles and Park²⁾, Sen³⁾ and Yamada, Sakae and Kondoh⁴⁾. But there were few investigations on square section beam-columns. Furlong¹⁾ tested 17 square sections under various amounts of axial loads, which were held constant while bending moments were increased until the columns ceased to sustain the axial loads. However, the moment-thrust-curvature relationships were not given, but the axial load and the moment at failure were tabulated. There was a lack of information about the inelastic behavior of concrete filled square steel tubular beam-columns.

The objective of this investigation is to test a number of square sections under fixed axial loads and increasing uniform bending moments in order to make clear the moment-thrust-curvature characteristics. The objective also includes the proposition of rational simple method for estimating the ultimate moment.

2. EXPERIMENTAL PROGRAM

Test specimens: 36-specimens with 10 cm × 10 cm cross sections were tested. These were divided into 4 series, I to IV. Each test series consisted of 2 concentrically loaded stub columns (column length was 30 cm) and 7 columns subjected to axial loads and bending moments (see Table 1). The

Fig. 1 Details of the test specimen and locations of strain gauges

Fig. 2 Stress strain relations for concrete

* Professor of Structural Engineering, Kyushu University, D. Eng.

** Research Assistant, Kyushu University, M. Eng.

Table 1 Properties of specimen and test results

Concentrically loaded columns

Series	Specimen	Steel tube used*	F_c' (kg/cm ²)	sN_0 (t)	N_0' (t)	N_0 (t)	sN_0/N_0	Age at test (days)	N_u (t)	N_u/N_0'	N_u/N_0
I	I-A	2-1	326	17.3	46.9	52.8	0.33	48	50.7	1.08	0.96
	I-B							49	50.8	1.08	0.96
II	II-A	2-3	218	29.2	49.3	53.3	0.55	44	52.1	1.06	0.98
	II-B							44	52.0	1.05	0.98
III	III-A	3-2	210	33.4	52.0	55.7	0.64	39	53.9	1.04	0.97
	III-B							39	53.8	1.03	0.97
IV	IV-A	4-1	202	46.0	63.0	66.4	0.69	37	68.0	1.08	1.02
	IV-B							37	67.9	1.08	1.02

Columns subjected to axial load and bending moment

Series	Specimen	Steel tube used*	F_c' (kg/cm ²)	sN_0 (t)	N_0' (t)	N_0 (t)	Axial load			Age at test (days)	M_u (t·cm)
							N (t)	N/N_0'	N/N_0		
I	I-0	2-1	245	17.3	39.6	44.1	0	0	0	46	73
	I-1		390	17.3	52.8	59.9	7.8	0.15	0.13	386	104
	I-2		390	17.3	52.8	59.9	16.1	0.30	0.27	388	114
	I-3		390	17.3	52.8	59.9	19.5	0.37	0.33	393	114
	I-5		390	17.3	52.8	59.9	27.3	0.52	0.46	395	117
	I-6		374	17.3	51.3	58.1	33.7	0.66	0.58	356	91
	I-6'		374	17.3	51.3	58.1	33.7	0.66	0.58	358	84
II	II-0	2-2	220	27.2	47.3	51.3	0	0	0	36	112
	II-1	2-2	220	27.2	47.3	51.3	4.7	0.10	0.09	35	126
	II-2	2-3	220	29.2	49.3	53.3	9.4	0.19	0.18	34	132
	II-3	2-3	220	29.2	49.3	53.3	14.1	0.29	0.26	34	129
	II-4	2-4	220	25.2	45.3	49.3	18.8	0.42	0.38	34	120
	II-5	2-4	220	25.2	45.3	49.3	23.6	0.52	0.48	35	109
	II-6	2-4	220	25.2	45.3	49.3	28.3	0.62	0.57	35	93
III	III-0	3-1	210	33.4	52.0	55.7	0	0	0	38	143
	III-1	3-1	210	33.4	52.0	55.7	5.2	0.10	0.09	37	149
	III-2	3-1	210	33.4	52.0	55.7	10.4	0.20	0.19	37	155
	III-3	3-2	210	33.4	52.0	55.7	15.6	0.30	0.28	37	148
	III-4	3-2	210	33.4	52.0	55.7	20.8	0.40	0.37	37	136
	III-5	3-2	210	33.4	52.0	55.7	26.0	0.50	0.47	38	127
	III-6	3-3	210	33.4	52.0	55.7	31.2	0.60	0.56	38	110
IV	IV-0	4-1	190	46.0	62.0	65.2	0	0	0	30	187
	IV-1	4-1	190	46.0	62.0	65.2	6.3	0.10	0.10	30	194
	IV-2	4-1	190	46.0	62.0	65.2	12.6	0.20	0.19	31	192
	IV-3	4-2	190	46.1	62.1	65.3	18.9	0.30	0.29	31	186
	IV-4	4-2	202	46.1	63.1	66.5	25.6	0.41	0.38	34	177
	IV-5	4-2	202	46.1	63.1	66.5	32.0	0.51	0.48	34	161
	IV-6	4-3	202	46.7	63.7	67.1	38.3	0.60	0.57	35	143

* See Table 3.

Note: $sN_0 = sA_s\sigma_y$, $N_0' = sA_s\sigma_y + cAF_c'$, $N_0 = sA_s\sigma_y + cA_c\sigma_B$, $c\sigma_B = 1.2F_c'$ F_c' =compressive strength of expansive concrete cylinder cured in the mould. $c\sigma_B$ =compressive strength of concrete core. M_u =experimental ultimate moment. N_u =experimental ultimate strength of the concentrically loaded column.

variable parameters between the series were the tube thickness and material properties, and the variable between the specimens in each series was the axial load. The external dimensions of the specimen are shown in Fig. 1.

The concrete was compacted by means of a vibrator during and after the placing. Specimens

of the same series were filled from one batch except for those of series I. A number of 10 cm×20 cm test cylinders were taken from each batch for determination of the concrete properties. The concrete cylinders and both ends of the specimens were capped with a thin layer of neat cement and were flush with the steel surface. The specimens and concrete cylinders in the moulds were then stored at ambient temperature and humidity until testing.

Table 2 Mix proportion of concrete

Test series	W/C (%)	Water (kg/m ³)	Expansive cement (kg/m ³)	Sand (kg/m ³)	Coarse aggregate (kg/m ³)	Slump (cm)
I	57	189	333	933	921	4.0
II	69	237	343	925	942	5.5
III	71	237	333	925	942	6.3
IV	70	226	321	925	942	6.7

Materials : Crushed coarse aggregate of 20 mm maximum size and sand of 2.5 mm maximum size were used in making the concrete. The concrete mix ratios by weight are shown in Table 2. Expansive cement containing 15 percent of expansive component was used in order to prevent the separation between concrete core and steel tube due to shrinkage of encased concrete. Compressive strength determined from the expansive concrete cylinders cured in moulds, F_c' , is shown in Table 1, and it is an average of three measurements. Stress-strain relations for the concrete are plotted in Fig. 2.

Tubes were cold worked welded mild steel, and all tubes were annealed to remove the residual stresses in the cross section. The material is the structural steel called STKR 41 in Japanese Industrial Standard. The actual dimensions are listed in Table 3. Material properties of steel tubes were determined from tensile coupon tests. Three coupons were taken from each steel tube, and those were cut out from plates which did not include welded seam. Typical stress-strain curve is shown in Fig. 3. The relevant values pertaining to the elastic, yield and strain-hardening ranges of the steel, as defined in Fig. 3, are listed in Table 3.

Experimental apparatus : A test set-up of columns subjected to axial load and bending moment is shown in Fig. 4. The axial load was transferred to the specimen through two parallel knife-edge supports (A) to allow for complete freedom of rotation of the specimen in the plane of the applied bending moment. Spherical ball bearing supports were used for the specimens of Series I instead of knife-edge supports. The end details were such that the steel and concrete were loaded together at the ends. Two hydraulic jacks (B), connected to the same pressure source, were used to exert upward loads through 10 t load cells. The tie rod arrangements (C) were designed to allow for unrestrained

Table 3 Actual dimensions and mechanical properties of steel tubes used in tests

	D (cm)	t (mm)	D/t	sA (cm ²)	$s\sigma_y$ (t/cm ²)	σ_u (t/cm ²)	$s\sigma_y/\sigma_u$	ϵ_f (%)	ϵ_{st}/ϵ_y	E_{st}/E	E (10 ⁴ t/cm ²)
2-1	10.0	2.29	44	8.78	1.98*	3.53	0.56	30	21	0.014	2.10
2-2	10.0	2.27	44	8.75	3.11	3.98	0.78	27	17	0.006	2.13
2-3	10.0	2.20	45	8.43	3.46	4.19	0.83	25	18	0.007	2.18
2-4	10.0	2.22	45	8.55	2.95	3.86	0.77	29	18	0.007	2.20
3-1	10.0	2.98	34	11.31	2.95	4.09	0.72	25	12	0.010	2.15
3-2	10.0	2.99	33	11.35	2.94	4.12	0.71	29	13	0.011	2.10
3-3	10.0	2.99	33	11.35	2.94	4.12	0.71	29	13	0.011	2.10
4-1	10.0	4.25	24	15.85	2.90	4.00	0.72	29	14	0.011	2.30
4-2	10.0	4.25	24	15.85	2.91	4.01	0.73	29	13	0.010	2.30
4-3	10.0	4.26	23	15.88	2.94	4.05	0.73	28	13	0.010	2.20

* Tube 2-1 was annealed in atmosphere at 740°C, and all other tubes were annealed in atmosphere at 650°C.

Note: D =breadth and depth of steel tube. $s\sigma_y$ =yield point stress. σ_u =tensile strength. sA =cross sectional area. ϵ_y =strain at first yield. ϵ_f =maximum elongation. t =thickness of wall. ϵ_{st} =strain at start of strain hardening. E =modulus of elasticity. E_{st} =strain hardening modulus.

Fig. 3 Stress strain curve for steel

Fig. 4 Test set-up

rotation of the specimen at load points (D). The weight of the bending moment arrangement was taken off by a counter-weight (E).

Rotation of test region was determined by the use of rotation meters consisting of two frames clamped to transverse sections 20 cm apart (see Figs. 1 and 5). Dial gauges reading to 0.01 mm were fitted to the top and bottom of the frames in order to measure displacements due to relative rotation of the sections. Three electrical deflection gauges 10 cm apart were used to measure the deflections along the test region at three points shown in Fig. 5. The layout of the strain gauges is shown in Fig. 1.

Columns subjected only to axial load were concentrically loaded through spherical ball bearing seats.

Experimental procedure : After the constant axial load was applied, a monotonic increasing moment was applied to the specimen by increasing the pressure in jacks (B). As a result of the deflections of the bent specimens during tests, the actual moment at midheight of specimens was the sum of the moment applied by the jacks (B) and the additional moment ($N\Delta$ moment) due to the axial load and deflection.

The rotation, deflections and strains were recorded between load applications. The deflections were held constant during this time. This resulted in a decrease of bending moment due to creep but, since the readings were performed relatively quickly, the creep effects on the measurements were probably unimportant. Local bucklings of the steel tube were marked as they were observed by the naked eye. A complete test took on an average about 45 to 60 min. utes

3. EXPERIMENTAL RESULTS

Fig. 6 Axial load-average longitudinal strain relationships for all specimens

Concentrically loaded columns: Relationships between axial load and average longitudinal strain for concentrically loaded columns are shown in Fig. 6, and the ultimate loads are shown in Table 1. The ultimate loads are slightly larger than calculated values $N_0' = s A_s \sigma_y + c A F_c'$ (see Table 1). It is considered that one reason for this is that compressive strength of concrete cores cured in steel tubes may have differed from that of concrete cylinders cured in moulds.

Tomii, Yoshimura and Morishita investigated experimentally the compressive strength of concrete cores cured in circular steel tubes and concrete cylinders cured in moulds both for concrete without ex-

Fig. 7 Experimental moment-thrust-curvature relationships for all specimens

pansive component (referred to as normal concrete) and for concrete made with cement containing 15 percent of expansive component. They reported that for normal concrete the compressive strength of concrete core, $c\sigma_B$, was approximately the same as that of concrete cylinder, F_c , while for expansive concrete $c\sigma_B$ was about 1.2 times the compressive strength of concrete cylinder, F_c' (see Table 4).

If $c\sigma_B$ is assumed to be 1.2 times F_c' , ultimate loads of concentrically loaded columns are approximately the same as nominal squash loads $N_0 = sA_s\sigma_y + cA_c\sigma_B$ (see Table 1).

Columns subjected to axial load and bending moment: The moment-thrust-curvature relationships (referred to as $M-N-\phi$ relationships) are shown in Fig. 7, and ultimate moments are given in Table 1. In Fig. 7, bending moment of midheight section includes $N-A$ moment, and ϕ_a is an average curvature which is given by the rotation readings. Strain readings also give the curvature of three sections, ϕ_{w1} , ϕ_{w2} , ϕ_{w3} (see Fig. 1). For small values, ϕ_{w1} , ϕ_{w2} , ϕ_{w3} and ϕ_a show close agreement, but in some tests for higher values they differ appreciably. The ϕ_{w1} , ϕ_{w2} , ϕ_{w3} values are not available at high curvature because the local buckling of steel tube wall occurs. Therefore, ϕ_a is used in all the $M-N-\phi$ plots. In Fig. 7, the axial load is given in the nondimensional form N/N_0 where N_0 is the nominal squash load. As can be seen from Fig. 7, the value of N/N_0 and the external wall breadth to wall thickness ratio, D/t , have a significant effect on the $M-N-\phi$ relationships. The $M-N-\phi$ relationships of columns I-5, I-6, I-6', II-5, II-6, whose N/N_0 and D/t are large, are rather brittle. However, it should be noted that even in those columns tension side of the steel tube yielded in tension.

4. ULTIMATE MOMENT

The ultimate moment for given axial load can be estimated by the computation procedure for the $M-N-\phi$ relationships. However, this procedure is complicated to calculate. Furlong concluded that accurate estimates of beam-column strength were made by treating the concrete filled tubes as ordinary reinforced concrete sections, and recommended to use the capacity rules of A.C.I. Building Code, but in his paper the intensity of concrete stress was taken as F_c instead of $0.85F_c$ recommended by A.C.I. (referred to as modified A.C.I. method). However, the calculation of this procedure is also complicated, because the stress distribution of steel tube web is much more complicated than ordinary reinforced concrete. In this section a simpler estimation method which is able to predict the ultimate moment is proposed.

Proposed method: The stress blocks shown in Fig. 8 are assumed for steel tube and concrete core in the proposed method. The value of concrete stress $c\sigma_B$ is taken as F_c as recommended by Furlong. However, $c\sigma_B$ is taken as $1.2F_c'$ in the calculations for specimens for which expansive concrete is used for the same reason described in the preceding section.

From the equilibrium of axial direction, depth of the neutral axis, x_n , at ultimate moment for given axial load is given by Eq. 1:

$$x_n = \left\{ \frac{2t_1 s\sigma_y + (1-2t_1)t_1 c\sigma_B + \frac{N}{D^2}}{(1-2t_1)c\sigma_B + 4t_1 s\sigma_y} \right\} D \quad \text{for } (t \leq x_n \leq D-t), \dots \dots \dots (1)$$

where

$$t_1 = t/D,$$

N = axial load.

Ultimate moment, M_u , for given axial load is given by Eq. 2:

Table 4* Comparison between the compressive strength of concrete cores cured in circular steel tubes, $c\sigma_B$, and the compressive strength of concrete cylinders cured in moulds, F_c or F_c'

Concrete	$c\sigma_B$ (kg/cm ²)			F_c or F_c' (kg/cm ²)
	$t=3$ mm	$t=4$ mm	$t=5.5$ mm	
Normal concrete	365	380	375	359
	262	256	264	260
	170	174	173	172
Expansive concrete	367	368	361	286
	257	255	256	201
	160	159	159	140

* These results are reported in Ref. 5.

Note: Each value of compressive strength is an average of three measurements. t = thickness of circular steel tube wall. Outside dimension of steel tube is 10 cm.

Fig. 8 Assumed stress blocks of steel tube and concrete

Fig. 9 Axial load-ultimate moment interaction curves for series III

$$M_u = \{t_1(1-t_1)s\sigma_y + 2(x_{n1}-t_1)(1-x_{n1}-t_1)t_1s\sigma_y + 0.5(1-2t_1)(1-x_{n1}-t_1)(x_{n1}-t_1)c\sigma_B\}D^3, \quad \dots\dots\dots(2)$$

where

$$x_{n1} = x_n/D.$$

The interaction curve between axial load and ultimate moment (referred to as N - M interaction curve) derived from the proposed method is shown in a solid line in Fig. 9. Also in Fig. 9, data points for the test specimens of Series III are shown with circled dots, and the result by modified A.C.I. method is shown in a dashed line. By the proposed method, ultimate moment for the case of zero axial load, M_0 , maximum ultimate moment in the N - M interaction curves, $M_{\max}$, and axial load corresponding to $M_{\max}$, N_m , are given by Eqs. 3, 4 and 5:

$$M_0 = \left\{1 + \frac{(1-\alpha)(1-4t_1)}{3(1-4t_1+2t_1\alpha)}\right\}M_p = \left\{1 + \frac{1-\alpha}{3}\right\}M_p, \quad \dots\dots\dots(3)$$

$$M_{\max} = M_p + {}_cM_{\max} = \frac{D^3}{4}s\sigma_y\{1 - (1-2t_1)^3\} + \frac{D^3}{8}c\sigma_B(1-2t_1)^3, \quad \dots\dots\dots(4)$$

$$N_m = \frac{{}_cA c\sigma_B}{2}, \quad \dots\dots\dots(5)$$

where

$$\alpha = \frac{sN_0}{N_0} = \frac{sA s\sigma_y}{sA s\sigma_y + {}_cA c\sigma_B},$$

M_p = full plastic moment capacity of steel tube,

${}_cM_{\max}$ = maximum moment capacity of concrete core.

The ratio M_u/M_{cal} against N/N_0 , where M_u is experimental ultimate moment of our test or Furlong's test¹⁾ and M_{cal} is theoretical ultimate moment given by the proposed method or modified A.C.I. method, is mentioned in Figs. 10 a, 10 b, 11 a and 11 b. Here, experimental ultimate moments of the specimens of Series IV were defined as the moments of flat plateaus of their M - N - ϕ curves (see Fig. 7). The values of ratio M_u/M_{cal} for Furlong's experimental results are scattered as shown in Fig. 10 b. It is considered that this scattering is due to the properties of steel tubes which are not annealed.

The proposed method introduces a great simplification of calculation, but it cannot predict the balance point where the tension side of steel tube just reaches its yield point when the ultimate moment is attained. For the column subjected to high axial load greater than the axial load corresponding to balance point, N_b , the tension side of steel tube never reaches its yield point. In that case, the assumed stress block for steel tube is not valid.

As shown in Fig. 10 a, the calculated ultimate moments given by the proposed method are overestimated for the two columns of Series I, I-6 and I-6', which were tested under high axial load

(a) The comparison using our experimental results

(b) The comparison using Furlong's experimental results

Fig. 10 Comparison between experimental ultimate moments, M_u , and theoretical ones, M_{cal} , given by our proposed method.

(a) The comparison using our experimental results

(b) The comparison using Furlong's experimental results

Fig. 11 Comparison between experimental ultimate moments, M_u , and theoretical ones, M_{cal} , given by modified A.C.I. method proposed by Prof. R.W. Furlong

Fig. 12 Nondimensional interaction curves

Fig. 13 sN_0/N_0 - D/t (or t/D) relationships

($N/N_0=0.58$).

Furthermore, validity of the proposed method for the columns subjected to N/N_0 greater than 0.58 cannot be confirmed by our experiment in which N/N_0 of the columns was taken as 0~0.58.

From the above consideration, the proposed method is applicable to the columns subjected to the N which is less than $0.5 N_0$.

Design charts : The nondimensional interaction curves derived from the proposed method are shown in Fig. 12 for various values of the parameter sN_0/N_0 . In Fig. 12, sN_0 is the squash load of steel tube, M_p is the full plastic moment capacity of steel tube, and e is the eccentricity of the axial load

N from the centroid of the column. Columns with different values of the steel tube wall thickness have slightly different nondimensional interaction curves, though they have the same value of parameter sN_0/N_0 . This is due to the difference of nondimensional section size, $(D-2t)/D$, of concrete core.

Fig. 13 shows the relations between sN_0/N_0 and D/t (or t/D). In our experiment, the value of sN_0/N_0 was taken as 0.31 $\sim$ 0.70.

Fig. 14 Interaction charts for given columns

The most convenient form for a design chart for columns is obtained by plotting N/D^2 against M/D^3 for various of the parameter t/D , and by assuming material properties to be constant for a particular chart. Fig. 14 is a sample of design chart for ultimate strength design.

5. CONCLUSIONS

The following conclusions are reached on bases of the study which has been presented.

- (1) The magnitude of constant axial load and wall breadth to wall thickness ratio, D/t , of steel tube had a significant effect on an inelastic behavior of the concrete filled square steel tubular columns subjected to axial loads and bending moments, especially on the falling branch of the moment-thrust-curvature relationships. The moment-thrust-curvature relationships of the columns having a D/t ratio of 24 did not have any falling branch. However, the columns having a D/t ratio of 44 were rather brittle when they were subjected to high axial load and moment (see Fig. 7).
- (2) The ultimate moment which is an important quantity for structural design can be estimated accurately by the proposed simple method for the columns in which annealed steel tubes are used (see Fig. 10 a). This proposed method is applicable for the columns subjected to the axial load N which is less than $0.5 N_0$ ($N_0 = sA_s \sigma_y + cA_c \sigma_B$). It is considered that this limitation of N/N_0 does not cause inconvenience to actual design.

The numerical simulation of the moment-thrust-curvature characteristics will be reported in Transactions of the Architectural Institute of Japan in the near future.

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軸力と曲げを受けるコンクリート充填角形鋼管柱の 曲げ終局強度に関する実験的研究（梗概）

正会員 富 井 政 英*
正会員 崎 野 健 治**

1. 序

鋼管は中空あるいはコンクリート充てん鋼管部材として構造部材に使用されることが多くなってきた。

コンクリート充てん鋼管柱は、せん断破壊よりも曲げ破壊が先行する場合が多く、曲げに対する性状を明らかにすることは、構造設計上重要な研究課題となっている。

軸力と曲げを受けるコンクリート充てん鋼管柱に関しては、円形断面柱については、Furlong¹⁾, Knowles等²⁾, Sen³⁾, 山田等⁴⁾の研究があるが、正方形断面柱に関しては研究が少ない。一定軸力と曲げを受けるコンクリート充てん正方形断面鋼管柱（以下コンクリート充てん角形鋼管柱と書く）に関する実験的研究として Furlong の研究¹⁾があるが、これは主として曲げ終局強度に注目したものであり、変形性状（曲げモーメント・曲率関係）については述べられていない。

本論文の目的は、軸力と曲げを受けるコンクリート充てん角形鋼管柱の曲げモーメント・曲率関係を実験的に明らかにするとともに、曲げ終局強度を簡単にかつ精度よく求める方法を提案することである。

2. 実験計画

試験体：10 cm×10 cm の断面を持つ計 36 体の試験体について、シリーズ I から IV までの 4 シリーズの実験を行なった。それぞれのシリーズは、中心圧縮柱が 2 体と、軸力と曲げを受ける柱が 7 体の計 9 体からなる（表 1 参照）。各シリーズ間の実験変数は鋼管板厚と材料強度である。また、同一シリーズ内での実験変数は、一定軸力の大きさである。試験体の形状を図 1 に示す。

シリーズ I を除き、同一シリーズの試験体には同一バッチのコンクリートを使用した。試験体製作と同時に 10 cm×20 cm のシリンダーを製作した。試験体の両端部はキャッピングをし、鋼管とコンクリートが同一面となるようにした。試験体とシリンダーは室内養生をした。シリンダーは型枠に入れたまま養生を行なった。

材料：コンクリートの重量調合比を表 2 に示す。な

お、コンクリートの収縮による鋼管と充てんコンクリートのはだ離れを防ぐために、膨張材をセメント内割で 15% 混入した膨張セメントを使用した。型枠養生した膨張コンクリートシリンダーの圧縮強度 F_c' を表 1 に、応力度・ひずみ度関係を図 2 に示す。

鋼管は冷間加工の電縫鋼管を焼鈍して使用した。焼鈍前の材質は STKR 41 である。引張試験より求めた焼鈍後の鋼管の力学的性質と実測寸法を表 3 に、応力度・ひずみ度関係の 1 例を図 3 に示す。

実験装置：軸力と曲げを受ける柱の加力装置を図 4 に示す。軸力加力部の拘束をなくすために、2 組のナイフエッジ (A) を用いた。ただし、シリーズ I の実験には球座を使用した。試験体両端面においては、鋼管とコンクリートを同時に圧縮した。2 台の油圧ジャッキ (B) は連動して用い、ロードセルにより力を計測した。加力点 (D) においては、拘束をなくすため、球をそう入している。曲げモーメント・載荷装置は、自重を取り除くため、カウンターウェイトにより吊り上げた。

20 cm 間隔に取り付けた測定フレーム上下にダイヤルゲージを取り付け、試験体部分の回転角の測定を行なった（図 1, 5 参照）。図 5 に示す 3 点で、試験部分のたわみを変位計により電氣的に測定した。

中心圧縮柱は 2 組の球座を用いて加力を行なった。

実験方法：一定軸力を加力した後曲げモーメントを加力した。鋼管の局部座屈の観測は肉眼によった。一体の実験に要した時間は 45~60 分であった。

3. 実験結果

中心圧縮柱：軸力 N と平均軸方向ひずみ度 ϵ_t との関係を図 6 に示し、中心圧縮耐力の値を表 1 に示す。中心圧縮耐力は、計算値 $N_0' = s_s A_s \sigma_y + c A F_c'$ よりもやや大きい（表 1 参照）。この原因として、シリンダーと充てんコンクリートの圧縮強度の違いが考えられる。

富井、吉村、森下らは、円形鋼管中で養生したコンクリートと型枠中で養生したシリンダーの圧縮強度の比較実験を行ない、普通コンクリートの場合は両者の圧縮強度はほぼ同じであるが、膨張材をセメント内割で 15% 混入した膨張コンクリートの場合は鋼管中で養生したコンクリートの圧縮強度 $c\sigma_B$ は型枠養生したシリンダー強

* 九州大学 教授・工博

** 九州大学 助手・工修

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度 F_c' の約 1.2 倍であることを報告⁵⁾している (表 4 参照)。

充てんコンクリートの圧縮強度 $c\sigma_B$ を $1.2 F_c'$ と考えると、中心圧縮耐力は単純累加中心圧縮耐力

$$N_0 = sA_s \sigma_y + cA_c \sigma_B$$

とほぼ等しい (表 1 参照)。

軸力と曲げを受ける柱：曲げモーメントと軸力と曲率の関係 (以下 $M-N-\phi$ 関係と書く) を図 7 に示し、曲げ終局強度の値を表 1 に示す。図 7 において、縦軸は軸力による付加曲げモーメントを含んだ試験部分中央断面の曲げモーメントであり、 ϕ_a は回転角の測定より得られた試験部分の平均曲率である。図 7 から、軸力比 N/N_0 と鋼管の幅と板厚の比 D/t は $M-N-\phi$ 関係に大きな影響を及ぼすことがわかる。 N/N_0 と D/t が大きな試験体 (I-5, I-6, I-6', II-5, II-6) はむしろ脆性的な挙動を示した。しかしながら、これらの試験体においても、鋼管の引張縁は引張降伏を起こしていた。

4. 曲げ終局強度

Furlong はコンクリート充てん鋼管の曲げ終局強度計算法として、A.C.I. 規準の耐力計算法を準用することを提案している⁷⁾。すなわち、コンクリートの応力ブロックの高さを A.C.I. 規準の $0.85 F_c$ から $1.0 F_c$ に高めて使用することを提案している (この計算法を以下修正 A.C.I. 法と書く)。しかしながら、この計算法は鋼管ウェブの応力分布が複雑になり、計算が煩雑である。

本節においては、より簡単に曲げ終局強度を計算出来る方法を以下に提案する。

計算法：鋼管および充てんコンクリートに対して、図 8 に示す応力ブロックを仮定する。コンクリートの応力ブロックの高さ $c\sigma_B$ は、Furlong の提案したように充てんコンクリートの圧縮強度 F_c をとる。ただし、膨張コンクリートを充てんした本実験の試験体に対しては、前に述べた理由から $c\sigma_B = 1.2 F_c'$ として計算した。

軸方向の力の釣り合い条件より、圧縮縁から中立軸までの距離 x_u は 1 式で与えられる。また、与えられた軸力に対する曲げ終局強度 M_u は 2 式で与えられる。

本計算法により求まる軸力と曲げ終局強度の相関曲線 (以下 $N-M$ 相関曲線と書く) を図 9 に実線で示す。また、図 9 にはシリーズⅢの実験結果を●で、修正 A.C.I. 法による計算結果を点線で示している。

本計算法によると、軸力零の場合の曲げ終局強度 M_0 と $N-M$ 相関曲線上の最大の曲げ終局強度 $M_{\max}$ および $M_{\max}$ に対応する軸力 N_m は 3, 4, 5 式で与えられる。

曲げ終局強度に関する実験値と計算値の比較を図 10 a, 10 b, 11 a, 11 b に示す。実験値 M_u は本実験および Furlong の実験⁷⁾により得られた実験結果であり、計算

値 M_{cal} は本計算法と修正 A.C.I. 法により得られたものである。ただし、シリーズⅣの試験体の曲げ終局強度は、 $M-N-\phi$ 曲線が平らになるところの曲げモーメントと定義した (図 7 参照)。

図 10 b に示されるように、Furlong の実験結果に対しては実験値と計算値の比はかなりバラついていて、この原因として、鋼管を焼鈍せずに使用していることが考えられる。

本計算法によると計算が簡単になるが、曲げ終局強度に達すると同時に鋼管引張縁が降伏するときの釣り合い軸力を求めることが出来ない。釣り合い軸力より大きな軸力を受ける柱の鋼管引張縁は曲げ終局強度に降伏点に達することがなく、そのような柱に対しては鋼管の応力ブロックに対する仮定は妥当ではなくなる。図 10 a からわかるように、シリーズⅠの試験体で軸力比の大きな ($N/N_0 = 0.58$) 試験体 I-6, I-6' に対しては計算値は実験値よりも大きい。また、本実験の軸力比 N/N_0 の範囲は 0~0.58 であり、軸力比が 0.58 以上の柱については本計算法の妥当性が実証されていない。

以上のことを考慮して、本計算法の適用範囲は軸力比が 0.5 以下の場合とする。

設計図表：本計算法により求めた無次元化相関曲線を図 12 に示す。 sN_0 は鋼管の中心圧縮耐力、 M_p は鋼管の全塑性モーメント、 e は偏心距離である。図 13 は sN_0/N_0 と D/t (あるいは t/D) の関係を示したものである。また、本実験における sN_0/N_0 の範囲は 0.31~0.70 である。

柱の設計図表としては、縦軸に N/D^2 、横軸に M/D^3 をとり、材料強度を一定、 t/D をパラメータとして書いた図表が便利である。図 14 に終局強度にもとづく設計図表の一例を示す。

5. 結 論

本論により下記の結論が得られた。

1. 軸力の大きさと鋼管の幅 D と板厚 t との比 D/t は、軸力と曲げを受けるコンクリート充てん角形鋼管柱の塑性域における挙動、特に曲げモーメント・曲率関係の耐力以後の性状に大きな影響を及ぼす。すなわち、 D/t が 24 の柱は塑性域においても曲げモーメントの低下が見られないが、 D/t が 44 の柱は高軸力下において脆性的な挙動を示す (図 7 参照)。

2. 設計上重要な曲げ終局強度は、焼鈍した鋼管を用いた柱に対しては、本論で述べる簡単な計算法により精度よく算定出来る (図 10 a 参照)。ただし、本計算法の適用範囲は軸力が $0.5 N_0$ ($N_0 = sA_s \sigma_y + cA_c \sigma_B$) 以下の場合とする。この適用範囲は、設計する上では支障とされないと考えられる。