STEEL JACKETING FOR IMPROVEMENT OF COLUMN STRENGTH AND DUCTILITY

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SUMMARY

In order to prevent loss of human life and property due to future earthquakes, the steel jacket retrofit has been used as a method to enhance the shear strength and ductility of square reinforced concrete (RC) columns in existing buildings in Japan since the 1995 Hyogoken-Nanbu earthquake. This paper is a state-of-the-art report on the seismic behavior of square RC columns retrofitted by the steel jacket based on many researches conducted in Japan. The emphasis of this paper will be placed on the load carrying and deformation capacities of the retrofitted RC columns. Main items described in this report are. (1) stress-strain curve model for concrete confined by the steel jacket, (2) methods to evaluate ultimate bending strength and shear strength of the retrofitted RC columns, and (3) design formula to predict deformation capacity of the retrofitted columns. The proposed methods and formula are verified by many experimental results of the retrofitted RC column specimens tested by Japanese researchers. It should be emphasized that the characteristic of the proposed methods and formula is their applicability to wide ranges of parameters such as material strength, wall thickness of the steel jacket, aspect ratio of the column, and magnitude of axial load.

INTRODUCTION

The 1994 Northridge earthquake and the 1995 Hyogoken-Nanbu earthquake have caused substantial damages to reinforced concrete bridge piers and building columns. Some of the major problems involved with the damaged reinforced concrete columns can be attributed to the poor transverse detailing, which resulted in inadequate shear strength and flexural ductility of columns. Therefore, in order to prevent loss of human life and property from future earthquakes, methods to enhance the shear strength and ductility of existing reinforced concrete columns have to be developed.

Confining reinforced concrete column in steel jackets is one of the effective methods to improve the earthquake-resistant capacity. As compared with conventional hoops or spirals, steel jacket has two more remarkable advantages; 1) to easily provide a large amount of transverse steel, hence strong confinement to the compressed concrete, and 2) to prevent spalling off of the shell concrete. Spalling of the shell concrete may be considered as the main reason for deterioration of bond and buckling of longitudinal bars of columns and is hardly prevented by conventional hoops. Because of these advantages of the steel jacket, confining method utilizing steel jacket has been increasingly used to retrofit or strengthen the existing reinforced concrete columns without adequate detailing. This method is referred to as the steel jacket retrofit hereafter.

The authors have conducted an integrated theoretical and experimental research on the seismic behavior of steel jacket retrofitted columns since 1985 (Sakino *et al*., 1985). This paper is a state-of-the-art report on the steel

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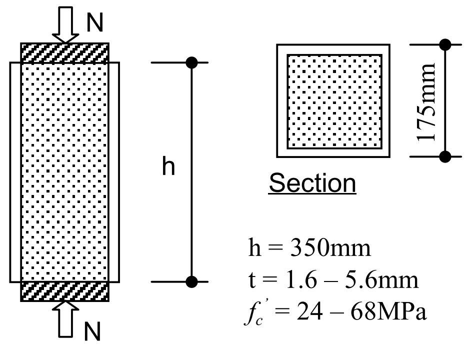


Fig.1: Centrally loading test

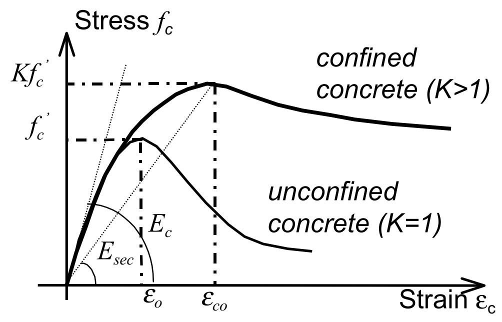


Fig.2: Stress-strain curve model for concrete

jacket retrofit of square RC columns based on many researches conducted in Japan which include author's research.

2. STRESS-STRAIN RELATIONSHIP FOR CONCRETE CONFINED BY STEEL JACKET

Knowledge on the stress-strain behavior of concrete confined by the square steel jacket is very important in accurately assessing the flexural response of the retrofitted columns. Experimental work has been carried out to establish a stress-strain curve model for confined concrete by authors (Sakino and Sun, 1994). The experimental variables were (1) the wall thickness of square steel jackets, expressed in terms of width to wall thickness ratio, B/t, (31, 55, 76, 107) and (2) the specified compressive strength of concrete, f_c', (24, 46, 68MPa). Forty eight specimens with 175mm×175mm cross section were tested. The inside surfaces of steel tubes were coated with a thin grease layer to reduce the bond stresses between the infilled concrete and steel tube. All specimens were loaded under monotonic concentric loading. At both ends of specimens, steel blocks for loading were placed to ensure that the axial load be applied only to the infilled concrete (see Fig. l).

Table 1 Details of stress-strain model for concrete

$$f_c = Kf_c' \frac{aX + (b-1)X^2}{1 + (a-2)X + bX^2}, \quad \left(X = \frac{\varepsilon_c}{\varepsilon_{co}} \right)$$

where

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

$$\rho_t = \left(\frac{B}{B-2t} \right)^2 - 1, \qquad a = \frac{E_c}{E_{\text{sec}}} = \frac{E_c \varepsilon_{co}}{Kf_c'}$$

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

$$b = 1.5 - 0.017 f_c' + 2.4 \sqrt{\frac{(K-1) f_c'}{23}}$$

$$\varepsilon_o = 0.94 \left(f_c' \right)^{1/4} 10^{-3}$$

$$E_c = \left(0.69 + 0.332 \sqrt{f_c'} \right) \times 10^4 \quad \text{(in MPa)}$$

in which

f_c' = strength of concrete cylinder (in MPa)

ρ_t, f_{yt}, t and B = volumetric ratio, yield stress, thickness and outside width of steel jacket.

Based on the test results, the stress-strain model shown in Table 1 has been proposed. The model is also illustrated in Fig. 2. In order to predict the stress-strain relationships of confined concrete by using the proposed model, it is necessary to determine values of four parameters; (1) maximum stress of confined concrete f_{cc}', (2) strain at the maximum stress ε_{co}, (3) parameter *a* which controls the shape of the ascending branch, and (4) parameter *b* which mainly governs the descending portion of the curve. These four parameters can be determined based on f_c', thickness of steel jacket wall, *t*, and yield strength of steel, f_{yt}, as shown in Table 1. The formula to obtain the most important parameter $K = f_{cc}'/f_c'$ was verified by experimental results of 39 specimens tested by other researchers in Japan. Fig. 3 shows typical examples of comparisons between experimental results and proposed stress-strain curve models.

3. DESDIGN FORMULAE FOR RETROFITTED COLUMNS UNDER COMBINED COMPRESSION, BENDING AND SHEAR

Data Base: In order to establish an aseismic design method for steel jacketing for improvement of existing RC columns, it is necessary to investigate behavior of the retrofitted RC columns subjected to combined forces and deformed in a double curvature as pattern shown in Fig. 4. The maximum horizontal force, V_{max}, and the limit rotation angle, R_u, which are defined in Fig. 5 as indices of load-carrying and deformation capacities of columns, are discussed based on experimental results obtained by authors and other Japanese researchers. A total of 80 test data (test specimens) were collected from the literatures listed in Table 2. Fig. 6 shows ranges of the test parameters and failure modes of these test specimens.

Ultimate Bending Strength: The ultimate bending strength of retrofitted RC columns for a given axial load can be estimated by the computing procedure for the moment-curvature analysis. However, this procedure is complicated. In this section, a simple method of estimating the ultimate bending (ultimate moment) strength is proposed. This simplification is made by replacing the actual stress distribution of concrete in compressive zone with an equivalent rectangular stress block as shown in Fig. 7. The specific concrete extreme fiber strain, ε_{cm}, shown in Fig. 7 is the strain corresponding to the maximum moment on the moment-curvature relationships. The formulas to obtain values of the ε_{cm} and the stress block parameters α, β shown in Fig. 7 can be formulated based on the stress-strain curve for confined concrete described in the preceding section, and are given in Table 3. The ultimate moment M_u can be calculated based on the proposed stress block by the computing procedure similar to that for ordinary RC columns.

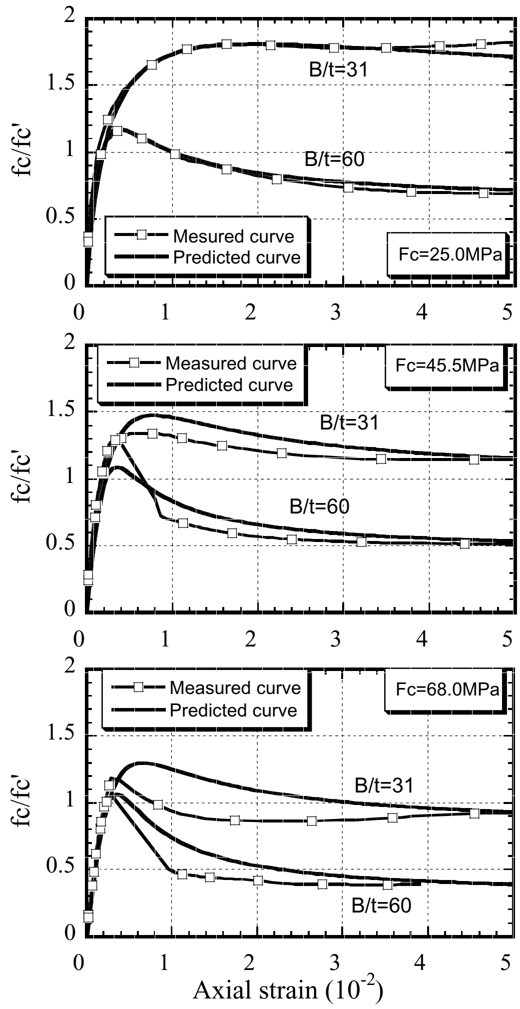


Fig. 3: Comparison between the experimental result and the proposed axial model

It is well known that both end sections of the reinforced concrete columns under moment gradient as shown in Fig. 4 can develop larger ultimate moment than theoretically predicted ultimate moment. The main reason for this phenomenon is that the additional confinement provided by the stiff loading stub adjacent to the end section would shift the critical section away from the end section to a section carrying smaller moment. The other reason can be attributed to a strain hardening effect of the longitudinal bars. In order to evaluate the effect of the extra confinement from the stiff loading stub, authors have proposed an empirical formula given by Eq.(1) based

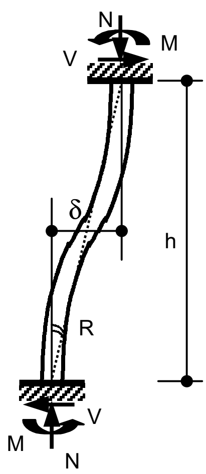


Fig. 4: Columns under combined forces

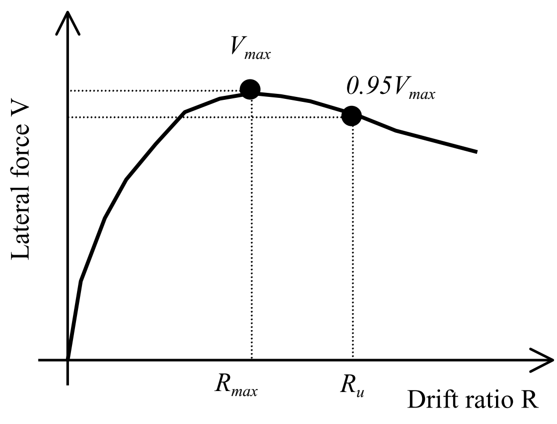


Fig. 5: Definition of limit rotation angle R_u

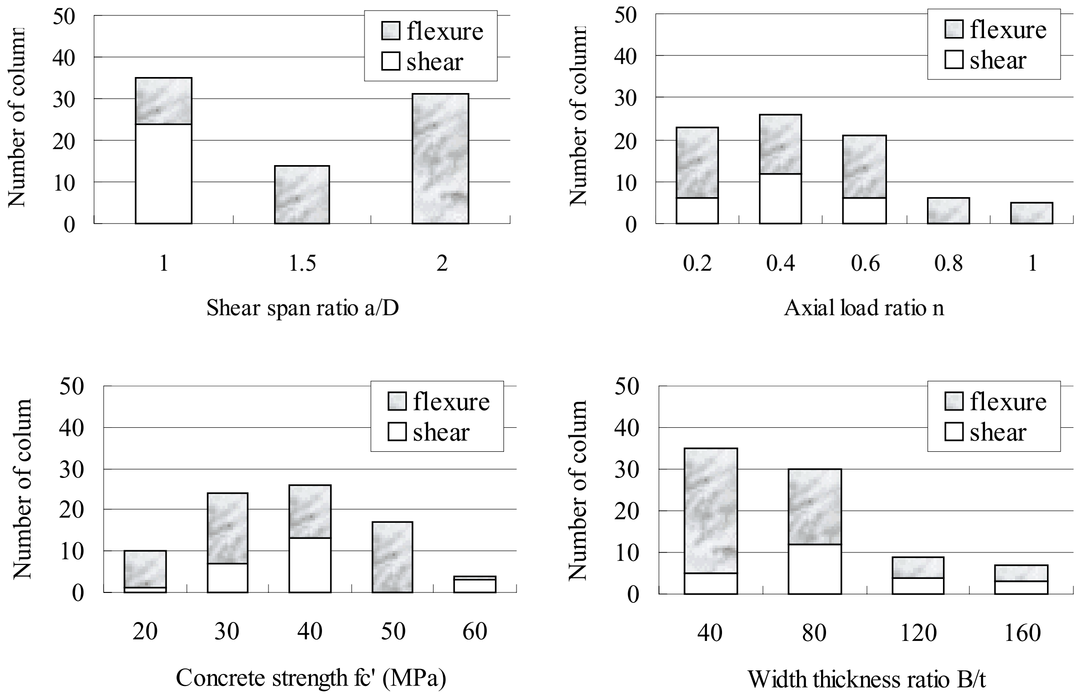


Fig. 6: Range of experimental variables and failure mode of retrofitted RC columns

Table 2 List of available test data concerning retrofitted RC columns

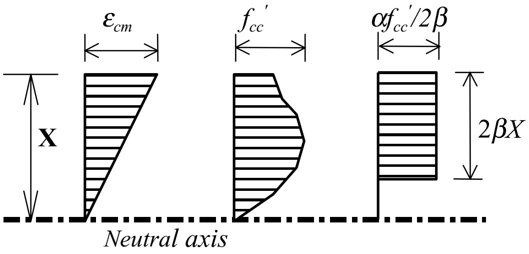
Ref.	Number of column	D (mm)	a/D	B/t	p_g (%)	f_c' (MPa)
Sasaki 1972	2	390	1.38, 2.28	172	3.0	21.8
Tomii. 1987	6	160	1.07	29	3.82, 7.65	32.2～42.3
Minami. 1988	3	196	1.53	50	2.64	19.9～22.9
Sun *et al*., 1989	4	165	1.06	77	3.71	40.2～47.8
Sasaki. 1989	7	195	1.02～1.53	48～135	2.65	21.9～31.9
Yoshioka , 1989	6	300	2.0	55～72	4.47～5.36	46.1
Yamamoto . 1990	1	294	1.53	98	2.77	28.4
Asakawa. 1994	9	180	1.25	42～152	7.06	37.4～42.0
Yoshikawa. 1995	5	300	0.9～1.8	52～69	1.69～2.26	48.4～55.2
Jinno., 1996	2	316-356	1.11, 2.11	101,103	1.60, 2.03	36.7
Masuo, 1996	2	460	1.30	146	0.75, 2.16	22.5
Sun *et al*., 1997	7	163	2.18	29	3.82～7.65	39.6～56.6
Sakino. 1997	8	250	2.0	28～117	2.44	32.0
Aklan. 1997	5	250	2.0	28～117	2.44	47.9～51.1
Sakino. 1998	6	250	1.0～1.5	43～82	2.44	29.4～34.9
Sun *et al*. 1998	7	250	1.0	28～117	2.44	27.8～37.2

p_g= ratio of total area of the longitudinal bars to gross sectional area of the concrete

on the experimental results of 148 reinforced concrete square columns confined by conventional hoops (Sun *et al*., 1996).

$$\frac{M_{ue}}{M_u} = \begin{cases} 1.1, & n \le 0.3 \\ 1.1 + 0.8(n-0.3)^2, & n > 0.3 \end{cases} \tag{1}$$

where M_{ue} is the column end ultimate moment, M_u is the theoretical ultimate moment; and n is the axial load



(a) strain profile (b) actual stress distribution (c) equivalent stress block

Fig. 7: Idealization of the stress block

Table 3 Parameters defining the stress block for the compressed confined concrete

$$\frac{\varepsilon_{cm}}{\varepsilon_{co}} = 1.375 + 0.108K - \frac{0.1}{K^4}\frac{f_c^{'}}{42}$$

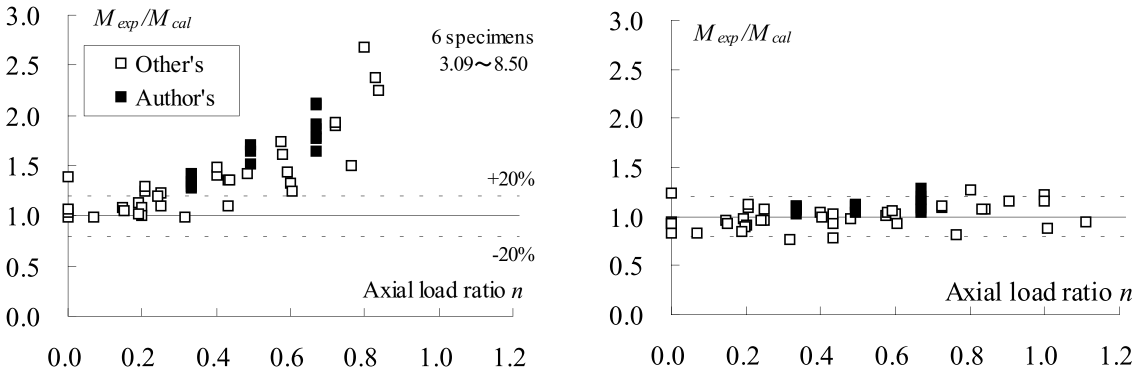
$$\alpha = 0.724 + 0.107K - \frac{0.037}{K - 0.007}\frac{f_c^{'}}{42}$$

$$\beta = 0.383 + 0.046K - \frac{0.019}{K + 0.687}\frac{f_c^{'}}{42}$$

where

$f_c^{'}$ = the strength of concrete cylinder (in MPa)

K = the strength enhancement ratio (See Table 1)



(a) ACI Code method (b) Proposed method

Fig. 8: Comparisons between the experimental and the theoretical flexural strength

ratio given by $N/(A_c f_c^{'})$.

The authors have also proposed that Eq.(1) can be applied to both the steel jacket retrofitted columns as well as the ordinary reinforced concrete columns (Sun and Sakino *et al.*, 1998). As can be seen from Eq.(1), the enhancement of column end moment due to the extra confinement becomes more remarkable as the axial load ratio becomes larger. The comparisons between the experimental ultimate moments, M_{exp}, and theoretical ones calculated by the ACI code method and the proposed method are shown in Fig. 8. The experimental ultimate moments are defined as the column end moments at the maximum shear, and include a secondary moment due to axial load and lateral displacement so called as N-Δ moment . The theoretical ultimate moment given by the proposed method are calculated by using stress block shown in Fig. 7, and modified by Eq.(1). As observed in Fig. 8, the ACI code method might give too conservative prediction, because the confinement effects of the steel jacket and the stiff loading stub are both ignored. On the other hand, the proposed method can predict the experimental results very well. It is recommended, however, that the extra confinement effect given by Eq.(1) be ignored provided that the N-Δ effect is not considered in design.

Shear Strength: The 1995 Hyogoken-Nanbu earthquake have inflicted severe damages on the RC columns designed according to the insufficient building codes of the 1950's and 1960's. The columns contained widely spaced transverse reinforcement that resulted in insufficient confinement and shear strength. The shear failure of these columns was a direct consequence of the inadequate lateral reinforcement. Since the steel jacket has been proved to be an effective method to retrofit such columns, it is very important to quantitatively evaluate the effectiveness of the steel jacket as shear reinforcement. In order to evaluate the shear strength of ordinary RC columns, tremendous work has been done in Japan. Several equations have been proposed to evaluate the shear

Table 4 Details of the modified Ohno-Arakawa's shear strength formula

$$V_s = \left\{0.115 k_u \cdot k_p \frac{17.65 + f_c'}{(a/D) + 0.12} + 0.85\sqrt{p_w f_{yt}} + 0.1\frac{N}{bD}\right\} bj, \quad \left(j = \frac{7}{8}d\right)$$

where

$$k_u = \begin{cases} 1.0, & d \le 160mm \\ 1.19 - 0.0012d, & 160mm < d \le 400mm \\ 0.72, & d > 400mm \end{cases}$$

$$k_p = 0.82(100p_t)^{0.23}$$

Notations:

k_u =	coefficient taking account for the scale effect
p_w =	reinforcement ratio of the transverse steel (= $2t/B$)
b, D =	overall width and depth of the concrete section (in mm)
a/D =	shear span ratio of column
N =	axial load applied (in kN)
d =	effective depth of the section (in mm)
p_t =	reinforcement ratio of the tension steel bars
f_{yt}, B, t =	see Table 1

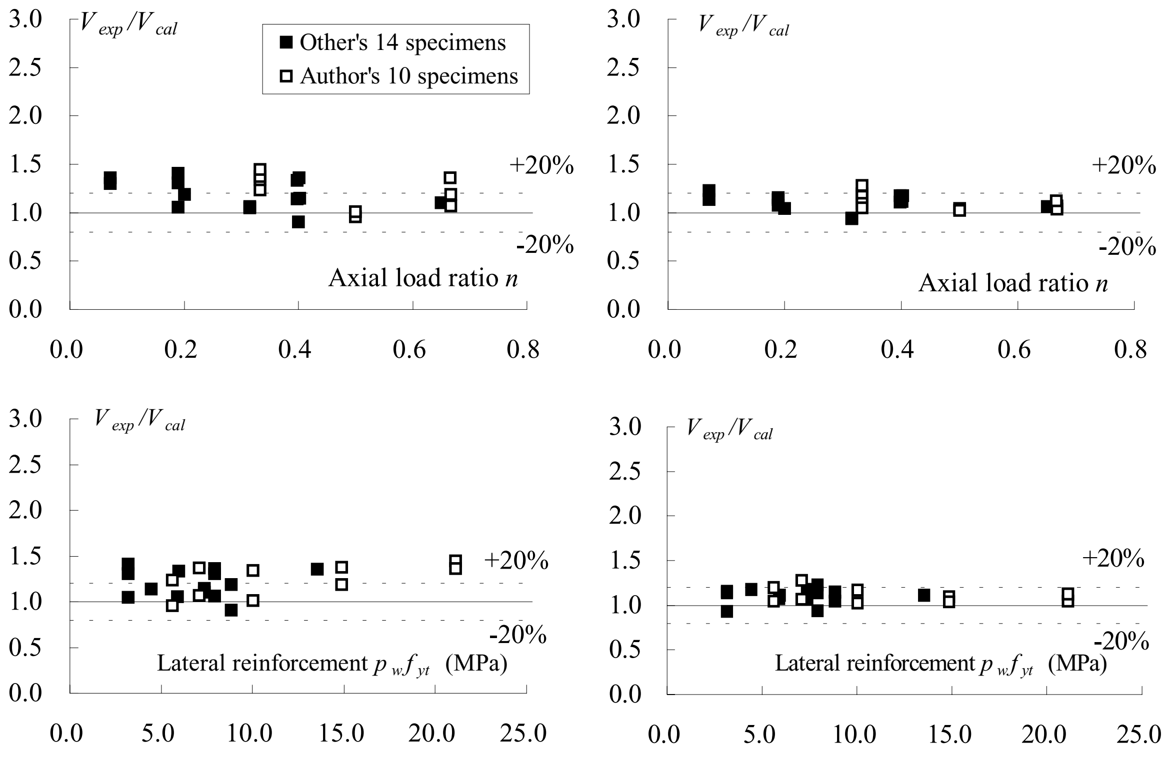


Fig. 9: Comparisons between the experimental and the theoretical shear strength

Strength, some of which have been kept being modified consistently to accommodate new findings arising from the introduction of new material or the range of application of such equations. In this paper, Modified Ohno-Arakawa's shear equation (referred to as the Arakawa's equation) detailed in Table 4 is studied and compared from the view point of their capability of predicting the shear strength of the steel jacket retrofitted RC columns

Table 5: Formula for the limit rotation angle R_u

$$R_u = \varphi \cdot \alpha \cdot n^{-\beta} (B/t)^{-\gamma}$$

where

$$\varphi = \begin{cases} 1.0, & f_c' \leq 36MPa \\ f_c'/36, & f_c' > 36MPa \end{cases}$$

$$\alpha = \begin{cases} 0.18(a/D) + 0.06, & a/D \leq 1.5 \\ 0.36 - 0.02(a/D), & a/D > 1.5 \end{cases}$$

$$\beta = 1.1$$

$$\gamma = 0.8$$

Notations:

n = axial load ratio = $N/(bDf_c')$

see Table 4 for the meanings of the other terms

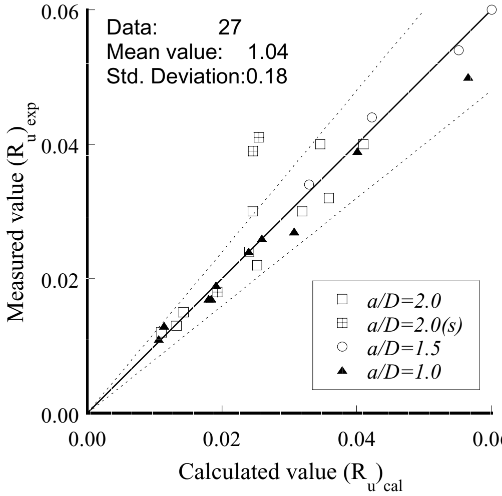


Fig. 10: Comparison of the limit rotation angles

that failed in shear. The Arakawa's equation is the most familiar empirical equation derived through a lot of experimental results and is now widely used for practical design in Japan for the evaluation of shear strength of RC beams and columns in existing buildings as well as new designed buildings. One of the main merits of this equation might be its simplicity and its incorporation of the main variables in a clear manner including the effect of the axial load. The comparisons between the experimental shear strength, V_{exp}, and the calculated ones, V_{cal}, given by the ACI shear equation and the Arakawa's equation are shown in Fig.9. The steel jackets are replaced with the closely spaced conventional hoops to obtain the calculated shear strength. As observed in Fig. 9, the ACI shear equation gives conservative prediction especially for the columns with large amount of lateral reinforcement (thick steel jacket wall). The reason for this is that the ACI shear equation specified limit on the contribution of lateral reinforcement. The conservative manner of the ACI shear equation shown in Fig. 9 can be attributed also to the other limit imposed on the final outcome of the concrete component. The Arakawa's equation also predicts the shear strength in slightly conservative manner. It can be said that the Arakawa's equation products the shear strength of the retrofitted columns more precisely than the ACI shear equation does.

Limit Rotation Angle: The limit rotation angle, R_u, of the retrofitted RC columns discussed in this section is a characteristic point on the envelope curve of hysteresis loops of shear force versus rotation angle relationship. The R_u is defined as the rotation angle at which 95% of the maximum shear force is maintained after reaching the peak load as shown in Fig. 5. The design formula for R_u given in Table 5 has been established by regression analysis on the experimental results obtained by authors. The comparisons between experimental limit rotation angles and the predicted ones are shown in Fig. 10. As observed in Fig. 10, it is possible to assure a limit rotation angle larger than 0.02 by the steel jacket retrofit method even for the short columns with *a/D* of 1.0. It is also noteworthy that though all the short columns with *a/D* of 1.0 failed in shear, some of them exhibited enough deformation capacity.

4. CONCLUSIONS

The design formulae for the square reinforced concrete columns retrofitted by the steel jacket are proposed based on the experimental results obtained by authors and other Japanese researchers. The proposed design formulae have following features.

(1) The ultimate bending strength of the retrofitted RC column sections can be accurately evaluated by the simple method using the ultimate strain, ε_{cm}, and the stress block parameters, α and β, shown in Fig. 7. The formulae to obtain values of the ε_{cm}, α and β have been developed based on the proposed stress-strain curve of concrete confined by the steel jackets.

(2) The shear strength of the retrofitted RC columns can be evaluated in a slightly conservative manner by the Arakawa's equation, an empirical equation derived through a lot of experimental results and widely used for the practical design of the ordinary RC column in Japan.

(3) The design formula for evaluating the limit rotation angle, R_u, defined in Fig. 5 and detailed in Table 5, predicted the experimental deformation capacity of the retrofitted RC columns with a reasonable accuracy.

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