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Autor: Sakino, Kenji / Sun, Yuping

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Ultimate Strain and Strength of RC Columns Retrofitted by Steel Tubes

Kenji SAKINO
Professor
Kyushu University
Fukuoka, Japan

Kenji Sakino, born 1946, received his PhD from Kyushu University in 1982. His research interests include earthquake resistance of reinforced concrete columns and shear walls, confinement of high-strength concrete, and inelastic behaviour of concrete-filled steel tubular columns.

Yuping SUN
Research Associate
Kyushu University
Fukuoka, Japan

Yuping Sun, born 1958, received his PhD from Kyushu University in 1992. His research interests include seismic behaviour of reinforced concrete columns, confinement of high-strength concrete, and non-linear analysis of concrete members.

Amin AKLAN
Graduate Student
Kyushu University
Fukuoka, Japan

Amin Aklan, born 1958, completed his ME degree in structural engineering at Kyushu Univ. in 1995. He is currently a doctoral student in graduate school of engineering at Kyushu Univ.

Summary

Ten reinforced concrete columns retrofitted by square steel tubes were tested under combined bending moment and constant axial load to investigate the main factors influencing ultimate strain and ultimate moment of the retrofitted R/C columns. Test results indicated that the ultimate strains of extreme concrete fiber at maximum moment were not constants, but varied mainly according to wall thickness of the steel tubes. Test results also showed that ultimate strength of the retrofitted columns increased as wall thickness of the steel tubes and the axial load increased.

1. Introduction

It is now well known that earthquake-resistant capacity of a reinforced concrete column can be remarkably enhanced when the column is encased or retrofitted by a steel tube. As compared with conventional transverse hoops, the steel tube has two more advantages of providing a large amount of transverse steel easily and preventing spalling of the shell concrete. Because of these merits, confining method utilizing the steel tube has been widely used to retrofit or strengthen existing concrete columns, particularly columns designed under previous Japanese design codes which nowadays are known to be unsound.

To establish a rational design method for the columns retrofitted by steel tube (referred to as tubed column hereafter), knowledge of the ultimate strain and strength of tubed column sections is of fundamental importance. In authors' previous study [1] of inelastic behavior of the tubed columns, ultimate strain and strength were only indirectly investigated through load-displacement responses of columns under axial load and bending moment with shear. For the columns under axial load and bending moment with shear, since the moment gradient and extra confinement from the end stiff loading stubs exist, it is hard to assess the actual ultimate strain and strength of tubed sections. Therefore, in order to better understand the real ultimate strain and strength of the tubed columns, experimental work on the tubed columns subjected to axial load and bending moment without shear is necessary.

The purpose of this paper is to experimentally study the effects of the axial load level and wall thickness of the steel tube on the moment-curvature relationship of square tubed columns without shear. A total of ten square reinforced concrete columns encased by square steel tubes available on the market were fabricated and tested under constant axial load and monotonic bending moment. Furthermore, based on a stress-strain curve model for the confined concrete proposed by authors,

Table 1 Details of the test columns and primary results

Specimen	fc (MPa)	B/t	Axial load N(kN)	n	Test results M_m	ε_m	M_{cm}	Theoretical results ε_{cm}	M_p	ratio	M_{ACI}	ratio
T6S03	50.7	44	98	0.03	76	0.0046	NA	0.0065	65	1.17	64	1.19
T6S35			991	0.34	124	0.0148	122	0.0065	125	0.99	105	1.18
T6S50			1461	0.51	133	0.0112	131	0.0065	132	1.00	98	1.36
T6S70			1971	0.68	117	0.0068	116	0.0065	128	0.92	81	1.45
T6C50			1461	0.51	145	0.0061	144	0.0065	134	1.08	98	1.48
T9S03		30	98	0.04	76	0.0041	NA	0.0102	65	1.17	63	1.20
T9S35			941	0.34	133	0.0127	130	0.0102	123	1.08	100	1.33
T9S70			1883	0.68	152	0.0119	146	0.0102	145	1.05	76	2.01
T9S90			2501	0.91	146	0.0186	139	0.0102	133	1.10	28	5.27
T9C70			2050	0.74	174	0.0076	NA	0.0102	144	1.21	69	2.50

fc: Compressive strength of concrete cylinder
B/t: Width-to-wall thickness ratio of steel tube
N: Axial load applied
n: Axial load ratio
M_m: Experimental ultimate strength
M_{cm}: Experimental moment at ε_{cm}
ε_m: Ultimate strain of extreme concrete fiber at M_m
ε_{cm}: Theoretcal ultimate strain [3]
M_p: Ultimate strength calculated using the proposd stress block
M_{ACI}: Ulitmate strength calculated using the ACI stress block [2]

a stress block for estimating ultimate strain and strength of the tubed column is proposed.

2. Experimental Work

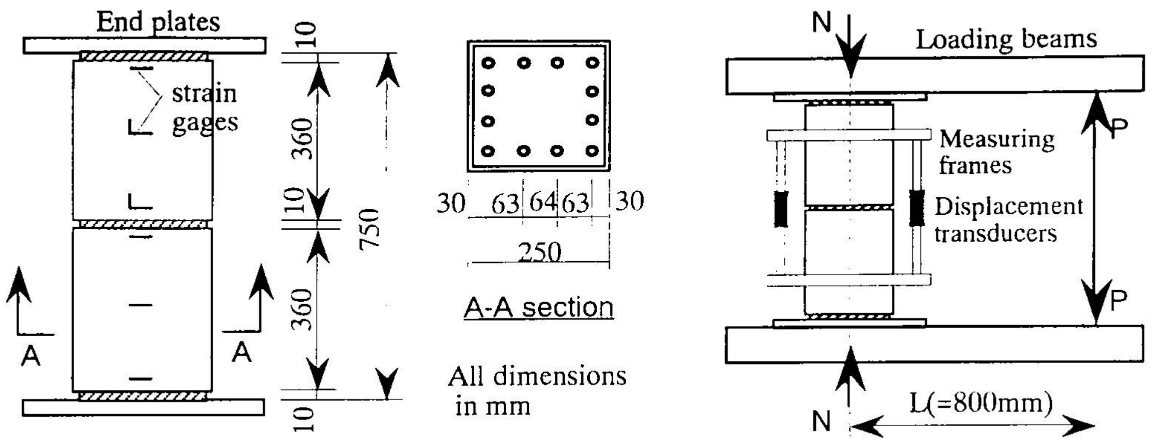
2.1 Specimens and Testing Method

Test specimens were 250x250x750mm prismatic columns with the same concrete strength and amount of longitudinal bars. A total of ten specimens were divided into two series, T6 and T9, according to wall thickness (6mm and 9mm) of the steel tubes used. Experimental variables among each series of specimens were axial load ratio and confining types of square steel tubes (split or continuous). Details and primary results of all specimens are listed in Table 1.

Longitudinal bars in each specimen consisted of twelve 13mm diameter (D13) deformed bars, which were welded to the end plates, to give a steel ratio of 2.69%. Confining steel tubes were provided by square steel tubes of 250x250x6mm and 250x 250x9mm available on the market. The yield strengths of the D13 deformed bar, T6 steel tube and T9 steel tube are 340MPa, 303MPa and 300MPa, respectively. Concrete with designed strength of 42MPa was made of Portlant cement and aggregate with a maximum size of 20mm. The average compressive strength of concrete cylinders during testing is given in Table 1.

Fig. 1 shows the sectional details of specimens and the loading condition. In order for the steel tube not to directly sustain the axial stresses due to the axial load and bending moment, clearances of 10mm were provided between the steel tube and end steel plates at both ends of specimens. The steel tubes of four specimens (S-specimen) in each series were split into halves at the middle of the column, and a clearance of 10mm was provided between the two halves. This was to avoid any unexpected flexural stresses in the confining steel tubes during the bending action. For the fifth specimen (C-specimen), the steel tube was left intact to investigate the effect of continuity of the steel tube. The values of axial load ratio, defined as N/A_cf_c', for the T6-series specimens were 0.03, 0.35, 0.50, and 0.70, while those of the T9-specimens were 0.03, 0.35, 0.70 and 0.90.

As shown in Fig. 1(b), the bending moment was exerted by pushing the loading beams through



(a) details of specimens (b) loading condition

Fig.1 Details of specimens and loading condition

two hydraulic jacks after applying the axial load N. The axial load was applied by means of a 5MN universal testing machine, and was adjusted continuously so that the axial loads (N-P) in the specimens was maintained constant during the tests.

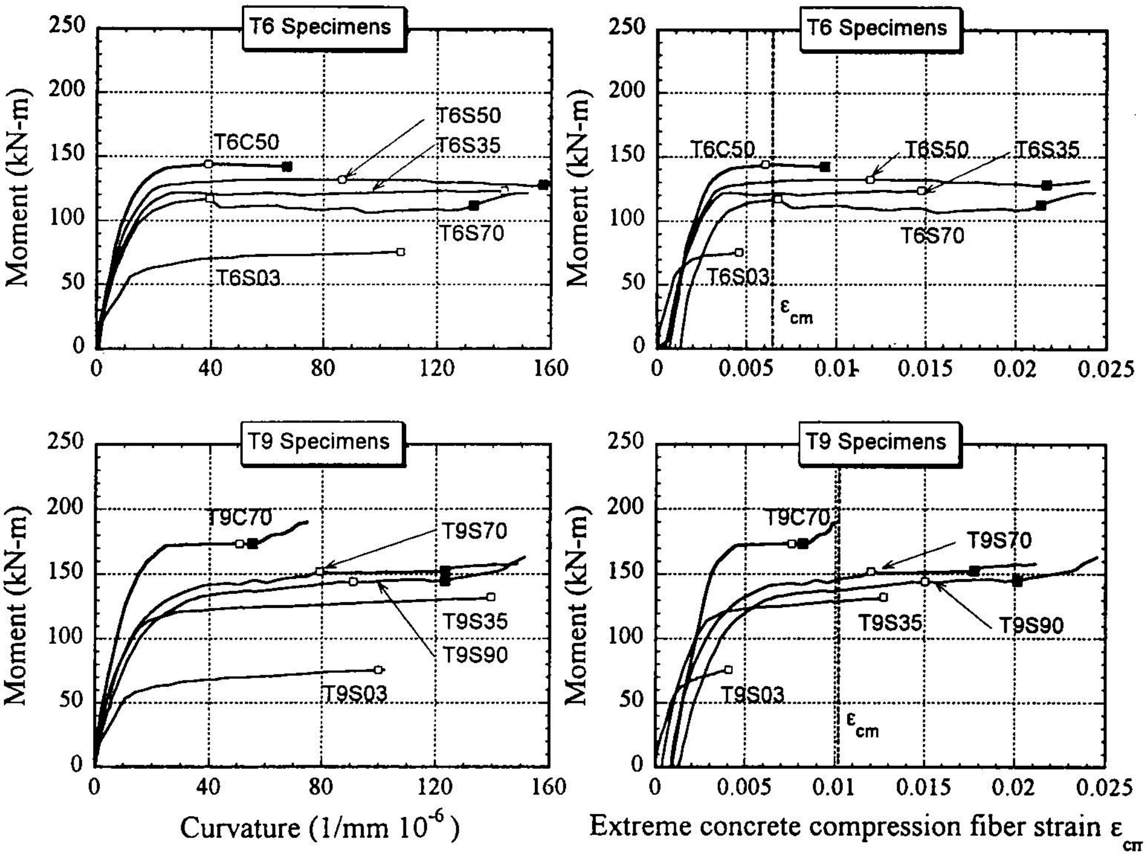
Strains were measured by 10-mm strain gages mounted on the compression and tension surfaces of the steel tubes. (See Fig. 1(a)). The average curvature over the central 500mm testing region of the specimen was measured by a pair of displacement transducers attached to measuring frames. Lateral deflection at the middle of the specimen was measured by a displacement transducer with reference to rotating centers to allow for the calculation of the secondary moment caused by the axial load N.

2.2 Experimental Results

Experimental results of the ultimate strength M_m and the corresponding ultimate strains of extreme concrete fiber ε_m are listed in Table 1. Fig. 2(a) and (b) shows the moment-curvature relations and the moment-ε_{cc} relations, respectively. In Fig. 2, ε_{cc} denotes compression strain of extreme concrete fiber, and the open squares and solid squares represent the testing stages when moment-curvature curves reached their peaks and when the steel tubes contacted the end plates, respectively. The dotted lines superimposed in moment- ε_{cc} curves express the theoretical ultimate strains ε_{cm}, which will be described in the following section. Experimental bending moments corresponding to the theoretical ε_{cm} are listed in Table 1 as M_{cm}.

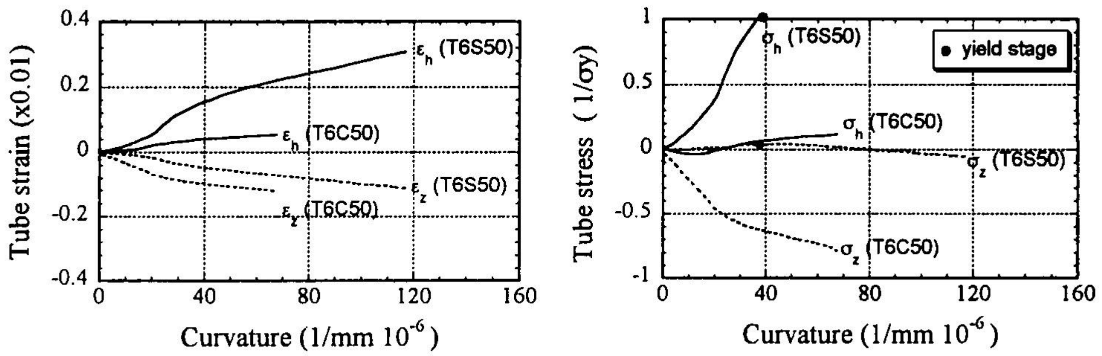
It will be seen from Fig. 2 that specimens of T6 and T9 series exhibited very stable behavior even under such a high axial load as n=0.7 and n=0.9, respectively. Higher ultimate strength was observed in specimens encased by the steel tube with 9mm wall-thickness. On the other hand, in specimens under very low axial load, T6S03 and T9S03, effect of the wall thickness on the ultimate strength was not remarkable. Because low axial load results in small compression area, hence the ultimate strength is less sensitive to the confinement degree of steel tubes.

As obvious in Table 1, the experimental ultimate strains at extreme concrete compression fiber ε_m showed a large scattering and varied from 0.004 to 0.019, much higher than the value of 0.003 as recommended in the ACI code [2]. The ultimate strains appear to be mainly affected by the wall thickness of steel tube, and no clear correlation existed between the ultimate strains and the axial



(a) moment-curvature relationships (b) moment-ε_{ce} relationships

Fig.2 Experimental results

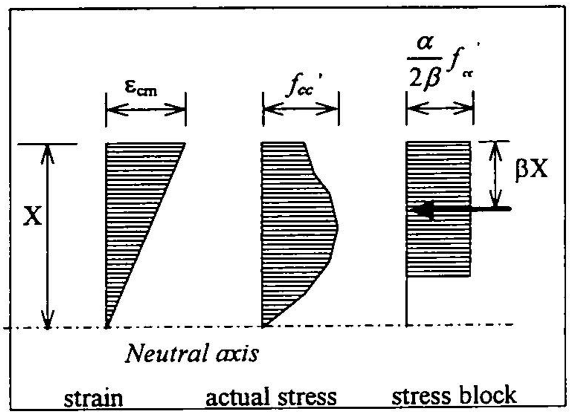


(a) tube strain-curvature relationships (b) tube strain - curvature relationships

Fig. 3 Strain and stress states in the steel tubes

load levels. It can also be seen from Table 1 that the experimental moments M_{cm} at the theoretical ε_{cm} closely approximated the experimental ultimate strengths M_m. Therefore, instead of a experimental formula for ε_m, it is reasonable to utilize the theoretical ε_{cm} for estimating the ultimate strength.

By comparing test results of two pairs of specimens, T6S50 and T6C50, and T9S70 and T9C70, it can be seen that the C-specimens exhibited higher ultimate strengths than S-specimens. To



(a) distributions of strain and stress

$$\frac{\varepsilon_{cm}}{\varepsilon_{co}} = 1.375 + 0.108K - 0.102K^{-4}\left(\frac{f_c'}{42}\right) \quad (1)$$

$$\alpha = 0.724 + 0.107K - \frac{0.037}{K - 0.007}\left(\frac{f_c'}{42}\right) \quad (2)$$

$$\beta = 0.383 + 0.046K - \frac{0.019}{K + 0.387}\left(\frac{f_c'}{42}\right) \quad (3)$$

$$K = \frac{f_{cc}'}{f_c'} = 1 + 11.5\frac{\rho_t f_{yt}}{f_c'}\left(\frac{t}{B - 2t}\right) \quad (4)$$

$$\frac{\varepsilon_{co}}{\varepsilon_o} = \begin{cases} 1 + 4.7(K - 1), & K \le 1.5 \\ 3.35 + 20(K - 1.5), & K > 1.5 \end{cases} \quad (5)$$

(b) parameters for the stress block

Fig. 4 The proposed stress block

investigate the reason for this phenomenon, the strain and stress states of steel tubes for these specimens were examined. Fig. 3 shows experimental results of the T6 specimens only, since the T9 specimens had similar results. Fig. 3 shows the lateral strains ε_h and axial strains ε_z of the steel tube measured on the compression surface at the middle of the specimens. The lateral stresses σ_h and axial stresses σ_z were calculated following Hook's law from measured ε_h and ε_z. Solid circles in Fig.3(b) express the stage when the steel tubes reached Von Mises' yield criterion.

For S-specimen encased by split tube, the ε_h was about three times of the ε_z , and the lateral stresses σ_h were predominant, which means that the steel tube mainly acted as a lateral confiner. On the other hand, the ε_z in continuous tube of the C-specimen showed larger values than the ε_h. The axial stresses σ_z became predominant, which implies that the steel tube in C-specimen mainly sustained the axial stress incurred during the bending process. Therefore, it is necessary to cut the continuity of the steel tube when experimentally studying confinement effect of the steel tube.

3. Theoretical Work

3.1 Stress Block and Ultimate Strain

It is well known that ultimate strength of a reinforced concrete section can be simply calculated by utilizing the stress block of the compressed concrete. As apparent in Table 1, however, the widely used ACI stress block resulted in very conservative estimation of ultimate strengths of the tubed columns because of conservative nature of the ultimate strain of 0.003 as well as ignorance of confinement effect of the steel tube in the ACI stress block. To calculate the ultimate strength of a tubed column section more accurately, a new stress block, where confinement effect of the steel tube can be taken into consideration, is proposed in this section. Fig. 4 shows details of the proposed stress block and expressions for related parameters, the ultimate strain ε_{cm}, α and β.

Generally, expressions for α and β corresponding to any strain at extreme concrete fiber ε_{ce} can be derived by equalizing the axial force and moment of the stress block with that of the actual stress distribution. However, when evaluating ultimate strength, expressions corresponding to the specific strain at peak of moment-curvature curve are desirable rather than the general ones.

Based on authors analytical work on the moment-curvature behavior of the confined concrete section[3], Eq.1 was derived to estimate the ultimate strain ε_{cm}, Eqs.2 and 3 to evaluate parameters α and β. The process for developing Eq.1 through Eq.3 can be found elsewhere [3].

In Eq.1 through Eq.5, K=ratio of confined concrete strength to unconfined cylinder strength, a factor representing confinement degree of steel tube[4]; $\varepsilon_o = 0.94(f_c')10^{-3}$; f_c'=concrete cylinder strength; ρ_t=volumetric ratio of steel tube; f_{yt}=yield stress of steel tube; B and t=width and wall thickness of steel tube, respectively. Note that stresses f_c' and f_{yt} in Eq.1 through Eq.5 are in MPa.

3.2 Comparison Between Measured and Theoretical Ultimate Strengths

It can be seen from Table 1 that for S-specimens, the ACI strengths M_A are 18% to 427% conservative, while the ultimate strengths M_p obtained by using the proposed stress block predict experimental results very well. The measured ultimate strengths M_m exceeded the predicted results M_p only 6% on average for eight S-specimens. For specimens encased by continuous tube, the proposed stress block should not be applied to calculate the ultimate strength, since the steel tube in these specimens mainly sustained the axial stresses rather than acted as a lateral confiner.

4. Conclusions

The following conclusions can be drawn from the study reported in this paper on the ultimate strain and strength of the reinforced concrete column retrofitted by square steel tubes.

(1) When confined by square steel tube, the ultimate strain at extreme concrete compression fiber was not a constant, but varied from 0.004 to 0.019. Because of large scattering in the experimental ultimate strains and scarcity of test data, Eq.1 is proposed to predict the ultimate strain instead of developing an experimental formula for the ultimate strain. The fact that the experimental moments at the theoretical ε_{cm} closely approximated the ultimate strengths implies that Eq.1 would give reasonable results for design purpose.

(2) Ultimate strengths of the tubed columns increased with the increase of wall thickness of the confining tube. The increment in ultimate strength due to use of thicker tube becomes significant when the axial load level is high.

(3) A new stress block for the confined concrete is proposed to evaluate the ultimate strength of reinforced concrete columns retrofitted by square steel tubes(See Eq.1 - Eq.5). The theoretical ultimate strength predict the experimental result conservatively only by 6% on average.

References

[1] Sun, Y.P. and Sakino, K., "Flexural Behavior of Reinforced Concrete Columns Confined in Square Steel tubes," Procs. of the Tenth WCEE, Madrid, SPAIN, Vol. 8, July 1992, pp. 4365-4370.
[2] ACI Committee 318, "Building Code Requirements for Structural Concrete and Commentary - ACI 318-95," ACI, 1995, pp. 104-105.
[3] Sun, Y.P., Sakino, K., and Yoshioka, T., "Flexural Behavior of High-Strength RC Columns Confined by Rectilinear Reinforcement," J. of Struct. Constr., AIJ, No.486, Aug.1996, pp.95-106.
[4] Sakino, K. and Sun, Y.P., "Stress-Strain Curve of Concrete Confined by Rectilinear Hoops," J. of Struct. Constr., AIJ, No.461, July 1994, pp.95-104. (in Japanese)