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Experiments on the shear strength of ultra-high strength reinforced concrete columns

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ABSTRACT : Recently in Japan, to realize construction of super high-rise and lightweight reinforced concrete buildings, the synthetic research project on ultra-high strength reinforced concrete structures has started under the leadership of the Ministry of Construction. A study on the shear strength of ultra-high strength reinforced concrete columns was conducted by the authors as a link in the chain of the project. Six column specimens using high strength concrete of 120 MPa grade and high strength longitudinal reinforcing bars of 700 MPa grade were tested. The variables investigated were the quantity and yield stress of transverse reinforcement and the level of axial load applied. The test results are described in this paper. The ultimate shear strength for each column calculated using the shear design equation in the AIJ design guidelines is compared with the test results, and the applicability of the design equations for ultra-high strength reinforced columns is also discussed.

1 INTRODUCTION

Recently in Japan, comprehensive experimental and analytical investigations have started in order to develop design and construction procedures for ultra-high strength reinforced concrete buildings. A study on the shear strength of ultra-high strength reinforced concrete columns was conducted by the authors as a link in the chain of their investigations. Six column specimens using high strength concrete of 120 MPa grade and high strength longitudinal reinforcing bars of 700 MPa grade were tested. The variables investigated were the quantity and yield stress of transverse reinforcement and the levels of axial load applied.

This paper presents the test results. The following items are also examined.

1) Applicability of the shear design equation in the AIJ design guidelines, Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Ultimate Strength Concept (1990), for reinforced concrete columns using high strength concrete of σ_B=120 MPa grade.

2) Adequacy of adopting the transverse reinforcing stress, $p_w\sigma_{wy}$, as an index of the estimation of transverse reinforcement for reinforced concrete columns using high strength concrete and transverse reinforcement.

2 TEST PROGRAM AND SPECIMENS

Six specimens of about one-third scale were prepared, representing an inner column in the first story of a high-rise building. The dimensions and details of the specimens are shown in Fig.1. All columns had a section that was 300 mm square and 900 mm height. The longitudinal reinforcement in each column consisted of twelve 19 mm diam SD70 grade (σ_y=736 MPa) deformed bars arranged symmetrically around the perimeter to give a total reinforcement ratio, p_g=3.83%. All columns had three overlapping transverse reinforcing bars (hoops) per set : one square peripheral hoop and two rectangular overlapping interior hoops. The spacing of transverse hoop reinforcement was 80 mm. The arrangements of hoop steel are considered to be

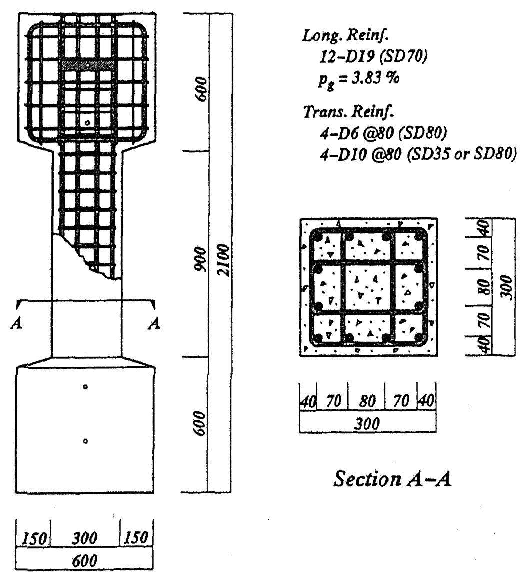

Fig. 1 Typical details of test specimens

logical arrangements for twelve longitudinal reinforcing bars as shown in Fig.1. And then individual hoop had a welding connection in the middle of the longer span.

The test program is shown in Table 1. Mechanical properties of the materials are also listed in Table 2. The main variables investigated were the quantity and yield stress of transverse reinforcement and the level of axial load applied. The transverse reinforcing bars used were 10 mm diam SD35 grade (σ_{wy}=407 MPa) for specimens C61 and C31, 6 mm diam SD80 grade (σ_{wy}=735 MPa) for specimens C62 and C32, and 10 mm diam SD80 grade (σ_{wy}=766 MPa) for specimens C63 and C33, respectively. The transverse reinforcing stress level as expressed by $p_w\sigma_{wy}$ was 4.85 MPa for specimens C61 and C31, 3.92 MPa for specimens C62 and C32, and 9.13 MPa for specimens C63 and C33, respectively. The applied axial load level as expressed by the ratio σ_o/σ_B was 1/6 for specimens C61, C62 and C63, and 1/3 for specimens C31, C32 and C33, respectively.

The concrete compressive cylinder strength at the stage of testing the specimens was 113.8 MPa. The concrete mixing with silica cement was normal weight, with a water cement ratio of 20% and a maximum aggregate size of 13 mm.

The test specimens were loaded by axial compression on the centrally loaded column and by repeatedly applying anti-symmetric bending moments of equal magnitude at both ends with controlled horizontal displacement (1987). The controlled horizontal displacements were the displacements at which the diagonal tension cracking occurred and corresponding to the story drift angles, R of 0.01 and -0.01 rad., respectively. The story drift angle, R is the angle formed by horizontal displacement between ends of the column, 900 mm.

3 TEST RESULTS

3.1 Load-displacement behavior

Applied shear force versus story drift responses for each specimen are presented in Fig.2. The sloping lines included in each figure indicate the theoretical flexural strength calculated using the concrete S-S model proposed by Fafitis and Shah (1985) (shaded line), the theoretical shear strengths calculated using the A and B methods of the AIJ design guidelines mentioned later (solid and chain line), and the P-Δ effect (dotted line), respectively. The white and black triangles on the envelope curves indicate the points at which the diagonal tension cracking occurred and the maximum shear capacity of the column was attained, respectively. The measured strengths and theoretical predictions of each specimen are listed in Table 3.

In all specimens, the maximum shear capacity was attained after some cracks occurred in the order of flexural cracking and diagonal tension cracking in the both top and bottom one-third regions of the column. After that, shear cracks expanded to the whole of the column. The typical shear failure mode was observed in specimens C62, C31 and C32. The flexural-shear failure mode was also observed in specimens C61, C63 and C33. Little differences of the diagonal tension cracking load and displacement were shown between the specimens subjected to the same axial load. The first diagonal tension cracking occurred at the story drift angle of about 0.006 rad. with the load of about 500 kN in specimens C61, C62 and C63. In specimens C31, C32 and C33, on the other hand, it occurred at about 0.007 rad. with about 750 kN.

After the diagonal tension cracking, behavior of the columns was considerably affected by the level of axial load applied and the quantity and yield stress of transverse reinforcing bars. The maximum shear capacities of the columns were attained at the story drift angle of about 0.025 rad. for specimens C61, C62 and C63, which were the applied axial load level, σ_o/σ_B of 1/6. On the other hand, for specimens C31, C32 and C33 subjected to higher axial load, σ_o/σ_B of 1/3, the maximum shear capacities were attained at the angles of 0.013, 0.016 and 0.025 rad., respectively. From the comparison between specimens C61 and C62 or specimens C31 and C32, it was indicated that the columns using the normal strength transverse reinforcement had higher capacity and stiffness after the cracking than the columns using the high strength one, if the level of transverse reinforcing stress, $p_w\sigma_{wy}$ was almost the same. It can be considered as the reasons that the shear transferring capacity between cracks in the columns using the high strength transverse reinforcement becomes less than that in the columns using the normal strength one, because the width of cracks is more expansive. Specimens C63 and C33 which were the level of transverse reinforcing stress of 9.13 MPa showed relatively ductile behavior without large deterioration in load carrying capacity after the attainment of the maximum capacity with increase of the story

Table 1 Summary of test program

Specimen	Axial stress		Transverse Reinforcement		
	σ_o (MPa)	σ_o/σ_B	p_w (%)	σ_{wy} (MPa)	$p_w\sigma_{wy}$ (MPa)
C61	18.96	1/6	1.19	407	4.85
C62			0.53	735	3.92
C63			1.19	766	9.13
C31	37.92	1/3	1.19	407	4.85
C32			0.53	735	3.92
C33			1.19	766	9.13

Table 2 Material properties (unit : MPa)

Concrete	Compressive Strength : σ_B		113.8
	Tensile Strength : σ_t		4.8
Long. Reinf.	Yield Stress : σ_y	D19	736
Trans. Reinf.	Yield Stress : σ_{wy}	D10	407
		D8	735
		D10	766

Table 3 Experimental results and comparison with predictions for each specimens

Specimen	Diagonal Tension Cracking				Maximum Shear Strength						
	Meas.		Theo.	Meas./Theo.	Meas.		Theo.* (kN)			Meas./min(V_f, V_s)	
	(kN)	[(rad.)]	(kN)		(kN)	[(rad.)]	V_f	V_{SA}	V_{SB}	A Method	B Method
C61	498.2	[0.0064]	642.4	0.776	932.6	[0.0249]	961.1	619.8	1013.0	1.505 [S]	0.970 [F]
C62	531.5	[0.0061]		0.827	797.3	[0.0226]		557.0	977.7	1.431 [S]	0.830 [F]
C63	490.3	[0.0050]		0.763	912.0	[0.0256]		873.8	1173.9	1.044 [S]	0.949 [F]
C31	804.1	[0.0074]	861.2	0.934	942.4	[0.0131]	1049.3	619.8	1013.0	1.520 [S]	0.930 [S]
C32	738.4	[0.0073]		0.857	839.4	[0.0162]		557.0	977.7	1.507 [S]	0.859 [S]
C33	774.7	[0.0070]		0.900	1019.9	[0.0249]		873.8	1173.9	1.167 [S]	0.972 [F]

* V_f : Flexural Eq., V_{SA} : A Method, V_{SB} : B Method

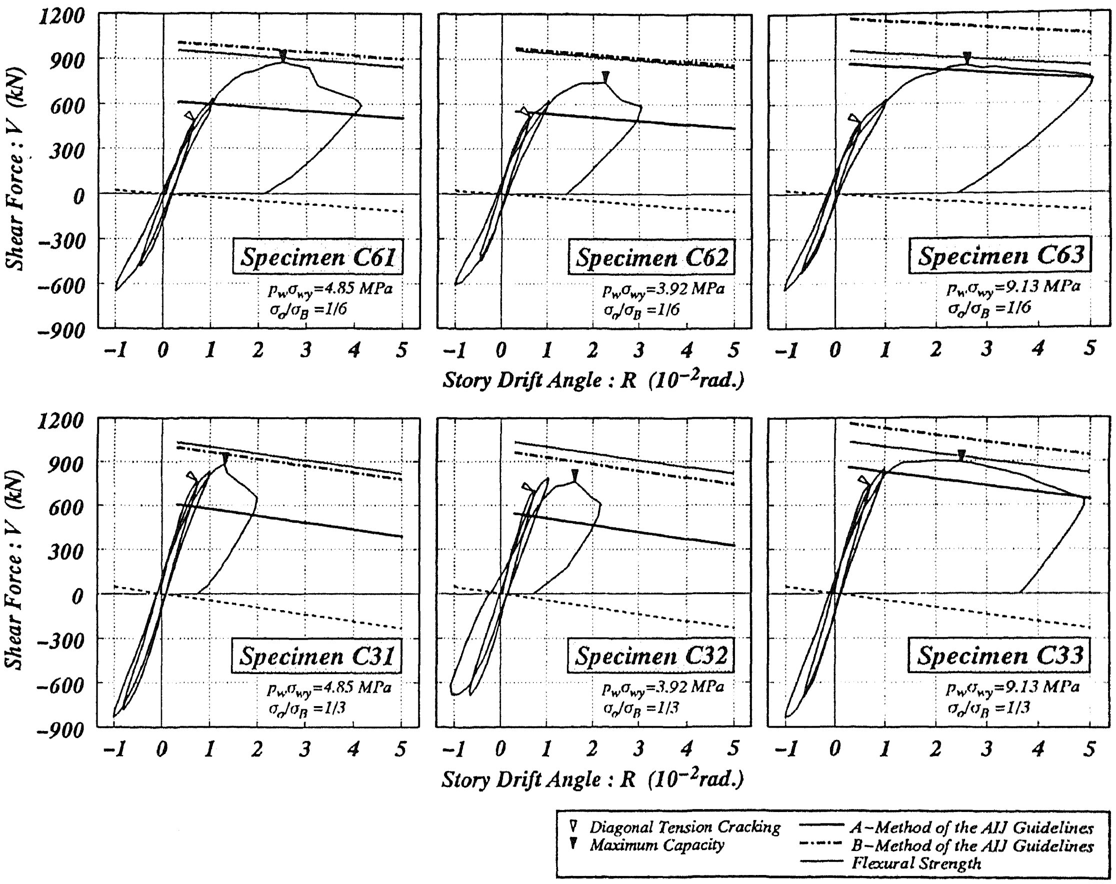

Fig. 2 Applied shear-story drift responses of columns

drift angle. Then in their specimens, crushing and spalling of the shell concrete occurred finally in the both top and bottom of the column.

In specimens C61, C63 and C33, the maximum shear capacity did not slightly exceed the theoretical flexural strength calculated although yielding of longitudinal reinforcing bars was observed in the top or bottom of the column. The maximum shear capacity of all columns was also between the theoretical strengths calculated using the A and B methods of the AIJ design guidelines.

3.2 Stress distributions of transverse reinforcement

Fig.3 shows the stress distributions of transverse reinforcement in each specimen calculated from strains measured electrical resistance gauges. The circles, triangles and quadrangles in each figure indicate the

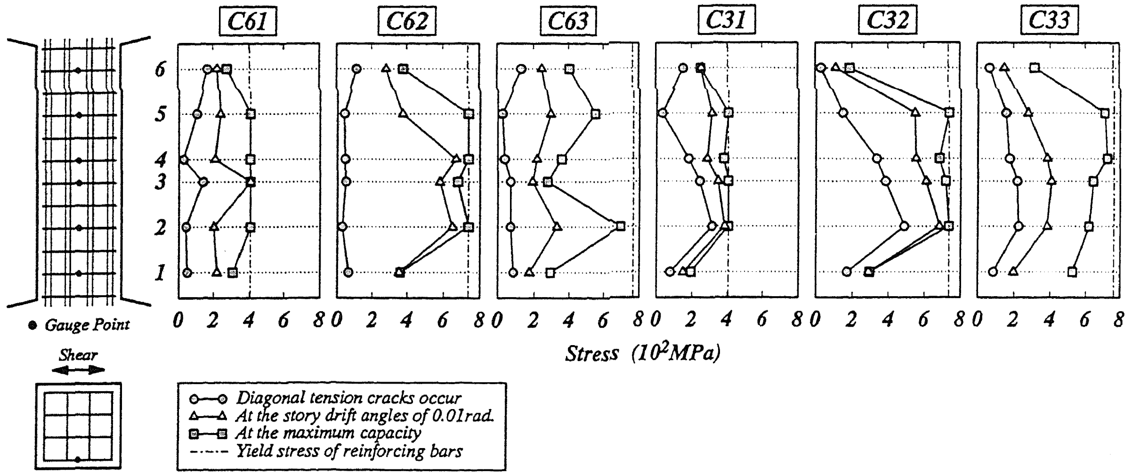

Fig. 3 Stress distributions of transverse reinforcement of columns

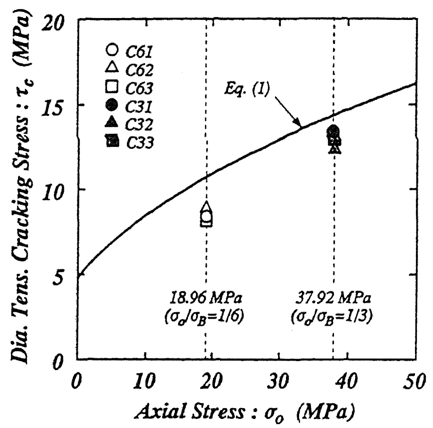

Fig. 4 Comparison of calculated and observed daiagonal tension cracking load

stresses at the occurrence of diagonal tension cracking, the story drift angle of 0.01 rad. and the attainment of maximum shear capacity. The chain lines also express the yield stress of transverse reinforcing bars.

Little differences in the stress distributions of the transverse reinforcement at the diagonal tension cracking were shown when the columns were subjected to the same level of axial load applied. However, at the story drift angle of 0.01 rad., the stress levels of individual hoop in specimens C62 and C32 using the high strength bars were respectively about two times those in specimens C61 and C31 using the normal ones, although the tensile force contributed per a hoop was nearly equal. It was indicated that the strain level of the transverse reinforcement after the cracking was much different between the columns using the high and normal strength bars even if the transverse reinforcing level as expressed by $p_w\sigma_{wy}$ was the same. These results correspond to the above mentioned results that the shear transferring capacity between cracks has reduced in the columns using the higher strength transverse reinforcement because of the larger cracking width.

For specimens C61, C62, C31 and C32, yielding of the transverse reinforcement was observed at the attainment of maximum capacity, disregarding the difference of the yield stress. On the other hand, for specimens C63 and C33 which are observed yielding of the longitudinal reinforcing bars, the transverse reinforcing bars were not yielded at the attainment of maximum capacity. Namely, these results indicated that the applied shear was effectively contributed by the transverse reinforcement for the specimens failing in shear.

4 COMPARISON OF TEST RESULTS WITH ANALYTICAL PREDICTIONS

4.1 Diagonal tension cracking load

In Fig.4, the diagonal tension cracking stresses for each specimen are plotted and compared with the theoretical predictions calculated using the elasticity as expressed by the solid line. The experimental and theoretical values for each specimen are already listed in Table 3. The theoretical diagonal tension cracking stress, τ_c of the columns can be obtained from the well-known relationship

$$\tau_c = \sqrt{\sigma_o \cdot \sigma_t + \sigma_t^2} \tag{1}$$

where σ_o is the axial stress applied and σ_t is the tensile strength of concrete (σ_t = 4.8 MPa).

Approximate agreements between the experimental and theoretical values are exhibited for all columns regardless of the level of axial load applied, although they have a tendency that the experimental results are less than the theoretical predictions. Also the predic–

tions for the inclination of cracks obtained from the elasticity agree with the experimental results approximately. These results indicate that the diagonal tension cracking stress of the columns using ultra-high strength concrete of 120 MPa grade can be estimated by Eq.(1) obtained from the elasticity.

4.2 Ultimate shear strength

In recent years, new seismic design guidelines called "Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Ultimate Strength Concept (1990)" was completed by the AIJ. A new shear design equation for reinforced concrete members based on the lower-bound theorem in the limit analysis procedure was also given in the design guidelines. The most significant characteristic of the equation is that its solutions are obtained by cumulating data on the strength of two types of shear resistant mechanisms, the truss mechanism and the arch mechanism, as illustrated in Fig.5. The equation is given as follows.

$$V_u = V_t + V_a$$
$$= b \cdot j_t \cdot p_w \cdot \sigma_{wy} \cdot \cot\phi + \tan\theta(1-\beta) b \cdot D \cdot \nu \cdot \sigma_B \qquad (2)$$

where V_t = shear contributed by the truss mechanism
V_a = shear contributed by the arch mechanism
b = width of the member
D = depth of the member
j_t = distance between the compressive and tensile reinforcement
p_w = ratio of transverse reinforcement $[= a_w/(b \cdot S)]$
a_w = area of transverse reinforcing bars within a hoop spacing of S
β = ratio of compressive stress of concrete struts in the truss mechanism to effective compressive strength of concrete, $\nu \cdot \sigma_B$
ϕ = angle of inclination of concrete struts in the truss mechanism
θ = angle of inclination of a concrete strut in the arch mechanism
ν = factor of effective compressive strength of concrete
σ_B = concrete compressive cylinder strength
σ_{wy} = yield stress of transverse reinforcing bars $[2400 \leq \sigma_{wy} \leq 25 \cdot \sigma_B$ (unit : kgf/cm^2)]

In the design guidelines, two methods which are the A and B methods are prepared for the shear design equation, Eq.(2). For the A method, the following values are taken on factors in Eq.(2).

$$\tan\theta = \sqrt{(L/D)^2 + 1} - L/D \qquad (3a)$$
$$\beta = (1 + \cot^2\phi) p_w \cdot \sigma_{wy}/(\nu \cdot \sigma_B) \qquad (4a)$$
$$\nu = 0.7 - \sigma_B/2000 \quad \text{(unit : kgf/cm}^2\text{)} \qquad (5a)$$
$$\cot\phi = \min(\cot\phi_1, \cot\phi_2, \cot\phi_3) \geq 1.0 \qquad (6a)$$

where $\cot\phi_1 = 2.0$ $\quad [\phi = 26.6°]$
$\cot\phi_2 = j_t/(D \cdot \tan\theta)$
$\cot\phi_3 = \sqrt{\nu \cdot \sigma_B/(p_w \cdot \sigma_{wy}) - 1}$
L = length of the member

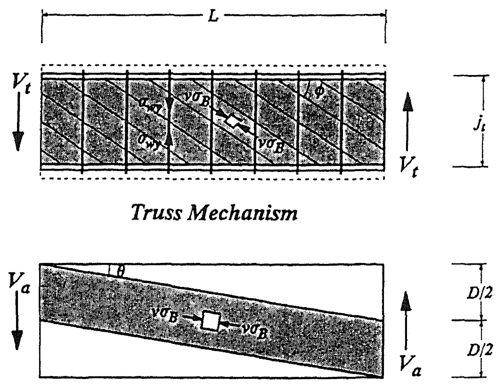

Fig. 5 Shear resistant mechanisms assumed in the shear design equation

On the other hand, the following values are used for the B method.

$$\tan\theta = \sqrt{(2 \cdot M/QD)^2 + 1} - 2 \cdot M/QD \qquad (3b)$$
$$\beta = 2 \cdot p_w \cdot \sigma_{wy}/(\nu \cdot \sigma_B) \qquad (4b)$$
$$\nu = (2 \cdot M/QD + 1)/4 \quad [0.5 \leq \nu \leq 1] \qquad (5b)$$
$$\cot\phi = 1.0 \quad [\phi = 45°] \qquad (6b)$$

where M/QD = ratio of shear span

Applicability of the shear design equation for the columns using ultra-high strength concrete of 120 MPa grade is examined here, though it can be applied to only members with concrete strengths of ranging from 20.6 to 35.3 MPa (210 to 360 kgf/cm^2) in the design guidelines. In examinations, Eq.(5a) which expresses the ν factor in the A method is changed to Eq.(7) which is the CEB proposal in the phase of drafting the model code (1987), because the value calculated using Eq.(5a) is obviously small for concrete of 120 MPa grade.

$$\nu = 1.7 \cdot \sigma_B^{-1/3} \quad \text{(unit : MPa)} \qquad (7)$$

Fig.6 compares the measured and predicted ultimate shear strengths for each specimen. The figures are expressed by the maximum shear stress versus transverse reinforcing stress relationship in each level of axial load applied, and include the theoretical lines indicating the shear strengths calculated using the A and B methods (solid and chain lines) and the flexural strength (shaded line). The experimental and theoretical values for each specimen are already listed in Table 3.

Note that the theoretical strength of the columns is given by a minimum value between the flexural and shear strength, V_f and V_u, as indicated in Table 3. The theoretical strength of all specimens is given by the shear strength when the A method is used. In the case of using the B method, it is given by the flexural strength for specimens C61, C62, C63 and C33, and the shear strength for specimens C31 and C32, respectively. The experimental strength of all specimens is between the theoretical strength calculated using the A and B methods. However, it can be considered that predictability of the B method is better than that of the A method, considering that yielding of longitudinal reinforcing bars has been observed in specimens C61, C63 and C33.

The shear strength measured for specimens C62 and C32 which used the high strength transverse reinforcement was obviously less than that for specimens C61 and C31, though the level of transverse reinforcing stress, $p_w\sigma_{wy}$ was almost the same, as shown in Chapter 3.1. It is indicated that the effect of transverse reinforcement on the ultimate shear strength of ultra-high strength reinforced concrete columns can not be appropriately estimated by only the index of $p_w\sigma_{wy}$ in the both A and B methods. To apply the shear design equation in the AIJ design guidelines to ultra-high strength reinforced concrete members, the factor of effective compressive strength of concrete used to the macroscopic models assumed should be appropriately estimated for high strength concrete, and the yield stress of transverse reinforcement used to the equation should be set on an upper limit considering the width of cracks and the transverse strain of the members.

5 CONCLUSIONS

From the experimental and analytical results on the shear strength of ultra-high strength reinforced concrete columns presented above, the following conclusions can be drawn :

1) The diagonal tension cracking load of ultra-high strength reinforced concrete columns increases in proportion as the applied axial compression increases and can be estimated by using the elasticity.
2) The ultimate shear strength of such columns decreases according as the level of transverse reinforcing stress decreases and the applied axial compression increases.
3) Columns using the normal strength transverse reinforcement have higher capacity and stiffness after the cracking than columns using the high strength one, if the level of transverse reinforcing stress, $p_w\sigma_{wy}$ is the same.
4) Predictability of the shear equation using the B method is better than that using the A method for ultra-high strength reinforced concrete columns.
5) To apply the shear design equation in the AIJ design guidelines to ultra-high strength reinforced concrete members, the factor of effective compressive strength of concrete and an upper limit of the yield stress of transverse reinforcement used to the equation should be examined in more detail.

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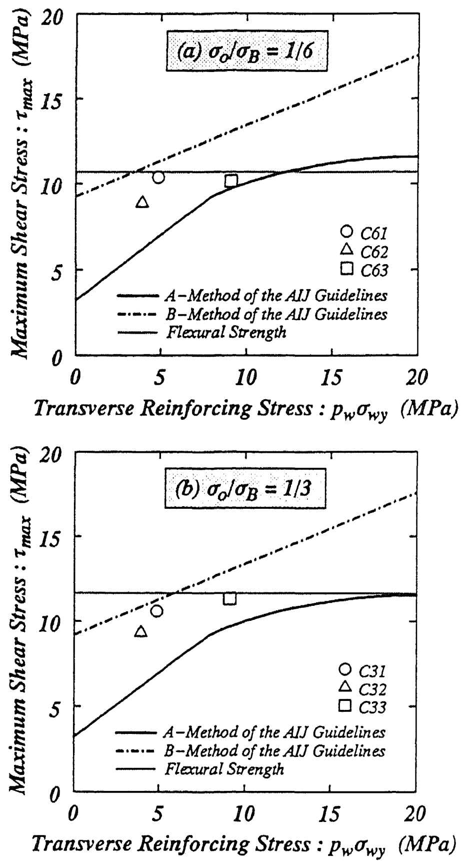

Fig. 6 Shear strength variation as transverse reinforcing stress is increased

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