

CYCLIC RESPONSE OF HIGHLY CONFINED HSC COLUMNS

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SUMMARY

Four 250mm square concrete columns were made and tested under combined reversed cyclic lateral load and constant axial load. These columns were made of high-strength concrete (HSC) with designed compressive strength of 80MPa and high-strength longitudinal bars having yield stress of 900MPa. These tests were programmed to investigate effectiveness of a new confinement method for the HSC. This confinement method utilizes steel tube as lateral confiner for HSC columns instead of common transverse hoops or spirals.

Test results have indicated that confinement of HSC columns by the steel tube could greatly improve the ductility of HSC columns. As the HSC column was subjected to applied axial compression with axial load ratio of 0.33, confinement by thin square steel tube with width-to-thickness (B/t) ratio of 80 was enough to ensure stable and ductile cyclic performance to HSC columns. As the axial compression applied became so high as the axial load ratio was 0.5, use of square steel tube with B/t ratio of 44 could ensure HSC column behave in a very ductile manner up to large deformation. Another important finding was that confinement by steel tube and use of both HSC and high-strength bar could made a ductile high-performance column with very limited residual permanent displacement without special technical effort.

To help structural engineers evaluate ultimate capacities of the HSC column confined by steel tube, a simple design method was also proposed in this paper. Comparison between the theoretical predictions and the experimental ultimate capacities has verified validity of the proposed design method.

INTRODUCTION

Due to its high strength and sound durability, high-strength concrete with compressive strength over 60MPa has gained more and more applications in high-rise buildings in Japan (Namiki et al. [1]). To promote the use of HSC in earthquake-prone Japan, structural researchers in Japan have conducted numerous theoretical and experimental studies in the last two decades (JICE [2], AIJ [3]) with aim at developing reliable and effective confinement methods for the HSC and establishing an reasonable design method for the confined concrete structures.

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It has been well known that confinement of concrete by transverse steels is effective in improving the ductility of concrete. Traditionally, structure engineers have used transverse spirals or hoops as the lateral confiner to concrete beams and columns. With the increase of concrete strength, however, the necessary amount of spirals or hoops may become so large that existence of transverse steels would worsen placement of the concrete and form a stiff "steel wall" between the confined core and the unconfined shell of concrete members. This stiff steel wall tends to spall the concrete shell and buckle the longitudinal bars prematurely. Therefore, a new and more reliable confinement method is required for the HSC.

This paper presents a new confinement method for the HSC. In this new confinement method, the steel tubes are used as lateral confiner for the concrete member in lieu of the conventional transverse hoops. Confinement by steel tube has several advantages over the confinement by traditional transverse hoops in that; 1) it is easy to provide confining pressure strong enough to improve ductility of HSC without hindering placement of concrete, 2) steel tube can provides confinement effect to the whole concrete section including the concrete cover, 3) the steel tube can work as form for concrete columns, and 4) use of steel tube makes it possible to assess "pure confinement effect" of transverse steel on the ultimate capacities of concrete members since no cover concrete exists.

Of the four beneficial features of confinement by steel tube, the second one is particularly noteworthy, considering that spalling-off of the concrete cover often causes serious structural problems such as significant degradation of load-carrying capacity due to loss of effective section area and premature buckling of longitudinal rebars, both of which cannot be effectively prevented by conventional transverse spirals or hoops. In addition, confinement of the whole concrete section can not only improve the ductility of the column, but also enhance its ultimate load-carrying capacity, which hardly can be expected in the columns confined by common hoops or spirals.

Purposes of this paper are: 1) to experimentally verify validity of the proposed confinement method as used in HSC columns with designed compressive strength of 80 MPa, and 2) to present experimental information about the seismic performance of confined HSC columns, and 3) to propose a simple design method enabling structural engineers to simply evaluate the ultimate capacities of confined HSC columns.

PROGRAM OF EXPERIMENT

Reinforcement details of test columns

Four 1/3 scale concrete columns were made to simulate the lowest story column in high-rise buildings. All of the four columns were square prismatic column and confined by square steel tubes with inner width of 250mm. The shear span was 625mm to give a shear span ratio of 2.5. The test columns were designed to fail in flexure.

Figure 1 shows the reinforcement details and Table 1 lists the outlines of the specimens. As can seen from Figure 1, the longitudinal bars consisted of twelve D13 deformed bar uniformly distributed along the perimeter of column to a reinforcement ratio of 2.44%. Square steel tubes were strengthened with cross-type inner stiffener that spread 1.0D (D = depth of column section) from the bottom of each column. The inner stiffeners were welded to steel tubes to enhance the confinement effectiveness of square steel tube. Clearance of 10mm was provided between the end of steel tube and the bottom of each column to ensure that the steel tube only provides lateral confining pressure rather than direct sustain to axial stress due to axial load as well as bending.



Figure 1 Reinforcement details of specimens

	Concrete		Width	Thickness of	Yield stress	Yield stress of
Notation	cylinder	Р	to thickness	steel tube	of steel tube	longitudinal
	strength f_c	$A_g f_c$	ratio	t	f _{yt}	bars f_{yt}
	(MPa)		B/t	(mm)	(MPa)	(MPa)
THRC32N33	84.7	0.33	80.1	3.2	275	
THRC32N50	93.2	0.50				926
THRC60N33	84.7	0.33	43.7	6.0	285	
THRC60N50	92.5	0.50				

 Table 1 Outlines of the test columns

The steel tubes used to confine columns were made in laboratory of Kyushu University by bending steel plates with target thickness into channel-shape, and then welding two of them to form a closed tube. The same steel plates were used to make the perimeter tube as well as the inner stiffeners. Figure 2 displays the tensile stress-strain curves of the steels used in the tests. Each curve in Figure 2 represents the average of three test coupons. As obvious from Figure 2, the D13 longitudinal bar is a high-strength reinforcement (KSS785) without clear plateau, and hence the yield stress shown in Table 1 means the well-known 0.2% offset yield strength.

The experimental variables were thickness of steel tube and axial load level. The thickness was 3.2mm and 6.0mm to give outside width to thickness ratio (B/t ratio) of about 80 and 44, respectively. The axial load level expressed in term of axial load ratio, η (=P/(A_g f_c), where P is the applied axial compression, A is the



Figure 2 Tensile stress-strain curves of the steels



Figure 3 test set-up and measurements

gross area of column section, and f_c is the concrete cylinder strength), was 0.33 and 0.50. The axial load ratio of 0.33 corresponds to the upper limit recommended by the AIJ standard (AIJ[4]) for the normal-strength concrete columns confined by common transverse hoops, and $\eta=0.5$ was targeted to investigate to what axial compression level the steel tube confinement can stand.

Ready-mixed concrete with designed compressive strength of 80 MPa was used to fabricate all of the four columns. Common Portland cement and coarse aggregate with maximum size of 20 mm were used to make the HSC. The standard cylinder (100mm diameter and 200mm height) strength at the testing stage for each specimen is given in Table 1.

Test set-up and measurements

Figure 3 shows the test set-up and measurements. This test set-up was used to apply reversed cyclic lateral loading after the test column was axially loaded to the target compression. Cyclic lateral load was a displacement-controlled type, and the drift ratio R of column was adopted to control the loading process. The column was first loaded to the amplitude of R=0.0025rad for one complete cycle, and then the R was increased to 0.005rad and 0.0075rad for two respective complete cycles. After that, drift ratio R was increased to 0.01rad (3 cycles), 0.015rad (3 cycles), 0.02rad (3 cycles), 0.025rad (2 cycles), 0.03rad (2 cycle), 0.04rad (2cycle), 0.045rad (2 cycles), and finally 0.05rad (a half cycle).



Figure 4 The measured lateral load V versus drift ratio R relationships

Two displacement transducers were used to measure the lateral deformation of the column, and dividing the average of them by the shear span gives the drift ratio R. In addition to the lateral displacement, the axial deformation of each test column was also measured using four displacement transducers as shown in Figure 3. For each test column, a total of 28 strain gages were embedded to measure the strains of longitudinal rebar and the surface strain of steel tube.

OBSERVED BEHAVIOR AND MEASURED RESULTS

Figure 4 displays the experimental results of lateral load V versus drift ratio R relationships. The solid and linked lines superimposed in Figure 4 represent mechanism lines corresponding to flexural strengths of confined columns and of unconfined columns, respectively. The calculation method for the ultimate moment of confined and unconfined HSC columns will be described in the next section. K=1.0 shown in Figure 4 means no confinement effect was taken into account when calculating the ultimate moment, while K>1.0 represents that confinement effect by the steel tube has been taken into consideration.

As one can see from Figure 4, all test columns developed their theoretical ultimate capacities, and ignorance of confinement by the steel tube resulted in very conservative prediction of the ultimate load capacity of tube-confined HSC columns.

Specimen THR32N33, which was confined by thin square steel tube with B/t ratio of 80 and subjected to axial compression having axial load ratio of 0.33, exhibited very stable cyclic response. This specimen reached its maximum lateral load as R reached 0.02rad, and maintained its load-carrying capacity till R was increased to 0.03rad. A little degradation in load-capacity was observed as R was larger than 0.035rad, but the column maintained over 80% of its maximum capacity till R reached 0.04rad. When the

column was deformed to R=0.045rad, rupture in welding portion of steel tube occurred, and the test was terminated.

Loading of the specimen THR32N50, due to its higher axial compression than specimen THR32N33, was terminated during it was pulled to the amplitude of 0.02rad because of rupture in welding portion of steel tube, which resulted in loss of lateral load-carrying capacity and axial load-sustaining capacity of the column. This specimen, however, did show stable cyclic behavior and higher ultimate load-carrying capacity than theoretically predicted at the deformation level of R=0.015rad.

Specimen THR60N33 reached its maximum lateral load capacity as R reached 0.03rad, and maintained over 90% of its maximum load up to so large deformation as R was 0.04rad. Cyclic response was very stable till the end of testing.

Specimen THR60N50, while subjected to high axial compression with axial load ratio of 0.5, showed very ductile and stable cyclic performance till the end of testing. The strength degradation at the deformation level of 0.04rad was only 10%.

As to the confinement effect of steel tube on the ultimate capacity of confined HSC columns, one can see from Figure 4 that this effect becomes more significant as the axial load level is higher. In the case of axial load ratio of 0.33, the difference in ultimate load capacity between the specimen THR32N33 and the specimen THR60N33 was only 5%; the later is higher than the former. This strength difference, however, becomes 20% for the specimens under axial load with axial load ratio of 0.5.

From the above observations, one can see that confinement by square steel tube can provide sufficient confinement effect to HSC columns as expected. For HSC column subjected to axial load with η =0.33, confinement by thin square steel tube with B/t ratio of 80 and strengthened with inner stiffeners can assure the HSC column deformation capacity larger than 0.04rad. As the applied axial load ratio is 0.5, confinement by steel tube with B/t ratio of 44 can assure ductile and stable performance till R=0.05rad.

Another important finding from Figure 4 is that use of high-strength concrete and longitudinal bars can reduce the residual permanent lateral deformation significantly. With exception of specimen THR32N50, whose steel tube prematurely ruptured at welding portion, the highly confined HSC columns exhibited very limited residual lateral deformation, which, for example, was only 0.002rad as the specimen was unloaded from the drift ratio amplitude of 0.02rad. Considering that R=0.02rad has been defined as the limit deformation at the life-safety state, this experimental fact implies that use of high strength materials can simply mitigate damage induced by large residual deformation without expensive and special efforts.

CALCULATION METHOD OF THE ULTIMATE MOMENT OF HIGHLY CONFINED HSC COLUMNS

Confinement effect of steel tube has been verified experimentally from the test results described in the previous section. In order for structural engineers to simply conduct ultimate capacity design of highly confined HSC columns, a reasonable design method, which can take confinement effect by steel tube on the ultimate capacities into consideration, is desirable.

This section presents a calculation method for the confined HSC column. This method was originally proposed by Sun and Sakino [5] for the HSC members confined by rectilinear transverse reinforcements inclusive of both square steel tube and conventional hoops. It is well known that ultimate moment of a



Figure 5 Idealization of the equivalent stress block for confined concrete



mot effectively confined
Figure 6 Definition of the lateral unsupported length of steel tube

concrete column is usually calculated from the axial load versus ultimate moment interactive diagram, i.e. N-M interactive curve, of the column section. According to the method proposed by the first author and his colleague [5], the N-M interactive curve of a tube-confined HSC column can be obtained on the basis of the following assumptions: 1) plane remains plane after bending, 2) concrete doesn't sustain tensile stress, 3) moment and axial force carried by the compressed concrete can be obtained by using the equivalent rectangular stress block shown in Figure 5, 4) stress-strain relation of the high-strength longitudinal bar without clear yield plateau can be defined as later.

The stress block shown in Figure 5 was developed by Sun and Sakino[5] for helping engineers to conduct confinement design of confined concrete members, after modeling of stress-strain curve for the confined concrete. Ultimate strain of the compressed concrete as well as the coefficients α and β that define the shape of the stress block are obtained by the following equations:

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

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

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

in which ε_{cm} is the ultimate strain of the confined concrete, ε_{co} is the strain at the confined concrete strength f_{cc} , and K is the ratio of confined concrete strength f_{cc} to unconfined concrete strength f_c , and is an index



Figure7 Stress-strain curve of the high-strength D13 (KSS785) bar

measuring the confinement degree by the steel tube. These two parameters, ε_{co} and K, can be obtained by the following equations:

$$\mathcal{E}_{co} = 0.94 (f_c)^{1/4} \times 10^3 \begin{cases} 1+4.7(K-1), & K \le 1.5\\ 3.35+20(K-1.5), K > 1.5 \end{cases}$$
(4)

$$K = \frac{f_{cc}}{f_c} = 1 + 46 \frac{(B/t - 1)}{(B/t - 2)^2} \left(\frac{t}{C}\right) \frac{f_{yt}}{f_c}$$
(5)

where f_{yt} is the yield stress of steel tube, B and t are the outside width and thickness of steel tube, respectively, and C is the unsupported length of steel tube (see Figure 6). Backgrounds for these equations can be found elsewhere [5] and not given here.

The stress-strain relationship for high-strength steel without clear yield plateau is defined as follow:

$$f_{s} = E_{s}\varepsilon_{s}\left\{Q + \frac{1-Q}{\left[1 + \left|\varepsilon_{s}/\varepsilon_{ch}\right|^{N}\right]^{1/N}}\right\}$$
(6)

in which f_s and ε_s are the stress and strain of longitudinal bar, respectively, E_s is the young's modulus, Q is the ratio of tangential stiffness at the peak of stress-strain curve to young's modulus E_s , N is the parameter defining the curvature of the stress-strain curve, and ε_{ch} is the characteristic strain. Figure 7 shows outline of the stress-strain curve.

Equation 6 was first proposed by Menegotto and Pinto [6] to trace the unloading and reloading paths of steel subjected to cyclic loading. The authors have recently applied it to evaluate monotonic stress-strain relation of high-strength longitudinal bar not showing clear yield plateau (Sun et al. [7]). As obvious from Equation 6, three parameters need to be determined to completely define the stress-strain curve. They are curvature parameter N, tangential stiffness ratio Q, and characteristic strain \mathcal{E}_{ch} . Based on their tensile tests on high-strength bars, Sun et al [7] developed the following expressions to calculate these parameters:

$$Q = \frac{E_{tu}}{E_s} = 0.1(\varepsilon_{su})^{-2.5}, \ (\varepsilon_{su} \ in \ \%)$$
(7)



Figure 8 Comparison of the theoretical and experimental ultimate moments

Notation	V _{max}	R _{max}	M _{max}	M _{cu}	M _{cc}	M _{max}	M _{max}
	(kN)	(0.01rad)	(kN-m)	(kN-m)	(kN-m)	M _{cu}	M _{cc}
THRC32N33	398	2.47	277.1	197.7	232.1	1.40	1.19
THRC32N50	404	1.47	279.2	212.0	241,2	1.32	1.16
THRC60N33	417	2.72	291.4	197.7	281.4	1.47	1.04
THRC60N50	480	2.22	340.1	210.7	307.5	1.62	1.11

 Table 2 Measured and calculated ultimate moments

Note: V_{max} is the measured maximum lateral load

 R_{max} is the inter-story drift ratio at the V_{max}

 V_{max} is the measured maximum moment including P- Δ effect.

 M_{cu} is the calculated moment corresponding to K=1.0, i.e. unconfined column

M_{cu} is the calculated moment corresponding to K>1.0, i.e. confined column

$$N = 3.0 \tag{8}$$
$$f_{su} - QE_s \varepsilon_{su} \tag{9}$$

$$\varepsilon_{ch} = \frac{J_{su} - QL_s \varepsilon_{su}}{E_s (1 - Q)} \tag{9}$$

where f_{su} and ε_{su} are the stress and strain at the peak of stress-strain curve. Obviously, one can simply determine the complete stress-strain curve of high-strength bar only if the information about the peak is known.

Figure 8 compares the theoretical N-M interactive curves with the measured ultimate moments, while Table 2 lists the digital comparison of the theoretical predictions and the experimental results. From Figure 8 and Table 2 one can see that the proposed method predicts the experimental results very well.

The ratio of experimental moment to theoretical result has a mean value of 1.12 and standard deviation of 0.05. On the other hand, ignorance of confinement by steel tube apparently underestimate the ultimate capacities of the confined HSC columns. Particularly for the columns under high axial load with axial load ratio of 0.5, the discrepancy becomes as large as 67%. This fact implies importance of consideration of confinement effect, since underestimation of the ultimate moment capacity doesn't always means beneficial design.

CONCLUDING REMARKS

A new confinement method for high-strength concrete columns was proposed in this paper. To prove its effectiveness, four 1/3 scale HSC columns were made and tested under reversed cyclic lateral load. From the experimental results reported in this paper, the following conclusions can be drawn.

- 1) Confinement by square steel tube can ensure high seismic performance to HSC columns. When strengthened with inner stiffener, confinement by thin steel tube with B/t ratio of 80 could make the HSC column under high axial compression behave in a very stable manner up to large deformation.
- 2) Use of high-strength concrete and high-strength longitudinal bar is effective in reducing the permanent residual displacement of concrete column.
- 3) The ultimate moment capacity of highly confined HSC column can be accurately evaluated by the proposed method. Ignorance of confinement by the steel tube results in unnecessary conservative estimation of the ultimate capacity of a confined HSC column, which may lead to unexpected failure mode.

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