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Structural Behaviour of Concrete-Filled Steel Box Sections

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Summary

An experimental study on the behavior of concrete-filled steel box stub columns was performed. Steel box columns with and without stiffeners were also tested under concentric compressive load to failure. The result of the test showed composite box columns had high ductility as well as high strength due to mutual confinement between concrete and steel plate. In addition, simple formulas for design of composite column were proposed based on the test results.

1. Introduction

The concrete-filled steel box column has a lot of advantages such as high strength, high ductility and large energy absorption capacity, so that it has become increasingly popular in various kinds of structures. Especially, its excellent earthquake-resistant properties have proved recently in other countries, hence it is strongly needed to investigate the behavior of composite columns.

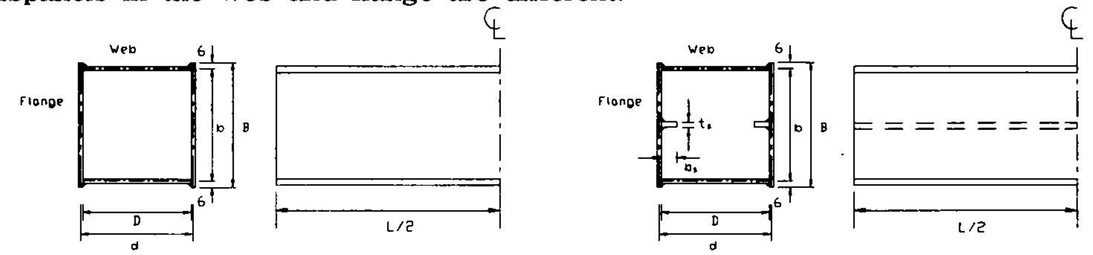
2. Test Specimen

2.1 Material Properties

The tensile coupon test was performed to determine the mechanical properties of the steel used(SS400 : nominal yield stress σ_y = 2400 kg/㎠). The results given in Table 1. show higher yield and ultimate strength than the nominal strength because of the welding and cutting from test specimens. To determine the compressive strength of the concrete, 15 cylinders (10cm diameter x 20cm height) were cast from the same concrete used inside the concrete-filled column. The cylinders were made with a water/cement ratio of 50% from ordinary portland cement and well graded aggregate(maximum size = 19mm) and were cured for 28 days until the column specimens were tested. The average values obtained 15 cylinders are listed in Table 1.

2.2 Shapes, Labeling and Size

Six concrete-filled steel box columns and seven steel box columns with and without longitudinal stiffeners were tested to compare the ultimate strength, ductility and postbuckling strength. In the case of the stiffened steel box column, specimens were classified again as spot welding and fillet welding to determine the effect of welding. In Table 2, the numerals following the letter US, UC, SS are related to the value of equivalent width-thickness ratio parameter R. Buckling coefficient k in the case of the SS series was decided by the numerical analysis using Bfplate(Lau and Hancock 1986)[1] as 5.45 because the number of subpanels in the web and flange are different.



(a) unstiffened section (b) stiffened section

Fig. 1 Test specimens

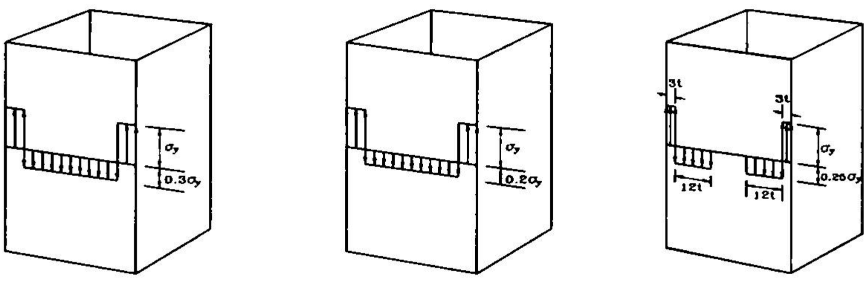
Table 1. Material properties

Steel	E_s (kg/㎠)	ν	σ_y (kg/㎠)	ε_y	σ_u (kg/㎠)	ε_u
	2,057,000	0.3	3,200	0.001560	4,940	0.002401
Concrete	Days	E_c (kg/㎠)	f_c (kg/㎠)	*β x f_c (kg/㎠)	*β = 0.7 [2]	
	28	288000	307	259		

Table 2. Measured dimensions of test specimens

Unstiffened Steel box column	Specimen	b (cm)	t (cm)	A_s (㎠)	L (cm)	b/t	R (k=4.0)	
	US 9	13.0	0.32	17.3	39.0	40.6	0.84	
	US 12	17.5	0.32	22.3	52.5	54.7	1.14	
	US 15	22.0	0.3	26.94	66.0	73.3	1.43	
Unstiffened Concrete-filled box column	Specimen	b (cm)	t (cm)	A_c (㎠)	L (cm)	b/t	R (k=4.0)	
	UC 9	13.0	0.32	168	39.0	40.6	0.84	
	UC 12	17.5	0.32	306	52.5	54.7	1.14	
	UC 15	22.0	0.3	483	66.0	73.3	1.43	
Stiffened Steel box column	Specimen	b(cm)	b_s(cm)	L(cm)	t(cm)	t_s(cm)	A_s(㎠)	R(k=5.45)
	SS15 (2.5)	22	2.5	66	0.3	0.3	28.61	1.3
	SS15 (3.5)	22	3.5	66	0.3	0.3	29.2	1.3
	SS15 (4.5)P	22	4.5	66	0.3	0.3	29.8	1.3
	SS15 (4.5)F	22	4.5	66	0.3	0.3	29.8	1.3

2.3 Residual Stress



(a) RS 1 [2] (b) RS 2 [3] (c) RS 3

Fig. 2 Assumptions of residual stresses distribution

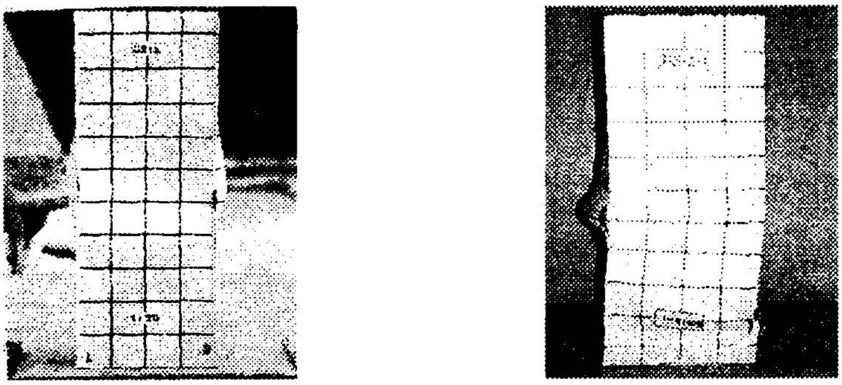
In this investigation, three types of residual stress were assumed as Fig.2 to conduct the inelastic buckling analyses and compare with the results of the test.

3. Test Results

3.1 Test Arrangements

The axial displacement was measured using four displacement transducers equipped at each edge of loading plate and the strain was measured with eight strain gages attached at the center of plates. To assure uniform compression and prevent the eccentricity, very thick loading plates(t=4cm) were attached at each end(top and bottom) of test specimens and preliminary tests were carried out within the elastic range by adjusting the loading plate, based on the measurements of strain and displacement. The loading process was paused at every step of 5 tons for a minute to determine the difference between static and dynamic load.

3.2 Failure Modes



(a) steel column (US12) (b) concrete-filled steel column (UC12)
Fig. 3 Buckling modes of test specimens

In the case of the steel box columns, it was observed that local buckling failure of the plate panels occurred before the maximum load was reached and local buckling shaped three half-waves along overall length of the specimens due to aspect ratio a/b=3.0 as shown in Fig. 3(a). A very symmetric buckling mode, at the two opposite faces of the specimens buckled inward and at the other two perpendicular faces buckled outward, against the axes of the cross section was observed at the central part of the specimens.

In the case of the 9 series concrete-filled columns, it was observed that local plate buckling occurred in one of the plates of the column just before maximum load was reached. As an increment of width-thickness ratio, the occurrence of local buckling came earlier and that could be seen at steel box columns. Since the failure of concrete-filled columns was controlled by the fracture of concrete, buckling of plates in concrete-filled columns showed asymmetric buckling mode against the axes of the cross section as shown in Fig. 3(b). All steel panels buckled outward because the buckling of the steel plate toward inside was prevented by the filled-in concrete. After the local buckling of the plates, deformation rapidly increased and cracks occurred in the weld.

3.3 Results and Design Curves

Table 3. shows the comparison between test results and inelastic buckling analysis using B3-Spline Finite Strip Method(Bfplate). In this comparison, test result of US9 indicates in good agreement with RS1 and in the case of US12, US15, good agree with RS3. It was supposed that US9 was much effected by welding because width b is relatively small.

Table 3. Test results of unstiffened steel box columns

Specimen	P_u (ton)	σ_u (kg/cm^2)	σ_b (kg/cm^2)				σ_u/σ_y		σ_b/σ_y	
			Test	Inelastic analysis			Test	Prediction (Eq. 2.6)	Test	Inelastic analysis
				RS 1	RS 2	RS 3				
US 9	52.25	3034	2601	2692	2945	3198	0.94	0.90	0.81	0.84
US 12	60.40	2709	2317	1781	2014	2398	0.84	0.73	0.72	0.75
US 15	54.68	2030	1305	668	911	1326	0.63	0.62	0.41	0.41

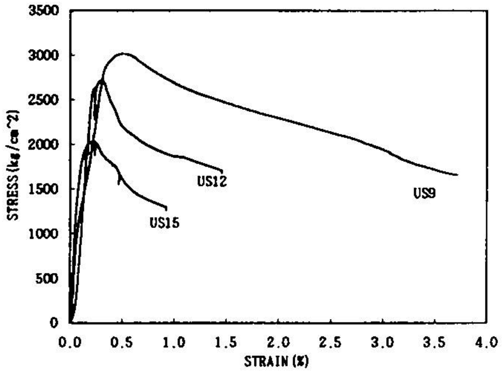


Fig. 4 Stress-strain curves (unstiffened steel column)

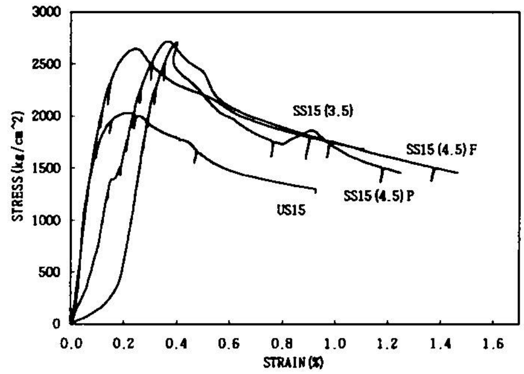


Fig. 5 Stress-strain curves (stiffened steel column : SS15)

Stress-strain curves of unstiffened steel columns were shown in Fig.4. It is noted that the ultimate and buckling stress of steel box columns were reduced as the width-thickness ratio of the section is increased. In Fig. 5, the stress-strain relation of SS15(4.5)P (spot welding) after ultimate load shows unstable behavior, that was supposed to be caused by separation of the stiffeners and plates as deformation getting serious after peak load. The ultimate strengths of SS15(4.5)P and SS15(4.5)F (fillet welding) were nearly same and somewhat higher than SS15(3.5). The comparison between US15 and SS15 shows that ultimate load of stiffened steel column higher than unstiffened steel column's about 40~50% and about 30~40% in the buckling stress. This means that longitudinal stiffeners which have enough stiffness to resist the distortional buckling of the section can be very effectively used as a column member due to the increase of buckling and postbuckling strength reserve.

Table 4. Test results of stiffened steel box columns

Specimen	P_u (ton)	σ_u (kg/cm^2)	σ_b(kg/cm^2)				σ_u/σ_y		σ_b/σ_y	
			Test	Inelastic analysis			Test	Prediction (Eq. 2.7)	Test	Inelastic analysis
				RS 1	RS 2	RS 3				
SS15 (3.5)	77	2637	1660	1241	1509	1627	0.82	0.73	0.52	0.51
SS15(4.5)P	81	2718	1720	-	-	-	0.85	0.72	0.54	-
SS15(4.5)F	81	2718	1715	1242	1510	1629	0.85	0.72	0.54	0.51

A similar design formula to that used by Chajes et al.(1966)[4] for inelastic flexural-torsional buckling stress was adopted for determining the inelastic local buckling stress of the tubular columns. The proposed formula(called design proposal 1) is given by

$$\sigma_b = \sigma_{be} \qquad (\sigma_{be} \leq 0.5\sigma_y)$$

$$\sigma_b = \sigma_y\left(1 - \frac{\sigma_y}{4\sigma_{be}}\right) \qquad (\sigma_{be} > 0.5\sigma_y)$$

---------------- (Eq. 1)

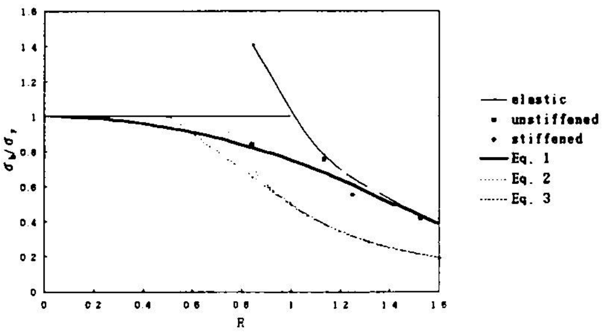


Fig. 6 Comparison of test results with design proposal 1

A comparison of design proposal 1(Eq. 1) with $\frac{\sigma_b}{\sigma_y} = \frac{0.5}{R^2}$ for unstiffened plate (Eq. 2) and $\frac{\sigma_b}{\sigma_y} = 1.5 - R$ $(0.5 < R \leq 1.0)$ or $\frac{\sigma_b}{\sigma_y} = \frac{0.5}{R^2}$ $(1.0 < R)$ for stiffened plate (Eq. 3) in Korean Standard Specifications for Highway Bridges[5] and test results is shown in Fig. 6.

In the case of steel box column, an alternative design approach(called design proposal 2) using ultimate stress is also proposed as $\frac{\sigma_u}{\sigma_y} = \frac{0.8}{R^{0.7}}$ $(R > 0.73)$ for unstiffened plate(Eq. 4) or $\frac{\sigma_u}{\sigma_y} = \frac{0.8}{R^{0.4}}$ $(R > 0.57)$ for stiffened plate(Eq. 5), in this paper. It is based on the idea that Eqs. have no regard of postbuckling strength reserve. The second proposal and test results are plotted in Fig. 7.

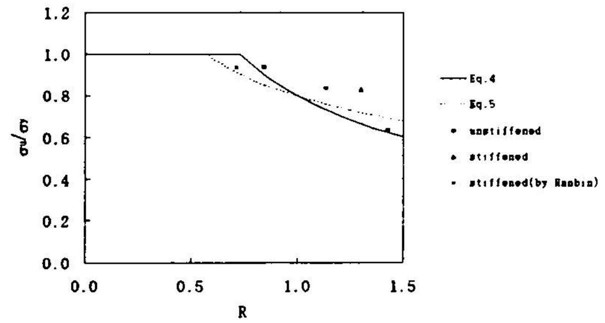


Fig. 7 Comparison of test results with design proposal 2

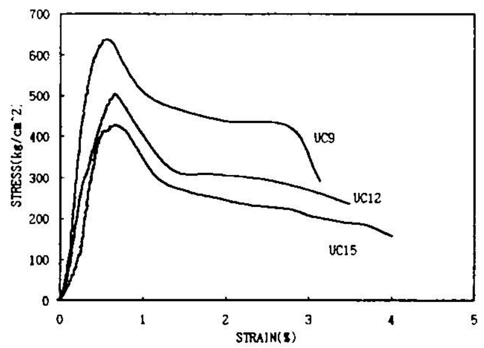


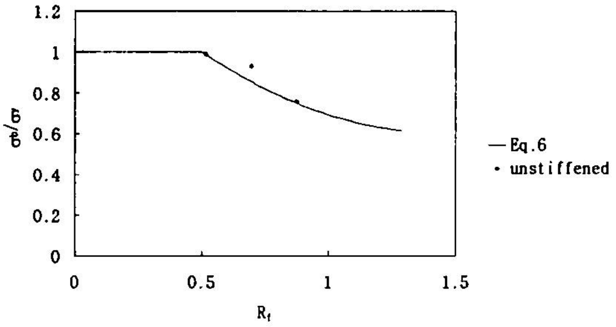
Fig. 8 Stress-Strain Curve (Concrete-Filled Columns)

The behavior of the UC series column is different from those of hollow steel tubular columns because of filled-in concrete. Sudden local buckling occurred after peak load due to brittle fracture of concrete at UC series. The concrete-

Table 5. Test results of unstiffened concrete-filled columns

Specimen	P_u (ton)	P_b (ton)	$P_y = \sigma_y A_s + \beta f_c A_c$ (ton)	σ_b/σ_y
US 9	118.5	98	98.9	0.99
US 12	166	140	150.6	0.93
US 15	246	160	211.3	0.76

filled section shows good structural performance such as higher strength and ductility than the hollow steel column, since steel and concrete confined each other. Eq. 6 proposed by Nakai et al.[6] and the test results of concrete-filled columns were plotted in Fig. 9.



$$\frac{\sigma_b}{\sigma_y} = 0.433\,(R_f - 0.5)^2 - 0.831\,(R_f - 0.5) + 1.0 \quad (0.5 < R_f < 1.3) \qquad \text{(Eq. 6)}$$

R_f : substitution for k=10.67 in R (plate with fixed loading-fixed unloading edges)

Fig. 9 Comparison of test results with Eq. 6

4. CONCLUSIONS

A series of compression tests on steel columns with and without stiffeners and concrete-filled columns has been performed. In the case of the column with the stiffener, spot welding has lots of advantages in construction with convenience and decrease of residual stress. The use of the longitutional stiffener with adequate stiffness is more economic manners than the way to increase the thickness of the panel. Concrete-filled column showed much higher ductility as well as strength than hollow steel columns. As increment of width-thickness ratio parameter R, although local buckling occurred, the concrete-filled columns showed considerable postbuckling strength reserve before fracture.

APPENDIX I. Reference

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APPENDIX II. Notation

E_s, E_c = Young's modulus of steel and concrete, respectively;
σ_y = yield stress; σ_u = ultimate stress; ε_y = yield strain of steel;
f_c = compressive strength of concrete; ν = Poisson's ratio of steel;
L = column length; b = plate width; t = plate thickness;
A_s, A_c = crosssectional area of steel and concrete, respectively;
R, R_f = plate width-thickness ratio parameters; $R = \sqrt{\frac{\sigma_y}{\sigma_{cr}}} = \frac{b}{t}\sqrt{\frac{12(1-\nu^2)}{\pi^2 k}}\sqrt{\frac{\sigma_y}{E}}$
k = buckling coefficient($4n^2$); n = number of subpanels in each plate panel;
σ_b = inelastic buckling stress; σ_{be} = elastic buckling stress;