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A MODEL OF CONFINEMENT EFFECT ON STRESS–STRAIN RELATION OF REINFORCED CONCRETE COLUMNS FOR SEISMIC DESIGN

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ABSTRACT

In this paper, a stress–strain model of concrete which takes confinement effects into account is developed, based on the results of a series of compression loading tests of reinforced concrete column specimens. The stress–strain model was formulated by evaluating the relationship between several key parameters and the stress–strain behavior observed in the experiments. It is shown that the predicted stress–strain relation provides better agreement with the experimental results than previous models.

KEYWORDS

Reinforced Concrete Bridge Piers, Seismic Design, Stress–Strain Model, Confined Concrete, Lateral Reinforcment, Compression Loading Test

INTRODUCTION

Various studies on the confinement effects of lateral reinforcement in column have already been conducted (Kent *et al.*, 1971, Sheikh *et al.*, 1982, Muguruma *et al.*, 1980, Mander *et al.*, 1988, Fujii *et al.*, 1988a, Saatcioglu *et al.*, 1992). The focus of the paper is discussed below.

Reinforced concrete bridge piers constructed in Japan have larger concrete sections and thus lower hoop reinforcement volumetric ratios than those constructed in the United States and New Zealand. In Japan, reinforced concrete bridge piers generally have a volumetric ratio of 0.3% to 0.5% of hoop reinforcement, and hoops with a yield stress of 295 MPa or 345 MPa. The models described above were developed based on data from the loading tests (Muguruma *et al.*, 1978, Sheikh *et al.*, 1980, Park *et al.*, 1982, Mander *et al.*, 1988b, Fujii *et al.*, 1988, Razvi *et al.*, 1989). Although a few specimens containing a volumetric ratio of hoop reinforcement less than 0.5% have been tested previously, they must be considered special cases, because high tensile strength steel was used for the hoop reinforcement, and some of the specimens were subject to eccentric loading. Therefore, the previous models may not adequately represent confinement effects in the range of low hoop reinforcement ratios. It is also pointed out that only Mander's and Saatcioglu's models can be directly applied to concrete bridge piers with wall–type cross sections. It is, therefore, necessary to study the confinement effects for wall–type sections as well.

Table 1 Test specimens

Specimen			Dimension of Section and Height (mm)	Strength of Unconfined Concrete (MPa)	Longitudinal Reinfor-cement Ratio (%)	Hoop Reinforcement				Aspect Ratio of Section
						Material / Diameter(mm)	Spacing (cm)	Volumetric Ratio (%)	Anchorage Type	
S	C	SC0	ϕ200 h=600	18.5	0	–	–	–	–	–
		SC1				SR235 ϕ6	15	0.39	Weld	
		SC2					10	0.58		
		SC3					5	1.17		
		SC4					2.5	2.33		
		SC5					1.25	4.66		
	S	SS0	200 × 200 h=600	23.2	0	–	–	–	–	1.0
		SS1				SR235 ϕ6	15	0.39	Weld	
		SS2					10	0.58		
		SS3					5	1.17		
		SS4					2.5	2.33		
		SS5					1.25	4.66		
L	C	LC0	ϕ500 h=1500	28.8	1.01	–	–	–	–	–
		LC1				SD295 D10	30	0.19	Weld	
		LC2					15	0.39		
		LC3					10	0.58		
		LC4					5	1.16		
		LC5				SD295 D13	30	0.34		
		LC6				SD295 D16	30	0.54		
		LC7				SD295 D10	10	0.58	90° Hook	
		LC8							135° Hook	
		LC9							180° Hook	
	S	LS0	500 × 500 h=1000	24.3	0.95	–	–	–	–	1.0
		LS1				SD295 D13	6	1.73	Weld	
		LS2				SD295 D16	7.5	2.19		
		LS3				SD295 D13	4	2.60		
		LS4				SD295 D16	4	4.10		
	W	LW1	350 × 700		0.97	SD295 D13	6.5	1.72		2.0
		LW2	300 × 900		1.03		6.7	1.74		3.0
		LW3	250 × 1000		0.95		7.5	1.77		4.0
		LW4						2.45		

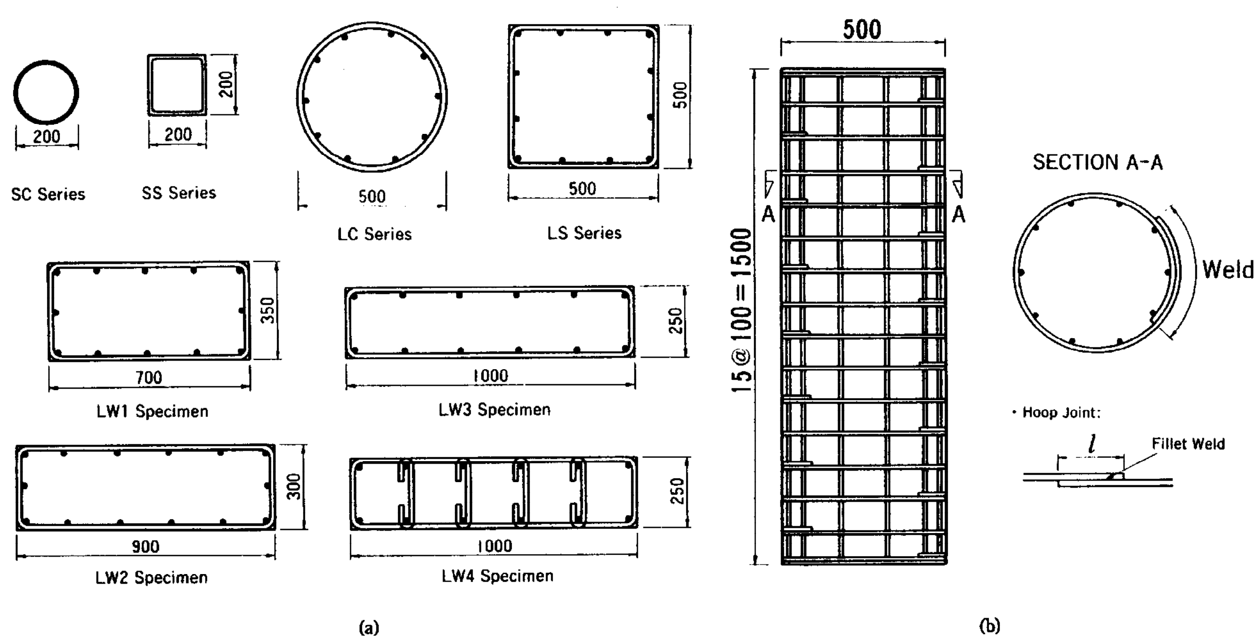

Fig. 1 Construction details of specimens: (a) cross sections; (b) details for specimen LC3

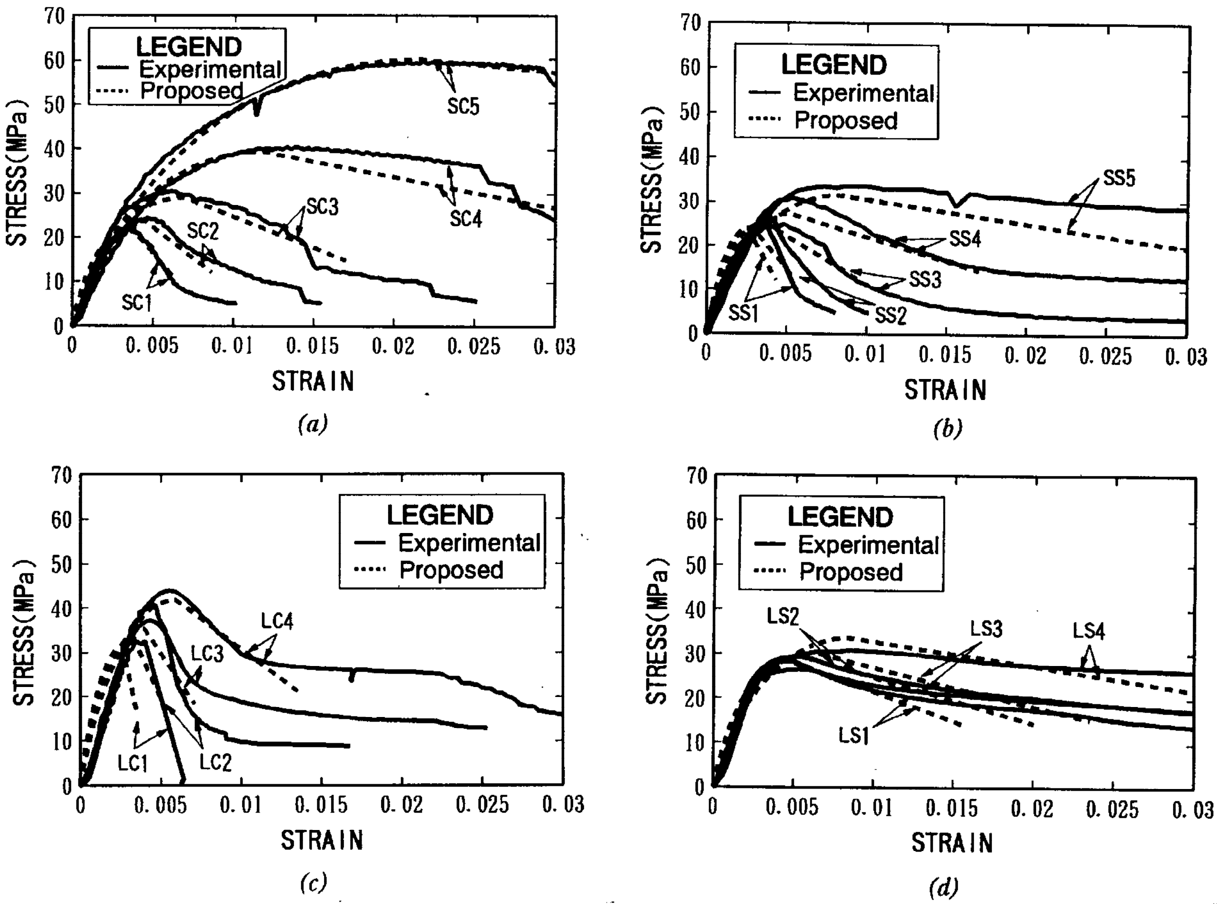

Fig. 3 Comparison of stress–strain relation from test results, and from predictions by (7) and (9): (a) SC Series; (b) SS Series; (c) LC Series; and (d) LS Series

Figure 2 shows the experimental setup in a 30MN capacity universal testing machine at the Public Works Research Institute. All specimens were subjected to uniaxial compressive loading under displacement control, at a rate of 1mm/min. The axial deformation of the specimen, between the top and bottom load heads, was measured by two linear potentiometers placed on opposite sides of the specimen. The longitudinal strain was calculated by dividing the measured axial deformation by the total height of the specimen. Because failure of the concrete did not occur over the whole height of the specimen, but only over part of the height, the question is raised as to what length should be assumed in calculating the longitudinal strain. One option is to use the height over which the failure of the concrete occurs. However, measurement of this height may be made only after the failure of the concrete is initiated. On the other hand, before concrete failure occurs, the total height contributes to the deformation. Thus, it is difficult to evaluate the appropriate total height for whole loading sequence. Based on these considerations, it was decided to use the total height of the specimen to evaluate longitudinal strain over the entire loading sequence.

OBSERVED STRESS–STRAIN RELATION

Figure 3 shows stress–strain curves for circular and square cross sections with various volumetric ratios of hoop reinforcement. A predicted relation, which will be described later, is also presented for comparison in Fig. 3. It should be noted that the initial stiffness is independent of hoop reinforcement ratio. As the volumetric ratio of hoop reinforcement increases, both the peak stress and the strain at the peak stress increase, and severe deterioration of concrete after the peak stress is prevented. It is also clear that the confinement effect is highly dependent on the cross sectional shape.

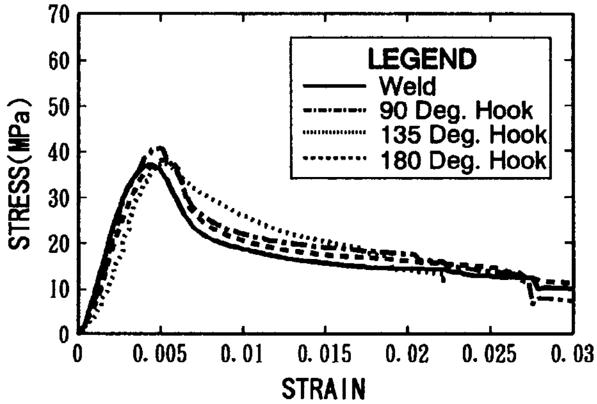

Fig. 4 Effect of anchorage type of hoop reinforcement

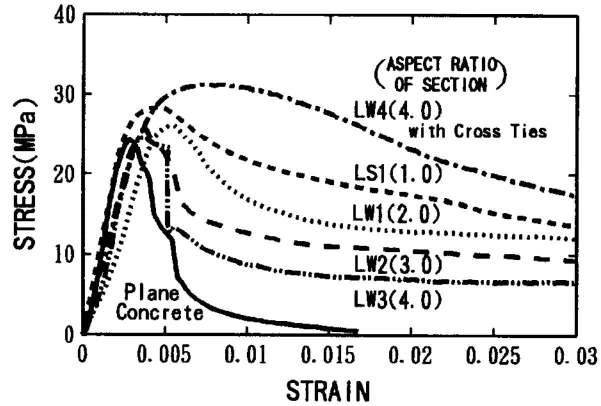

Fig. 5 Effect of aspect ratio of section

Figure 4 shows the effect of the confining reinforcement hook configuration on the stress–strain curve. Similar stress–strain curves were observed regardless of the anchorage type. In all specimens, failure of the hoop reinforcement did not occur near the hook. It may be said, therefore, that for the three types of hooks in circular cross section the stress–strain behavior was insensitive to hook configuration.

Figure 5 shows the effect of aspect ratio of the cross section for 5 specimens with the same volumetric ratio of hoop reinforcement of 1.75%. It can be seen that when a constant amount of hoop reinforcement is provided, the peak stress and the ductility deteriorate as the aspect ratio of the section increases. The stress–strain relation of Specimen LW3 is quite close to that of unconfined concrete. It is apparent that confinement can not be effectively achieved using rectangular hoop reinforcement in a section with an aspect ratio of the section of 4.0. On the other hand, the stress–strain relation of Specimen LW4, with cross ties, shows significantly improved performance over Specimen LW3. Therefore, the effect of cross ties is significant in increasing the ductility of concrete piers with wall–type cross sections.

MODELING OF STRESS–STRAIN RELATION

It can be seen in Fig. 3 that the stress–strain curve consists of three parts, i.e., an ascending branch, falling branch and sustaining branch. In most of the stress–strain models proposed previously (ex. Kent et al., 1971), the ascending branch has been formulated by a second order parabola. This is because a second order parabola is a simple mathematical expression and it represents well the stress–strain relation. However, it should be noted that the second order parabola can reflect only three boundary conditions, even though the stress–strain model for the ascending branch of confined concrete should reflect the following four boundary conditions.

a) initial condition : $f_c = 0$ at $\varepsilon_c = 0$ (1)

b) initial stiffness condition : $df_c / d\varepsilon_c = E_c$ at $\varepsilon_c = 0$ (2)

c) peak condition : $f_c = f_{cc}$ at $\varepsilon_c = \varepsilon_{cc}$ (3)

d) peak stiffness condition : $df_c / d\varepsilon_c = 0$ at $\varepsilon_c = \varepsilon_{cc}$ (4)

where, f_{cc} = peak stress; ε_{cc} = strain at peak stress ; and E_c = initial stiffness.

By disregarding (2), the second order parabola is expressed as

$$f_c = f_{cc}\left\{\frac{2\varepsilon_c}{\varepsilon_{cc}} - \left(\frac{\varepsilon_c}{\varepsilon_{cc}}\right)^2\right\} \tag{5}$$

The initial stiffness $2f_{cc}/\varepsilon_{cc}$ in (5) is an implicit function of the volumetric ratio of hoop reinforcement, since both f_{cc} and ε_{cc} depend on the hoop reinforcement content. However, test results show that the initial stiffness is essentially independent of the volumetric ratio of hoop reinforcement, as illustrated in Fig. 3. To avoid such an inconsistency, Mander *et al.* (1988) adopted a fractional function, which includes the initial stiffness as one of four boundary conditions. Muguruma *et al.* (1980) and Fujii *et al.* (1988) considered (2) as well as (1) and (3) in their second order parabola expressions. However, they needed to use two equations to represent the stress–strain relation of the ascending branch.

To include (2), and avoid having to adopt two equations for the ascending branch, it is proposed here to assume that the stress of concrete is represented by the function

$$f_c = C_1 \varepsilon_c^n + C_2 \varepsilon_c + C_3 \tag{6}$$

in which C_1, C_2, C_3 and n are constants to be determined from (1) to (4). Eq.(6) enables simplification of the stress-strain expression, while at the same time satisfying the four boundary conditions. By substituting (1) through (4) into (6), one obtains

$$f_c = E_c \varepsilon_c \left\{ 1 - \frac{1}{n} \left(\frac{\varepsilon_c}{\varepsilon_{cc}} \right)^{n-1} \right\} \tag{7}$$

where, n is a coefficient, and is given as

$$n = \frac{E_c \varepsilon_{cc}}{E_c \varepsilon_{cc} - f_{cc}} \tag{8}$$

A standard values of E_c, provided by Japanese Specifications (Japan Road Association 1991) is used here. It is assumed E_c is not influenced by confinement.

The falling branch of the stress-strain curve is idealized here by a straight line, as indicated by the test results shown in Fig. 6, and is formulated as

$$f_c = f_{cc} - E_{des} (\varepsilon_c - \varepsilon_{cc}) \tag{9}$$

where, E_{des} = deterioration rate, which is developed from regression analysis of test data in the range of ε_{cc} to ε_{cu}. The definition of ultimate strain, ε_{cu}, is important. In the tests, crushing of core concrete and buckling of longitudinal reinforcement were observed when the compressive stress dropped to less than $0.5 f_{cc}$. Because such damage is excessive and unrepairable, the strain corresponding to 50% of the peak stress f_{cc} is assumed as the ultimate strain ε_{cu}. By substituting $f_c = 0.5 f_{cc}$ into (9), the ultimate strain ε_{cu} is obtained as

$$\varepsilon_{cu} = \varepsilon_{cc} + \frac{f_{cc}}{2E_{des}} \tag{10}$$

Kent and Park, Sheikh et al., Saatcioglu et al., and Fujii et al. considered the sustained stress after the falling branch to be 20% or 30% of the peak stress. In the proposed model, a special effort was not made to develop a formulation of the sustaining branch, since that region is not critical in the seismic design of bridges.

EVALUATION OF CONFINEMENT EFFECTIVENESS

Parameters for Confinement Effect

In the proposed model, expressed by (7) and (9), factors for controlling the stress-strain relation of confined concrete are the peak stress, the strain at the peak stress and the deterioration rate. The effect of confinement on these three parameters is determined based on the test results, as described below.
Figures 7 to 9 show the effet of the confinement on above three parameters. Results of the statistical approximation are also presented in those figures. The test results indicate that the confinement effectiveness for circular and square sections may be represented as

$$\frac{f_{cc}}{f_{co}} = 1.0 + 3.8 \alpha \frac{\rho_s f_{yh}}{f_{co}} \tag{11}$$

$$\varepsilon_{cc} = 0.002 + 0.033 \beta \frac{\rho_s f_{yh}}{f_{co}} \tag{12}$$

$$E_{des} = 11.2 \frac{f_{co}^2}{\rho_s f_{yh}} \tag{13}$$

in which α and β are modification factors depending on cross sectional shape: for circular sections α =1.0 and β =1.0; for square sections α =0.2 and β =0.4.

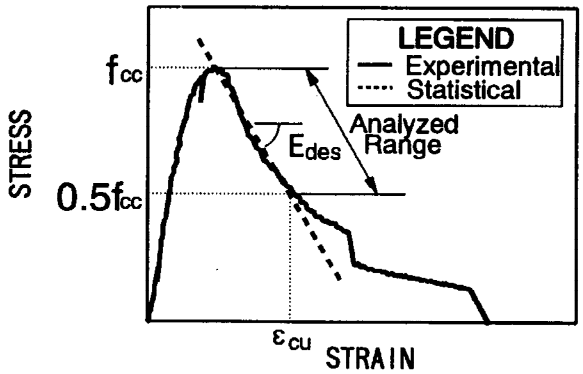

Fig. 6 Definition of ultimate strain and deterioration rate

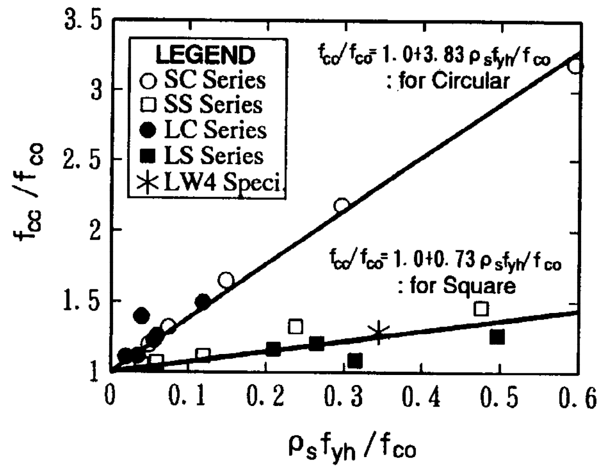

Fig. 7 Effect of confinement on peak stress

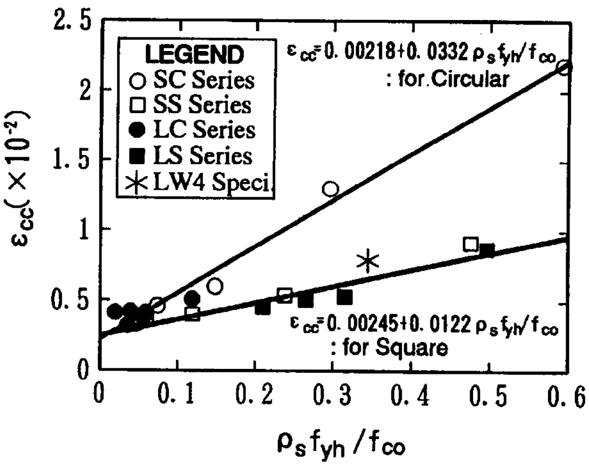

Fig. 8 Effect of confinement on strain at peak stress

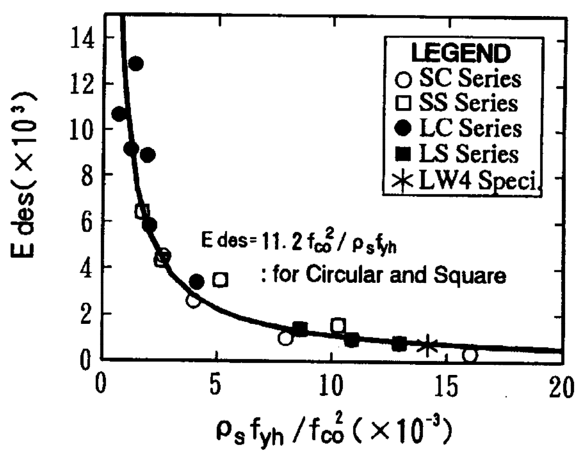

Fig. 9 Effect of confinement of deterioration rate

Estimation of Stress–Strain Relation

Based on (11) to (13), the stress–strain relation was computed for the test specimens shown in Table 1. Analytical results are compared with the test data in Fig. 3. The computed stress–strain relations are in good agreement with the test results.

Evaluation of Effect of Cross Ties

To evaluate the confinement effectiveness in wall–type columns with cross ties, Mander *et al.* (1988) reported a theory that accounts for the horizontal arching action of the concrete, which occurs between longitudinal reinforcing bars. In Mander's model, the arches were represented by second order parabolas with an initial tangent slopes of 45°.

To study the confinement effect of cross ties, Specimen LW4 was analyzed. It is assumed herein that the whole section with cross ties can be divided into five confined areas, and that each confined area is equivalent to a concrete column of 25cm x 25cm square cross section as shown in Fig. 10. The volumetric ratio of hoop reinforcement, ρ_s, in the equivalent confined section is thus calculated as 2.85%. Therefore, the relations between the confinement effectiveness and three factors f_{cc}, ε_{cc}, E_{des} are obtained as shown in Figs. 7 to 9. It is apparent that the test data of f_{cc}, ε_{cc}, and E_{des} in Specimen LW4 are very close to the prediction by (11), (12) and (13) for square sections. Therefore, based on the test results, the confinement effect for wall–type sections with cross ties may be simply evaluated using equivalent confined sections, as illustrated in Fig. 10.

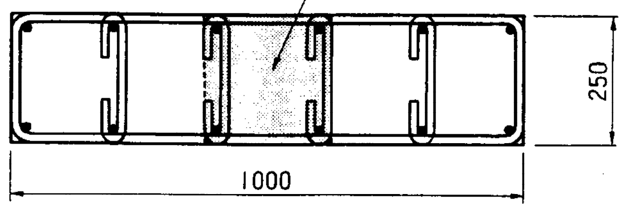

Fig. 10 Equivalent confined section

CONCLUDING REMARKS

To propose a reliable and practical stress–strain model for confined concrete, which includes the effects of low confining reinforcement ratios, a series of loading tests were conducted for 31 concrete column specimens. The conclusions from the study are as follows.

1) The formulation of (7) and (9) is proposed for the stress–strain relation of confined concrete.
2) The ultimate strain ε_{cu} is defined in this study as the strain corresponding to 50% of the peak stress, since knowledge of behavior beyond ε_{cu} is not significant in the seismic design, due to excessive and unrepairable damage.
3) It is proposed to evaluate the peak stress, the strain at the peak stress, and the deterioration rate according to (11) to (13). These expressions were derived based on the test results for specimens with a cross sectional dimension of 20 to 50cm and a volumetric ratio of hoop reinforcement of 0.19 to 4.66%.
4) Cross ties in the wall–type section have a significant effect on increasing ductility. The confinement effectiveness of cross ties can be evaluated by assuming an equivalent confined section, as shown in Fig. 10.

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