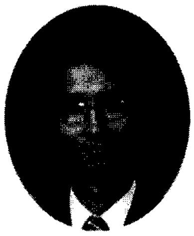
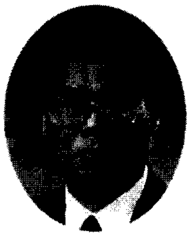
Seismic Behavior of High-Strength Concrete Square Short Columns Confined in Thin Steel Shell

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ABSTRACT

Experiments were carried out to investigate the seismic behaviors, such as lateral strength, ductility, and energy-dissipation capacity, of high-strength concrete (HSC) square short column confined in thin steel shell. The primary objective of the study was to investigate the suitability of using HSC square columns confined in thin steel shell in region of moderate-to-high seismic risk. A total of six columns, consisting of two ordinarily reinforced concrete square short columns and four reinforced concrete square short columns confined in thin steel shell was tested. Column specimens, short columns in a moment resisting frame with girder, were tested under a constant axial and reversed cyclic lateral loads. To design the specimens, transverse reinforcing methods, level of axial load applied, and the steel tube width-thickness ratio (*D/t*) were chosen as main parameters. Test results were also discussed and compared in the light of improvements in general behaviors, ductility, and energy-absorption capacities. Compared to conventionally reinforced concrete columns, the HSC columns confined in thin steel shell had similar load-displacement hysteretic behavior but exhibited greater energy-dissipation characteristics. It is concluded that, in strong earthquake areas, the transverse reinforcing method by using a thin steel shell (D/t=125) is quite effective to make HSC short columns with very strong and ductile.

Keywords : Square Column, Confined in Thin Steel Shell, High-Strength Concrete, Seismic Behavior, Transverse Reinforcing Method, Earthquake-Resistant Structures

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1. Introduction

Shear failure of HSC square column in a moment resisting frame with wall girder has been one of the major problems associated with the performance of high strength reinforced concrete buildings under earthquake excitations. Such short and, hence, relatively stiff members tend to attract a greater portion of the seismic input to the high-rise building during an earthquake and require the generation of large seismic shear force to develop the moment capacity of columns. It has been widely accepted that the most effective method to prevent the shear failure of high strength reinforced concrete short square columns is to use closely placed hoops. However, the very strong columns with large shear capacity cannot be made by the shear reinforcement such as closely spaced steel hoops or spirals. This is one of reasons why the transverse reinforcing method confined in steel tubes should be developed. A transverse reinforcing method to prevent reinforced concrete columns from brittle failure is introduced utilizing the thin steel shell by the shear reinforcement.

This paper describes results of an experimental investigation conducted to study the hysteretic behavior, such as lateral strength, ductility and energy-absorption capacity, of rectangular HSC column confined in a thin steel shell subjected to constant compressive axial and reversed cyclic lateral loads. The investigation addressed the following issues : the effect of transverse reinforcing methods and level of axial load applied on the hysteretic behavior of rectangular HSC columns confined in thin steel shell. The purpose of this research was to provide experimental data on the seismic behavior of these members as a basis for developing guidelines for designing square HSC columns confined in thin steel shell.

2. Privious Research and Design Specifications

The use of steel tube to enhance shear strength of columns is not new. Park, Priestley, and Walpole[5] investigated the seismic performance of steel-encased plain and reinforced concrete sections with D/t ratio of 34 to 214 but found that plastic buckling of the steel tube in the hinge regions tended to occur when the columns were subjected to large cyclic lateral displacement. To avoid the buckling of the tube observed, Tomill, Sakino, Xiao, and Watanabe[6] proposed a type of steel-encased composite columns that utilized the steel tube for transverse reinforcement only, while conventional longitudinal steel was use for flexural reinforcement. This super-reinforcing concrete column exhibited both more ductile response and an increase in the ultimate shear load capacity when compared to conventionally reinforced concrete columns. Although satisfactory results were obtained by Tommi, Sakino, Xiao, and Watanabe for rectangular columns, it was found that steel tube was very thick.

Current nonseismic design practice for HSC columns confined in steel tube is based on provisions provided in the Korean Concrete Institute (KCI) code. The KCI code[1] requires that the strength of HSC columns confined in steel tube be computed in a way similar to that for conventionally reinforced concrete sections with modifications to the radius of gyration and EI. The specification places limitations on the minimum thickness of the steel tube. The minimum thickness(t) for steel tubes in round composite columns is calculated using the following equation

$$t \geq b\sqrt{f_y/3E_s} \qquad (1)$$

where f_y, E_s and b are yielding stress of steel, modulus of elasticity, and width of each face, respectively. This equation is based on achieving yielding stress in an empty shell under monotonic longitudinal compressive load before local buckling occurs.

In Japan, the steel reinforced concrete (SRC) construction has long been popular. A considerable amount of research has been conducted on the subject, including the effects of seismic loading. Although the Japanese guidelines are comprehensive for seismic design, much of the research and design guidelines are not applicable to Korean construction field due to differences in design practice. Moreover, though designers want to apply high-strength concrete, the scope of application in the standard is still limited to a moderate strength concrete.

Neither specification accounts for any possible enhancement in strength nor ductility due to confined concrete. The minimum thickness requirement in the KCI code is designed to prevent buckling of an empty steel shell prior to longitudinal yielding. However, The AIJ code[(4)] has shown that this requirement is unnecessarily restrictive for concrete-filled tubular members. Unfortunately, in Korea. the advantages of composite construction cannot be exploited in design because of the current lack of KCI design provisions.

Therefore, improvements in current code provisions are necessary to provide a rational design basis for HSC members confined in steel tube and to realize the benefits of this type of composite construction.

3. Outline of Experiments

3.1 Test Specimens

Six one-third-scaled column specimens with square cross sections were tested. The properties of each specimen are shown in Fig. 1 Each column had a cross section of 250 mm×250 mm and height of 1000 mm, representing the one-third-scaled model of a 750 mm×750 mm prototype column. A ratio of the depth to clear height was 2.0 for all the specimens. The stubs were attached to the top and bottom of the column and were heavily reinforced, as shown by typical reinforcing details in Fig. 2.

The longitudinal reinforcement in each test specimen consisted of twenty-four 10mm-diameter deformed bars D10 arranged symmetrically around the perimeter to give a

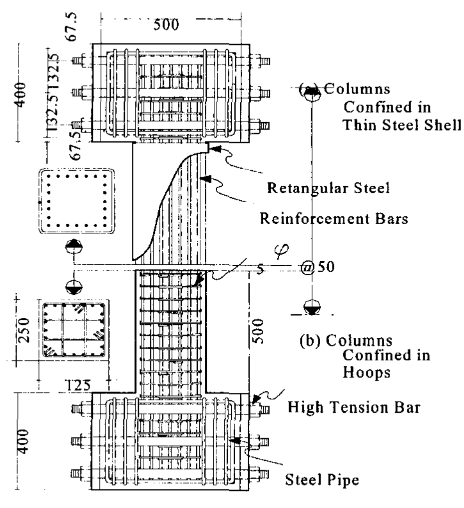
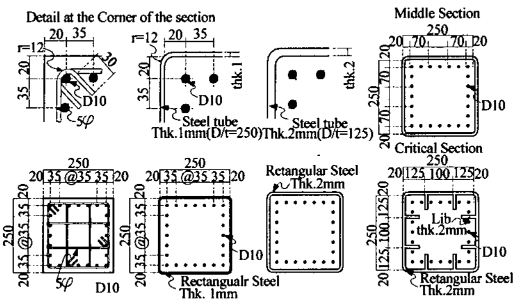


Fig. 1 Overall view of test specimens, unit=mm

steel ratio of 2.73 percent.



(a) RC-0, RC-3 (b) TC-A3 (c) TC-B0, TC-B3 (d) TC-B3R
Fig. 2 Details of transverse reinforcement

3.2 Reinforcement Details

The reinforcement details of the test specimens of HSC column are listed in Table 1 and shown in Fig. 2. The selection of column details was based upon current KCI requirements, typical industry applications in buildings, and testing equipment limitations.

However, the KCI minimum thickness requirement was intentionally violated. As shown in Table 1, the test specimens were classified into two groups, Group-RC and Group-TC, according to the transverse reinforcing method.

Group-RC had a three overlapping hoops per set: one square peripheral hoop and two rectangular overlapping interior hoops, giving four effective hoop leg areas in each direction. The amounts of transverse reinforcement provided in the test of Group-RC specimens were 160 percent of KCI requirements. In Group-TC, wall thicknesses of the two steel tubes were 1 and 2mm respectively. The thickness necessary to achieve yielding stress before local buckling occurs is 6mm. Therefore the thickness of the steel shell of Group-TC specimens violated the KCI code. The steel tube plays a role of a transverse reinforcement only. TC-B3R specimen was exactly the same as Group-TC columns except for the presence of ribs at the top and bottom potential plastic hinge regions to prevent the local buckling of steel tube. The ribs attached to the interior surface of TC-B3R specimens were 2 mm×50 mm×375 mm (1.5 times of depth) plate and were attached to the shell with a fillet weld.

Table 1 Details of test specimens

Specimen	Transverse reinforcing method			f_c'	Axial Force	
	Size cm	D/t	P_s %	MPa	N kN	$N/A{\cdot}f_c'$ (N/P_b)
RC[1)]-0	φ5 @50 3 overlapping hoops	-	0.73	44	-	-
RC-3					784	0.28 (1.24)
TC[2)]-A3	□-25×25×0.1	250	1.60			
TC-B0	□-25×25×0.2 with ribs	125	3.20		-	-
TC-B3					784	0.28 (1.24)
TC-B3R						

Note :(1) RC : Ordinary Reinforced Concrete Columns
(2) TC : Columns Confined in Thin Steel Shell
(3) Steel tube with ribs at the top and bottom potential plastic hinge regions

3.3 Experimental Methodology

As shown in Fig. 3, the test setup was designed to subject the model columns to constant axial load (P/Pb=1.24) and cyclic horizontal forces in double curvature condition. The horizontal force was applied by a ±150 mm stroke hydraulic jack with the capacity of

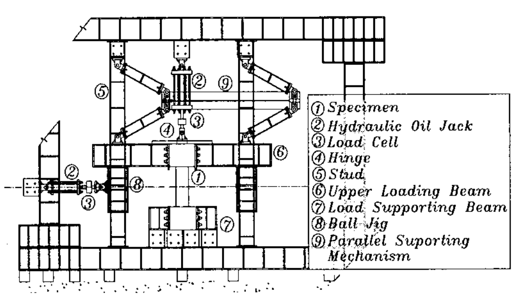


Fig. 3 Details of test setup

490 kN in compression and tension. An axial load of 784 kN was applied to the column by a hydraulic jack at the upper stub of the column except to RC-0 and TC-B0 specimens. Lower stub was post tensioned to a reaction frame using six high tension bolts.

The initial loading cycles were applied corresponding to a peak displacement increment of 1.0mm until the column developed the yielding. After the column developed the first yield, three complete loading cycles were performed at peak displacements corresponding to the ductility factors, μ= 1, 2, 3, 4, 5, 9, and 11.

3.4 Material Properties

Mechanical properties of concrete are shown in Table 2. The maximum size of the aggregate was 25mm. The concrete used had a water/cement ratio of 35 percent (by weight), and its slump was 180 mm. Table 3 summarizes the properties for the steel tubes and reinforcement bars.

Table 2 Material properties of concrete

Days	f_c'	f_{sp}	ε_u	υ	E_C	Test Column
	MPa	MPa	%		GPa	
28	46.4	4.4	2,1	0.18	30.5	RC-0, RC-3
50	44.7	4.5	2,0	0.18	30.9	TC-A3,TC-B0
60	48.5	4.6	2,5	0.20	32.4	TC-B3,TC-B3R

Note f_c' = Uniaxial compressive cylinder strength of concrete
f_{sp} = Tensile strength of concrete by split cylinder test

Table 3 Material properties of steel

Type	E_s	F_y	ε_y	F_t	Elongation
	GPa	MPa	%	MPa	%
D10	201	466	2,1	520	12.0
φ5	185	551	4,1	667	10.0
Plate	197	349	4,5	397	7.8

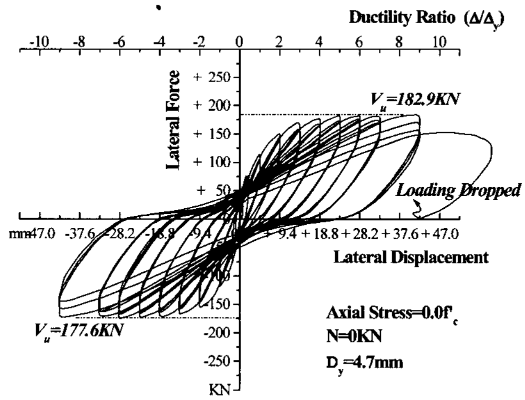
Note E_S = Young's Modulus, ε_y = Yield Strain
F_y = Yield Stress, F_t = Maximum Stress

4. Experimental Results and Comments

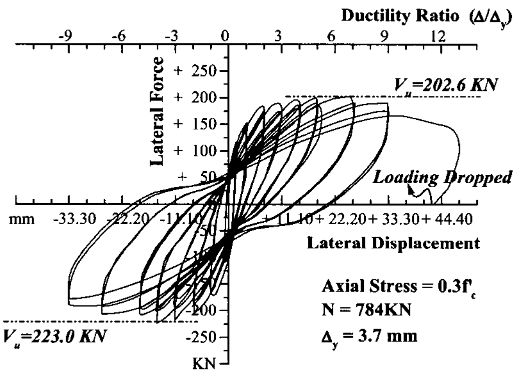
4.1 General Behavior

Horizontal force versus horizontal displacement, measured at the top of columns, hysteretic curves for six test specimens are shown in Figs. 4 through 5. Fig 6 shows the load-displacement envelope curves for each test column.

For the two ordinarily HSC columns, the first flexural crack was observed at the bottom end of the columns. As the displacement increased, flexural cracks extended into the center region and then propagated into flexural-shear cracks.

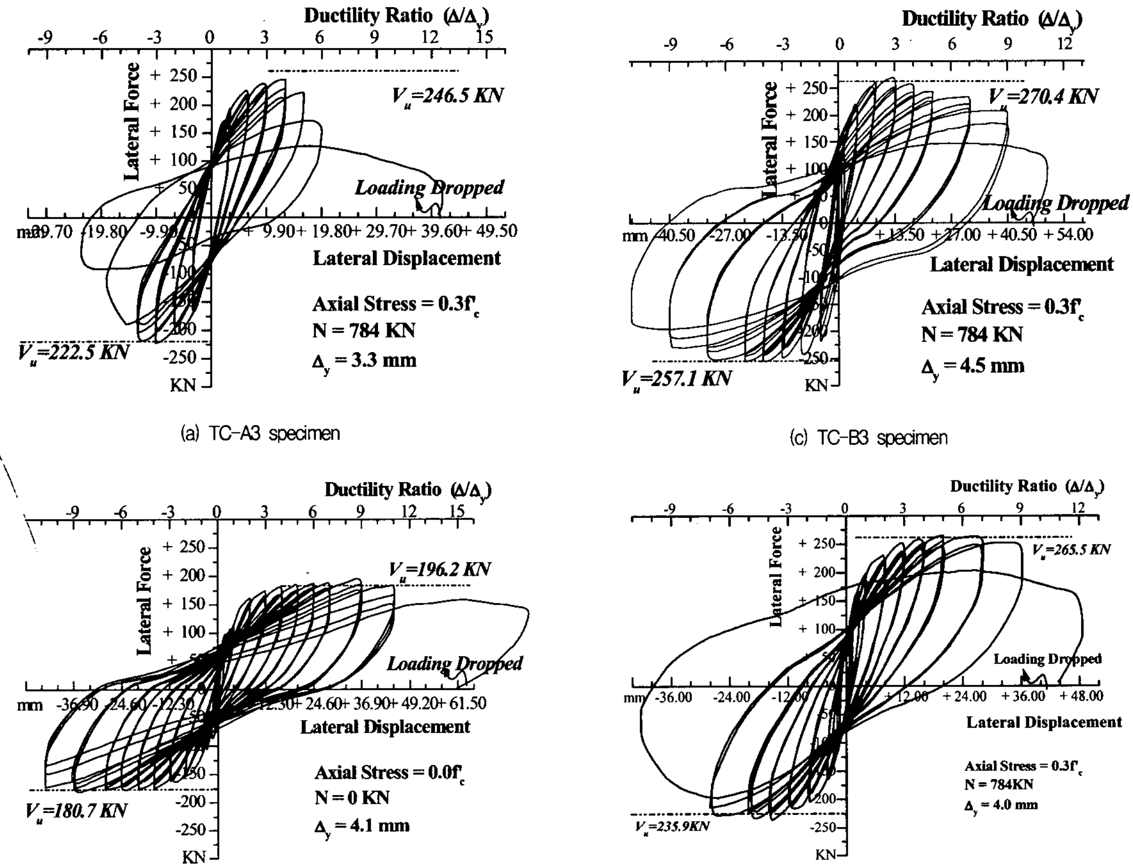


(a) RC-0 specimen



(b) RC-3 specimen

Fig. 4 Horizontal load-displacement hysteretic curves in group-RC



(a) TC-A3 specimen

(c) TC-B3 specimen

(b) TC-B0 specimen

(d) TC-B3R specimen

Fig. 5 Horizontal load-displacement hysteretic curves in group-TC

In the case of column confined in hoops, almost entire cover concrete at the base of column was spalled off, which resulted in the significant deterioration of the bond characteristics. The rupture of the hoops was also observed. For ordinarily HSC column, crack pattern and final failure mode regardless of axial stress within $0.3f_c'$ was similar. RC-0 was designed to be compared to RC-3 with different levels of axial load applied and to TC-B0 with the same levels of axial load applied, but with different transverse reinforcing method. Yielding displacement for RC-0 was larger than that of RC-3. Comparisons of between RC-0 and RC-3 showed that the maximum lateral loads were increased by about 8.8 percent when the axial stress of specimen was up to $0.3f_c'$. In case of column confined in steel tube, TC-B0 was designed to be compared to TC-A3 with different D/t ratios and to TC-B3R with the same D/t ratio, but with ribs on the interior surface of the steel tube at the top and bottom potential plastic hinge regions. In steel tube with $D/t=250$, local buckling occurred during the first cycle to $\mu=\pm4$ as shown in Fig. 9(a) and fractured during the first cycle to $\mu=\pm6$. This is because steel tube is not effective in confining HSC with large D/t ratios.

In the case of TC-B3, local buckling also occurred, but the hysteretisis curves shown in Fig. 7 indicate stable behavior with good energy-dissipation characteristics.

Crack patterns of the tube columns were observed by cutting and removing the steel tube after test. Except for specimens TC-A3, bond splitting cracks, which extended vertically, were observed in HSC columns confined in steel tubed columns, but width of these cracks was very small. Remarkable crushing of the concrete was not observed for HSC columns confined in thin steel shell. This means that the concrete confined in the HSC square short columns confined in thin steel shell behaved in very ductile manner.

Table 4 Ductility ratio of test specimens

Test specimens	Ductility[1]	Rotation angle	Energy dissipation index[2]	Work damage index[3]
	μ	Rad	D_i	I_w
RC-0	11.09	0.052	95.40	56.2
RC-3	10.68	0.040	81.30	43.2
TC-A3	5.97	0.030	50.20	15.4
TC-B0	13.30	0.053	85.20	45.1
TC-B3	10.05	0.040	91.20	46.7
TC-B3R	11.97	0.048	105.20	63.1

Note(1) $\mu=\Delta_{80}/\Delta_y$ in which Δ_{80}=the lateral displacement until the end of cycle in which the lateral force is dropped to 80 percent of the maximum value, Δ_y=the yield displacement of the columns
(2) $D_i=E_t/[0.5P_y\delta_y(1+A'_s/A_s)^2]$
(3) The work index(I_w) for each cycle in which the load at peak deflection reached at least 75 percent of the load at first loading. $I_w=\Sigma(P_n/P_y)(\delta_n/\delta_y)$ in which n=number of cycles, $P/P_y\geq 0.75$

4.2 Comparison of Column Performance

In the following discussion, experimental test results are used to discuss the effects of different parameters on the performance of test columns.

Effect of transverse reinforcing methods

Table 5 shows two groups of test columns that could be compared for effect of transverse reinforcing methods on strength and ductility. For both test columns, the axial load was 30 percent of the column's axial load capacity. In the case of column confined in steel tube, comparisons of Fig. 5(b) and Fig. 5(c) show that the maximum lateral loads increased by about 32 percent.

However, results indicate that at this axial load level, transverse reinforcing method of steel confinement, had no influence on ductility ratio, as both specimens achieved μ of approximately ±9 corresponding to a 4 percent story drift index.

Both specimens have adequate displacement properties. It should also be noted that Series-RC specimens had approximately 160 percent of the area of transverse reinforcement required by seismic provisions of KCI.

However, Fig. 7 shows that TC-B3 had a greater energy absorption capacity than RC-3 at each level of displacement ductility, prior to reaching $\mu=\pm 9$. Table 4 also indicates that TC-B3 specimen has a higher values of energy dissipation index and work damage index than RC-3 specimen. The larger lateral loads and displacements exhibited by TC-B3 specimen generate a greater area within the load-displacement curve and, hence, greater energy dissipation. This may be a result of the fact that the rectangular HSC column confined in thin steel shell was more ductile than the ordinary HSC column.

Effect of axial load

Test column results that could be compared for effect of axial load level are shown in

Table 6. In the case of ordinarily reinforced concrete columns, comparisons of Fig. 4(a) and 4(b) show that the maximum loads were increased by about 8.8 percent when the axial stress of HSC column was up to $0.3f_c'$.

For steel tubed column with D/t=125, it was observed from Fig. 5(b) and 5(c) that compared with TC-B0 column, the maximum loads of TC-B3 increased by about 31.3 percent.

Table 5 Effect of transverse reinforcing methods

Series	Test Specimens	f_c' MPa	Δ_y mm	Δ_{max} mm	μ Δ_{max}/Δ_y	Drift index %	V_{max} KN
RC	RC-3	46.4	3.7	18.5	5	1.85	200.6
TC	TC-B3	44.7	4.0	20.0	5	2.06	265.5

Table 6 Effect of axial load level

Series	Test Specimens	f_c' MPa	Axial load level(%)	V_{max} KN	μ Δ_{max}/Δ_y
RC	RC-0	46.4	0	184.3	9
	RC-3	46.4	30	200.59	5
TC	TC-B0	44.7	0	196.18	9
	TC-B3	44.7	30	265.49	5

Table 7 Effect of width-thickness ratio

Test Specimens	f_c' MPa	Δ_y mm	Δ_{max} mm	μ Δ_{max}/Δ_y	Drift index %	V_{max} KN
TC-A3	44.7	3.3	19.7	5.97	1.97	246.5
TC-B3		4.0	40.2	10.05	4.02	265.5

Table 8 Effect of columns with ribbed and non-ribbed steel

Test Specimens	f_c' MPa	Δ_y mm	Δ_{max} mm	μ Δ_{max}/Δ_y	Drift index (%)	V_{max} KN
TC-B3(non-ribbed)	44.7	3.3	40.2	10.05	4.02	265.49
TC-B3R(ribbed)	48.5	4.5	53.86	11.97	5.39	284.02

Specimens without axial load did not exhibit noticeable strength deterioration and experienced very little damage, even at lateral drift level beyond 4 percent (displacement ductilities of approximately 9 and greater).

The specimens subjected to higher axial loads experienced noticeable strength deterioration at displacement ductilities (μ) beyond 4. Series-RC results indicate that increasing axial load level from 0 to 30 percent reduced μmax by 44 percent for HSC columns confined in hoops. This behavior was also observed from results of Series-TC specimens.

Effect of width-thickness ratio

The comparison of two TC-A3 and TC-B3 columns is shown in Fig. 5(a) and 5(c). Column TC-A3 had a D/t ratio of 250, while column TC-B3 had a D/t ratio of 125. The steel tube on TC-A3 buckled during the first cycle to $\mu=\pm4$ and fractured during the first cycle to $\mu=\pm6$. In the case of TC-B3, the hysteresis curves shown in Fig. 5(c) indicate stable behavior with good energy-dissipation characteristics. Buckling of the steel tube began at $\mu=\pm9$.

However, fracture of this specimen did not occur. From Fig. 6, the load-carrying capacity of column TC-B3 prior to fracture of the steel is higher than that of TC-A3. Furthermore, TC-B3 exhibited greater displacement ductility while still maintaining its load-carrying

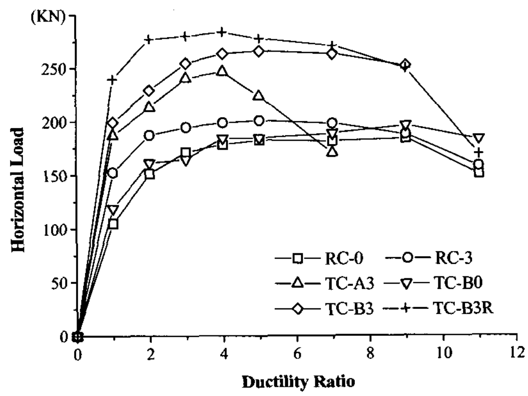


Fig. 6 Envelope of horizontal load-horizontal displacement

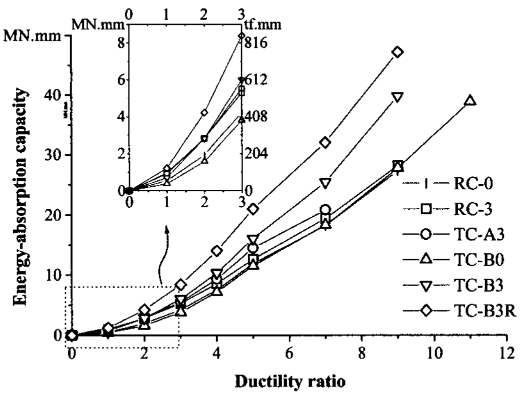


Fig. 7 Energy absorption capacities

capabilities than TC-A3 did. The maximum load of the latter was about 7.71 percent higher than that of the former. It can be seen that ductility factor decreased as the width-thickness ratio increased from 125 to 250 in the case of columns under the level of $0.3f_c'$ axial stress.

One reason for using steel tube for transverse reinforcement in HSC columns confined in thin steel tube is to allow lowering construction costs and larger amount of shear reinforcement. However, one should be very careful in using this approach. This point could be explained by comparing the behavior of specimens RC-03, TC-A3 and TC-B3, in which the amount of transverse reinforcement was 0.73, 1.60, and 3.20, respectively.

From the Table 4, the load-carrying capacity of TC-A3 and TC-B3 specimens prior to fracture is higher than that of RC-03. In case of ductile behavior, RC-03 and TC-B3 behave in very ductile manner without any degradation of lateral load capacity even after ultimate strength. However, TC-A3 specimen with a *D/t* ratio of 250 shows poor ductile behavior with significant degradation of lateral load capacity after ultimate strength.

Effect of columns with ribbed and non-ribbed steel

Column TC-B3 can be compared with column TC-B3R, equipped with a ribbed steel. The purpose of the ribs is to enhance the bond between the concrete core and the steel, thereby increasing the composite action and strength of the column.

The load-displacement responses for each of these columns are shown in Fig. 6. The figure shows that the mechanical behavior of the tubed columns without ribs is quite similar to that of the tubed columns with ribs at low values of lateral displacement.

At low values of lateral displacement, the frictional bond that exists between the non-ribbed, as well as the ribbed, steel tube and concrete core is main component that affects the strain distributions.

Hence, there was not a significant increase in bond between the concrete core and the steel tube.

At higher values of displacement, however, the ribs appear to be effective, probably as a result of a breakdown in the frictional bond.

TC-B3R experienced less degradation in strength when compared to TC-B3 was for $\mu <$ ± 9. The ultimate strength of TC-B3 degraded by 5 percent, while the strength of TC-B3R degraded only by 1 percent.

4.3 Nominal Moment Capacity

Two major questions must be addressed when calculating flexural strength of HSC columns confined in steel tubes. First, does the steel tube act as longitudinal reinforcement? Second, does the rectangular stress block described in the code apply to HSC columns confined in steel tube?

If these attributes are to be effective when gradients in moments, shear, and changes in the proportions of axial forces distributed between the steel and concrete exist, then shear stress transfer between the tube and the concrete must occur. This transfer can be accommodated by bond stress between the steel and concrete.

Past research[7] has shown that columns confined in steel tube do not have good natural bond stress capacity for steel tubes with large D/t ratios. Although the bond stress is larger when subjected to both bending and axial load because of the mechanical interlocking of the concrete and steel, the influence of D/t on bond stress are not completely understood. In this paper, therefore, it was assumed that the thin steel shell does not act as longitudinal reinforcement.

Maximum measured moment $M_{max.exp}$ for all test columns is given in Table 9. For ordinary HSC columns, the ratio of $M_{max.exp}$ over MKCI, nominal moment capacity predicted by KCI provisions using strength reduction factors of ϕ = 1.0, is consistently greater than one with approximately a 3 percent margin of safety. However, the same ratio for test columns confined in thin steel shell is larger than 1, ranging from 1.08 to 1.21. This behavior warrants reconsideration of the procedures given in KCI code.

The following discussion offers an alternative procedure that appears to conservatively predict the nominal moment capacity of these columns confined in thin steel shell. Available test data[8] indicate that typical stress-strain curves in compression for columns confined thin steel shell are characterized by an descending portion that is no degradation of strength.

Considering this information, it may be more appropriate to use fully rectangular compressive stress blocks with properties shown in Fig.8. It can be shown that the rectangular compressive block would have the following properties: intensity of compressive stress would be $1.0f_c'$ rather than $0.85f_c'$, the value currently specified in KCI; and the depth of the rectangular compression block would be 1.0 times the depth of the neutral axis, corresponding approximately to current KCI code requirements.

Table 9 Maximum measured and calculated moment capacities

Test Specimens	$M_{max.exp}$ experimental	M_{KCI}	$M_{KCI.modify}$	M_{max}/M_{KCI}	$M_{max}/M_{KCI.modifv}$
	KN.m	KN.m	KN.m		
RC-3	119.8	116.2	126.8	1.03	0.94
TC-A3	124.7	114.8	125.1	1.08	1.00
TC-B3	140.8	117.9	128.6	1.19	1.09
TC-B3R	143.7	117.9	128.6	1.21	1.12

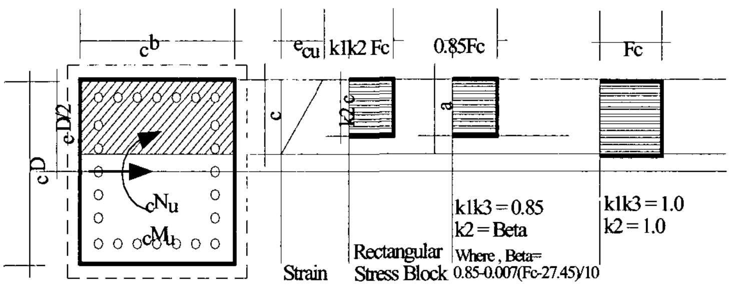


Fig. 8 Assumed stress blocks of concrete

Table 9 gives nominal moment capacity calculated using the modified in compressive stress blocks for test columns confined thin steel tube with f_c'. This approach results in conservative predictions of test column flexural capacity, as the ratios of measured over predicted values range from 1.0 to 1.12.

5. Conclusions

Based on the results of this study, the following conclusions and recommendations are presented.

(1) HSC columns confined in thin steel shell appear to be an acceptable alternative to conventionally reinforced concrete columns. The columns of this study satisfied current seismic performance criteria, and they were shown to have similar or higher energy-dissipation ratios than comparable ordinarily reinforced concrete columns. Therefore, the very strong and ductile reinforced concrete short columns can be made by steel tube transverse reinforcing method. The strong reinforced concrete frame composed of wall girders and short columns is expected to be made as ductile by steel tube transverse reinforcing method.

(2) The column utilizing a shell with a D/t ratio 250 has lower ultimate strength values and dissipated less energy rather than similar ordinary HSC columns. However, the column with the D/t ratio of 125 had greater ductility and did not experience fracture in the steel shell. Therefore, it is clear from this results that for HSC columns confined in thin steel shell with $D/t \leq 125$, the seismic behavior was higher than that for the ordinarily HSC columns.

(3) Until further research is conducted, it is suggested that the following changes be incorporated in KCI provisions when calculating the nominal moment capacity of HSC columns confined in thin steel shell with D/t exceeding minimum thickness for steel tubes in round composite columns. For D/t exceeding minimum thickness, it is recommend that the thin steel shell showed not be considered as longitudinal reinforcement and the stress intensity of the equivalent rectangular compressive block and the depth of the rectangular compression block be enlarged to $1.0f_c'$ from $0.85f_c'$.

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