Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Flexural behavior of reinforced concrete columns in square steel tube

Y.P. Sun & K. Sakino
Kyushu University, Fukuoka, Japan

ABSTRACT: Experimental and theoretical investigations conducted to study the flexural strength enhancement and ductility improvement by encasing the reinforced concrete columns with square steel tube are described. It is shown that the columns can be not only prevented from brittle shear failure, but also made to exhibit stable behavior even under high axial compression when confined in square steel tube.

1 INTRODUCTION

In order to bring on adequate ductility to reinforced concrete columns subjected to major earthquake, a new reinforcing method , which is called transversely-super-reinforcing (T.S.R.) method, has been proposed (Tomii 1987). In this method, the steel tubes are utilized to transversely confine the potential plastic hinge regions of reinforced concrete columns in lieu of common transverse hoops or ties.

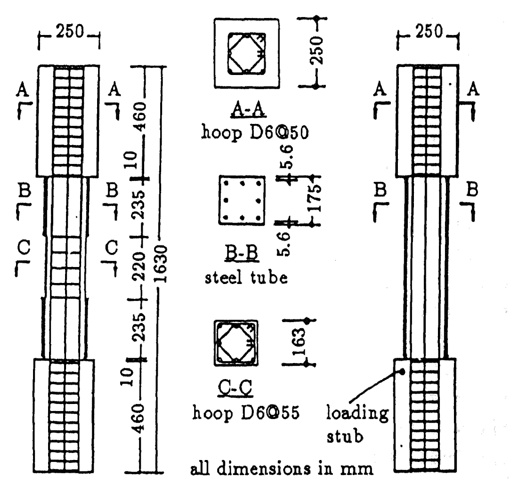
Besides three-fold functions of the common transverse reinforcement, which are to confine the compressed concrete, to prevent lateral buckling of longitudinal bars, and to provide transverse shear reinforcement, the steel tube has two more functions: (1) to prevent the spalling off of the cover concrete, which is generally considered to be the main reason for brittle splitting bond failure, and (2) to act as a form for the column.

The purpose of this research is to present information on the flexural behavior of reinforced concrete columns locally or wholly confined in square steel tubes (referred to as tubed column hereafter). Furthermore, a simple method to evaluate the ultimate moment of the tubed columns is presented, and comparison between the experimental and theoretical ultimate moments calculated by using this method is aslo conducted.

2 TEST PROGRAM

In order to verify the enhancement of ultimate moment and the improvement of ductility by using the T.S.R. method, eight columns locally or wholly confined in square steel tube with wall thickness of 5.6 mm were fabricated and tested. The test specimens are one-third scale model with aspect ratio M/(VD)= 2.0, and the reinforcement details for specimens are shown in Figure 1. In each test specimen eight 12.7 mm diameter deformed bars called D13 were used as longitudinal bars to give a steel ratio of 3.84%.

Test specimens were divided into two groups according to the length of steel tube used to confine the columns. Spaces with length of 10mm were provided between the loading stub and steel tube to ensure that the steel tube only provided a confining effect, rather than directly sustained the axial stress. For four locally confined test specimens, the



(A) locally confined (B) wholly confined

Figure 1 Reinforcement details of test specimens

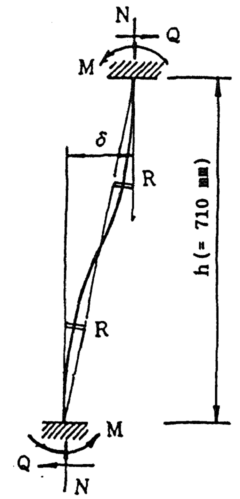


Figure 2 Loading condition

Table 1 Details and main results of test speimens

Specimen	Column Type	f'_c (MPa)	N (kN)	$\frac{N}{Af'_c}$	V_u^+ (kN)	V_u^- (kN)	M_{exp} (kN-m)	M_{cal} (kN-m)
L8-04		43.1	461	0.4	134	132	50.8	47.9
L8-07	Locally	46.5	873	0.7	159	161	61.7	53.3
L8-08	Confined	43.2	926	0.8	166	159	65.1	52.7
L8-09		43.2	1040	0.9	164	170	62.4	51.6
T8-04		43.1	461	0.4	134	133	50.9	47.9
T8-09	Wholly	43.2	1040	0.9	160	170	62.2	51.6
T8-10	Confined	43.2	1157	1.0	171	171	65.1	49.5
T8-11		46.5	1371	1.1	168	172	64.4	48.8

Note
1) f'_c: compressive cylinder strength of concrete
2) N: axial load applied
3) V_u^+, V_u^-: ultimate lateral load in positive and negative loading
4) M_{exp}, M_{cal}: experimental and theoretical ultimate moment

mid parts of them, which were not confined in steel tube, were transversely reinforced using 6 mm diameter deformed bars called D6 in order to prevent premature shear failure from occuring. The arrangement of transverse reinforcement for the locally confined specimens is also shown in Figure 1. This arrangement is typical for 8-bar columns.

The yield strengths of used steel tube, D13 bar and D6 bar were 426 MPa, 330 MPa, and 321 MPa, respectively. High strength concrete, with compressive strength of 43.1 - 46.5 MPa, was used for making the test specimens. The compressive cylinder strength of concrete at the stage of testing the specimen is shown in Table 1. The concrete was normal weight concrete with a slump of 75 mm and a maximum aggregate size of 20 mm.

The parameter varied among specimens of each group was the magnitude of axial load applied, whose level is expressed by the ratio $N/(Af'_c)$, where N is the axial load applied, A is the confined concrete area, and f'_c is the compressive cylinder strength of concrete. The values of axial load ratio $N/(Af'_c)$ were 0.4, 0.7, 0.8, and 0.9 for locally confined specimens, and 0.4, 0.9, 1.0, and 1.1 for wholly confined specimens. The values of axial load ratio $N/(Af_c)$ of 0.4 and 0.7 were nearly equal to the upper limits recommended in AIJ Design Guidelines (1990) for ordinary confined reinforced concrete column and well transversely confined reinforced concrete column, respectively. Therefore, axial load ratio equal to or greater than 0.8 might be considered such that under the axial load of this level the reinforced concrete column cannot be expected to behave in a stable manner even well transversely confined by common hoops or ties. The details of test specimens are given in Table 1 with the main test results.

The test set-up described in reference (Tomii 1987) is used to apply the axial load and cyclic lateral load to specimen, and deform the specimen in a double curvature pattern shown in Figure 2. The loading pattern was a cyclic type with alternating drift reversals. The peak drifts were increased stepwise from 0.005h, where h was the clear height of the column, until 0.03h with increment of 0.005h after three complete cycles at each drift level. Finally, one cycle was applied at a peak drift of 0.06h. For wholly confined specimen with axial load ratio $N/(Af_c)$ of 1.1, it was observed that the steel tube touched the loading stubs during the third positive cycle of 0.02h peak drift, and from then the cycle of loading was terminated.

3 LATERAL LOAD-TRANSLATION ANGLE RELATIONSHIPS

Lateral load - translation angle hysteresis loops are shown in Figure 3 and Figure 4 for locally confined specimens and wholly confined specimens, respectively. In these figures, translation angle of the columns R is defined as δ/h, where δ is the lateral displacement. Open and solid triangles shwon in these figures express the testing stages when the steel tube partially and completely touched the loading stubs, respectively.

Solid and linked lines superimposed on the measured lateral laod - translation angle hysteresis loops are the theoretical ultimate lateral loads based on the ultimate moment capacities calculated from two kinds of equivalent rectangular stress blocks. One of them corresponding to the solid lines was based

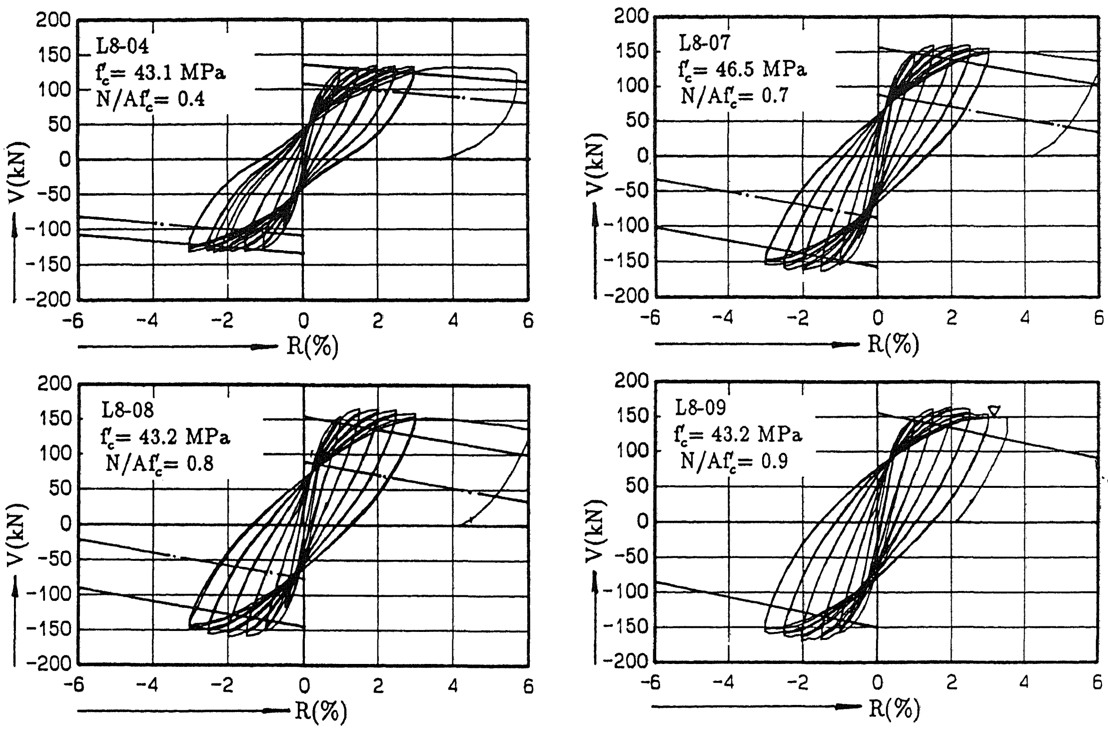


Figure 3 Lateral load V - translation angle R relationships for locally confined specimens

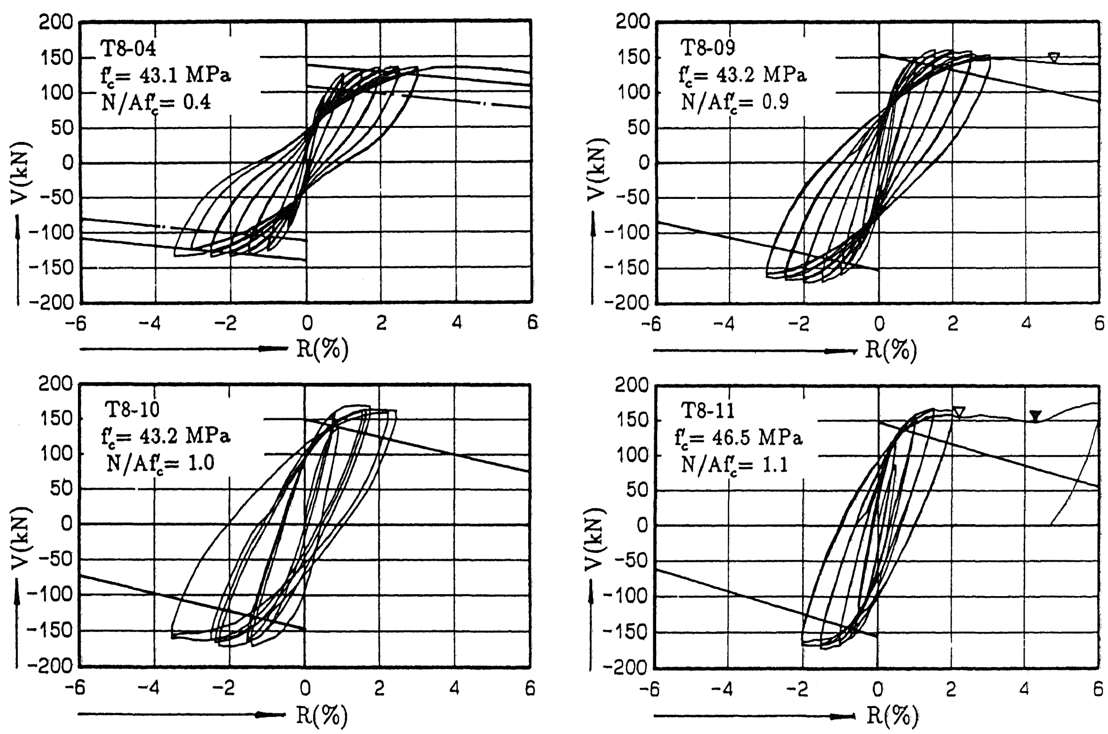


Figure 4 Lateral load V - translation angle R relationships for wholly confined specimens

on a stress-strain curve for concrete confined in the square steel tube proposed by authors (Sakino and Sun 1991), which will be mentioned in the following section, and the other corresponding to the linked lines was the stress block recommended in ACI Code (1990). It has been pointed out that when the neutral axis lies outside the section, the equivalent rectangular stress block recommended in ACI Code cannot be applied (Park and Paulay, 1975). Therefore, for test specimens with axial load ratio greater than 0.8, only the solid lines are presented in these figures, because the calculated neutral axis depths were greater than the depth of the section for these specimens.

From the test results shown in Figure 3 and Figure 4, the following observations can be made:

1. All the test specimens, locally or wholly confined in square steel tubes, exihibit relatively stable bahavior up to translation angle of 0.03 without serious strength or stiffness degradation even under high axial load levels such as the axial load ratio is 0.9 for locally confined columns and 1.1 for wholly confined columns.

2. There is no indication of shear deformation resulting in the pinching effect of the hysteresis loops except for test specimens with low axial load ratio of 0.4, where only a slight pinching effect is observed. All the specimens show large energy absorption capacity.

3. All the specimens reached or exceeded the theoretical ultimate moments calculated by using a equivalent rectangular stress block, in which the confinement effects of the square steel tube are taken into consideration.

4. The hysteresis loops and maximum lateral loads of the specimens only whose plastic hinge regions are confined in steel tube are similar to those of the specimens whose total height is confined in steel tube.

4 ULTIMATE MOMENT

As shown in Figure 3 and Figure 4, the ultimate moment capacity based on the ACI Code results in conservative estimation of the ultimate lateral load, particularly for specimens subjected to high axial load. Main reasons for these discrepancies are the conservative nature of the ultimate compressive strain of ϵ_{cm}=0.003, adopted in ACI Code, and an increase in concrete compressive strength due to the confinement provided by the square steel tube and loading stubs. Therefore, it can be said that accurate assessment of the flexural strength of column needs to use equivalent stress block parameters of concrete which take into account the confinement effects provided by the steel tube. In this section, a simple method is introduced to estimate the ultimate moment capacity of the tubed columns using a equivalent stress block whose parameters are based on a stress-strain curve model for confined concrete, and the comparision between the experimental and theoretical ultimate moments is conducted to verify the validaty of this method.

4.1 *Stress-Strain Curve Model for Confined Concrete*

The model for the compressive stress-strain behavior of concrete confined in the square steel tube has been proposed by authors (Sakino and Sun, 1991). This model is proposed based on the experimental results of concentrically loading test conducted at Kyushu University of Japan, and gives complete stress-strain curve up to very high concrete strain from which the parameters of the concrete equivalent stress block may be calculated at any extreme fiber compressive strain in a member (Sun, 1992).

The stress-strain curve model suggested by authors takes into consideration both the enhancement in concrete compressive strength and the increase in concrete ductility due to the confinement from the square steel tube, and is defined as follow:

$$Y = \frac{AX + (D-1)X^2}{1 + (A-2)X + DX^2} \quad (1)$$

where $Y = f'_c/f'_{cc}$; $X = \epsilon_c/\epsilon_{co}$; f'_c and ϵ_c are the stress and strain; f'_{cc} and ϵ_{co} are the stress and strain at the peak, $A = E_c/E_{sec}$, $E_c = (0.69 + 0.332\sqrt{f'_c}) \times 10^4$ is the Young's modulus of elasticity of concrete in MPa (Martinez, 1984), E_{sec} $(=f'_{cc}/\epsilon_{co})$ is the secant modulus at the peak point, and D is the parameter governing the shape of descending portion of the stress-strain curve.

In order to predict the stress-strain curve, it is necessary to determine the values of three parameters in Eq.1: (1) the strength of confined concrete, f'_{cc}, (2) the strian corresponding to the peak stress, ϵ_{co}, and (3) parameter D. The expressions for these three paramenters were determined from regression analysis on the test results of 175 mm square and 350 mm high concrete stub columns. These columns were tested under concentrically axial compression and were confined in square steel tubes with outside width to wall thickness ratio between 31 - 107. Their compressive cylinder strengths of concrete varied from 20.6 MPa to 67.7 MPa. The following equations were derived,

$$f'_{cc} = f'_c[1 + 11.5(\frac{t}{B})(1 - \frac{t}{2B})\frac{\rho_h f_{ty}}{f'_c}] \quad (2)$$

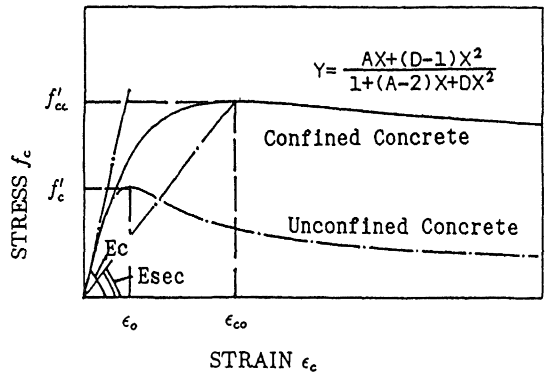


Figure 5 Proposed stress-strain curve for concrete

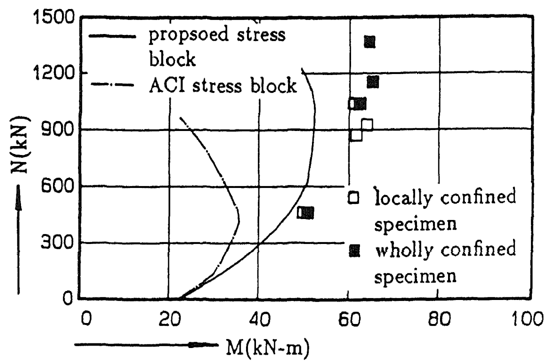


Figure 6 Comparison between the experimental and theoretical ultimate moment

$$\epsilon_{co} = \epsilon_o \begin{cases} 1 + 4.7(K-1), & K \leq 1.5 \\ 3.35 + 20(K-1.5), & K > 1.5 \end{cases} \tag{3}$$

$$D = 1.5 - 0.017 f'_c + 2.49\sqrt{(K-1)f'_c/23} \tag{4}$$

where $K = f'_{cc}/f'_c$, f'_c is concrete compressive cylinder strength, $\rho_h = (\frac{B}{B-2t})^2 - 1$ is the ratio of volume of steel tube to volume of concrete core, f_{ty} is the yield strength of steel tube, B and t are the outside width and wall thickness of steel tube, respectively, and $\epsilon_o = 0.937(f'_c)^{1/4} \times 10^{-3}$ (Popovics, 1973). A typical example of the proposed stress-strain curve model is shown in Figure 5.

4.2 *Compressive Stress Block and Ultimate Strain*

It is well known that the flexural strength of reinforced concrete section can be estimated as the maximum moment on the moment- thrust-curvature relationship for given axial load. This method, however, involves an iterative procedure to find the depth of the neutral axis for the internal forces to balance the external applied load, and becomes extremely tedious when the stress-strain curve model of concrete employed is complicated. An attempt to simplify the calculation procedure of the flexural strength is necessary. This simplification can be made by replacing the actual stress distribution of concrete in compressive zone with an equivalent rectangular stress block with total compressive force C $(=\alpha f'_{cc} bx)$ acting at distance γx from the extreme compression fiber, where b is the width of section, and x is the distance from the neutral axis to the extreme compression fiber.

The general expressions for stress block parameters α and γ corresponding to any given concrete extreme fiber strain can be derived by equalizing the compressive force and the first moment of the stress block with that of actual stress distribution, and are given as follows:

$$\alpha = \frac{\int_o^{\epsilon_{cm}} f_c d\epsilon_c}{f'_{cc}\epsilon_{cm}}, \quad \gamma = 1 - \frac{\int_o^{\epsilon_{cm}} f_c \epsilon_c d\epsilon_c}{\epsilon_{cm}\int_o^{\epsilon_{cm}} f_c d\epsilon_c} \tag{5}$$

However,when evaluating the ultimate moment capacity of the column section, the expressions for α and γ corresponding to a specific concrete extreme fiber strain ϵ_{cm} is necessary rather than the general ones given by Eq. 5. This specific concrete strain corresponding to the maximum moment on the moment-thrust-curvature relationships is referred to as ultimate strain in this paper. The value of ultimate strain ϵ_{cm} has been found not to be a constant like that recommended in ACI Code, but a variable affeced by the conrete strength and the degree of confinement from the steel tube, and is proposed to be $1.5\epsilon_{co}$ which is given by Eq. 3 (Sun, 1992). The approximate expressions for α and γ corresponfing to the ultimate strain $\epsilon_{cm} = 1.5\epsilon_{co}$ are derived as

$$\alpha = \begin{cases} 0.724 + 0.107K - \frac{0.036}{K - 0.007}(\frac{f'_c}{42}), & K > 1 \\ 0.831 - 0.076(\frac{f'_c}{42}), & K = 1 \end{cases} \tag{6}$$

$$\gamma = 0.383 + 0.046K - \frac{0.019}{K + 0.387}(\frac{f'_c}{42}) \tag{7}$$

where f'_c is in MPa.

4.3 *Comparison Between Experimental and Theoretical Results*

The ultimate moment capacity, which is usually expressed in the form of the interaction curve between the axial load and ultimate moment, is estimated on bases of the following assumptios: (1) plane sections remain plane after bending, (2) longitudinal bars are elastic perfectly plastic materail, (3) tensile strength of concrete is neglected, and (4) the conrete ultimate strain ϵ_{cm} is 1.5 ϵ_{co}. The calculation method dose not need iterative procedure, and is summaried as follows:

1. An initial value is given to x, distance from the extreme compression fiber to neutral axis.

2. Based on the first two assumptions, the strains and then the stresses of longitudinal bars and concrete can be computed.

3. The axial load and coreesponding moment are given as

$$N = \alpha f'_{cc} bx + \sum_{i=1}^{n} f_{si} A_{si} \tag{8}$$

$$M = \alpha f'_{cc} bx(\frac{h}{2} - \gamma x) + \sum_{i=1}^{n} f_{si} A_{si} (\frac{h}{2} - h_{si}) \tag{9}$$

where the values of parameters α and γ are given by Eq. 5 and Eq. 6, respectively.

4. The calculation procedure from step 2 to step 3 for a new value of x is repeated until x = h.

The theoretical interaction curve between the axial load and ultimate moment obtained by the method mentioned above is shown in Figure 6, along with the experimental results. The open and solid squares shown in Figure 6 express the experimental ultimate moments, defined as the maximum column end moment at the positive loading within the drift level of 0.03h and including the N-δ moment, for locally and wholly confined specimens, respectively. The theoretical ultimate moments of all specimens are also listed in Table 1.

From Figure 6 and Table 1, it can be seen that the using the equivalent stress block proposed in this paper can give a reasonably accurate lower bound of the ultimate moment for tubed columns. The main reason for the discrepancies between the experimental and theoretical ultimate moments is that the extra confinement effect from the stiff loading stubs at large deformation would shift the critical section away from the end section of columns, and this effect would become prominent for columns under higher axial load.

5 CONCLUSIONS

The following conclusions can be drawn on bases of the study on the flexural behavior of the reinforced concrete columns confined in the square steel tube.

1. The T.S.R method can not only bring on adequate ductility to the reinforced concrete columns even under very high axial load, but also remarkably increase the lateral load capacity of columns due to the confinement effects of the steel tube.

2. The reinforced concrete columns only whose potential plastic hinge regions are confined by the T.S.R. method behave in a manner similar to that of columns whose total height is confined by the T.S.R. method. Therefore the former columns are superior to the latter columns from the viewpoint of economy.

3. The ultimate moment capacity of the tubed columns can be estimated in a reasonably accurate way using the equivalent stress block of the concrete proposed in this paper.

REFERENCES

ACI 318-89, ACI Committee 318. 1989. *Building Code Requirements for Reinforced Concrete.* American Concrete Institute, Detroit, Mich,. 107 - 109.

Architectural Institute of Japan (AIJ). 1990. *Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Ultimate Strength Concept* (in Japanese).

Martinez, S., Nilson, A.H. and Slate, F.O. 1984. *Spirally Reinforced High-Strength Concrete Columns.* ACI *Journal,* Vol. 81, No. 35. 431 - 442.

Park, R. and Paulay, T. 1975. *Reinforced Concrete Structures.* John Wiley and Sons, Inc., New York. 130.

Popovics, S. 1973. *A Numerical Approach to Complete Stress-Strain Curve of Concrete.* Cement and Cpncrete Research, Vol. 13. 583 - 599.

Sakino, K. and Sun, Y.P. 1991. *Behavior of Concrete Confined in Square Steel Tubes under Axial Compression.* Transactions of JCI, Vol. 13. 495 - 502.

Sun, Y.P. 1992. *Elasto-Plastic Behavior of Reinforced Concrete Columns Confined by Rectilinear Transverse Reinforcement* (in Japanese). Thesis of Engineering Doctor Submitted to Kyushu University, Japan.

Tomii, M., Sakino, K. and Xiao, Y. 1987. *Ultimate Moment of Reinforced Concrete Short Columns Confined in Steel Tube.* Proceedings of Pacific Conference on Earthquake Engineering, Vol. 2, New Zealand. 11 - 22.