Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 999 (1997)

Artikel: Structural behaviour of concrete-filled steel box sections

Autor: Song, Jun Yeup / Kwon, Young Bong

DOI: https://doi.org/10.5169/seals-1067

Nutzungsbedingungen

Die ETH-Bibliothek ist die Anbieterin der digitalisierten Zeitschriften auf E-Periodica. Sie besitzt keine Urheberrechte an den Zeitschriften und ist nicht verantwortlich für deren Inhalte. Die Rechte liegen in der Regel bei den Herausgebern beziehungsweise den externen Rechteinhabern. Das Veröffentlichen von Bildern in Print- und Online-Publikationen sowie auf Social Media-Kanälen oder Webseiten ist nur mit vorheriger Genehmigung der Rechteinhaber erlaubt. Mehr erfahren

Conditions d'utilisation

L'ETH Library est le fournisseur des revues numérisées. Elle ne détient aucun droit d'auteur sur les revues et n'est pas responsable de leur contenu. En règle générale, les droits sont détenus par les éditeurs ou les détenteurs de droits externes. La reproduction d'images dans des publications imprimées ou en ligne ainsi que sur des canaux de médias sociaux ou des sites web n'est autorisée qu'avec l'accord préalable des détenteurs des droits. En savoir plus

Terms of use

The ETH Library is the provider of the digitised journals. It does not own any copyrights to the journals and is not responsible for their content. The rights usually lie with the publishers or the external rights holders. Publishing images in print and online publications, as well as on social media channels or websites, is only permitted with the prior consent of the rights holders. Find out more

Download PDF: 01.12.2025

ETH-Bibliothek Zürich, E-Periodica, https://www.e-periodica.ch

Structural Behaviour of Concrete-Filled Steel Box Sections

Jun Yeup SONG
Post Graduate Student
Yeungnam University
Taegu, S/Korea

Jun Yeup Song, born 1968 received M.A. at Youngnam University in S/Korea

Young Bong KWON Assistant Professor Yeungnam University Taegu, S/Korea

Young Bong Kwon, born 1954 received M.A. at Seoul National Univ. 1983 and Ph.D. at Univ. of Sydney 1992

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/cm). 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.

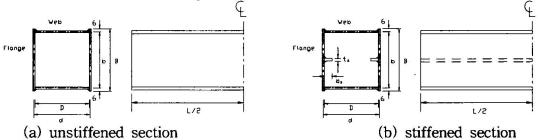


Fig. 1 Test specimens

Table 1. Material properties

	3 31001	6 8 16						TO TO THE RESERVE	
Steel	$E_{\rm s}$ (kg/c	्रापं)	ν		$\sigma_{\rm s}$ (kg/c	ш')	$\boldsymbol{\epsilon}_{\mathrm{y}}$	σ_{u} (kg/cm/)	ε _u
Sieei	2,057,00	Ю	0.3		3,200		0.001560	4,940	0.002401
Concrete	Days	E_{c}	(kg/cm/)	f_c	(kg/cm²)	*β	$x f_c (kg/cw)$	*B = C	7 [2]
Concrete	28	2	288000		307		259) p - 0	[4]

Table 2. Measured dimensions of test specimens

Unstiffened	Specimen	b (cm)	t (cm)	A, (c	चारे)	L	(cm)		b/t	R	(k=4.0)
C. 11	US 9	13.0	0.32		17.	3	3	9.0		40.6		0.84
Steel box	US 12	17.5	0.32		22.	3	5	2.5		54.7		1.14
column	US 15	22.0	0.3		26.9)4	6	6.0		73.3		1.43
Unstiffened	Specimen	b (cm)	t (cm)	A_c (cm')	L	(cm)		b/t	R	(k=4.0)
C	UC 9	13.0	0.32		168	3	3	9.0		40.6		0.84
Concrete-filled	UC 12	17.5	0.32		306	3	5	2.5		54.7		1.14
box column	UC 15	22.0	0.3		483	3	6	6.0		73.3		1.43
Circ	Specimen	b(cm)	$b_s(cm)$	L	(cm)	t(c	m)	$t_s(cn$	a)	$A_{\mathfrak{s}}(\operatorname{cnr})$	R	(k=5.45)
Stiffened	SS15 (2.5)	22	2.5		66	0.	3	0.3		28.61		1.3
Steel box	SS15 (3.5)	22	3.5		66	0.	3	0.3		29.2		1.3
column	SS15 (4.5)P	22	4.5		66	0.	3	0.3		29.8		1.3
Column	SS15 (4.5)F	22	4.5		66	0.	3	0.3		29.8		1.3

2.3 Residual Stress

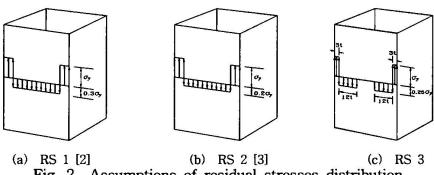


Fig. 2 Assumptions of residual stresses distribution

In this investigation, three types of residual stress were assumed as Fig.2 to

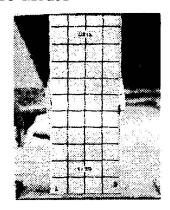
conduct the inelastic buckling analyses and compare with the results of the test.

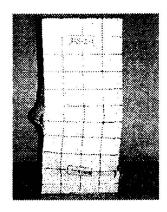
3. Test Results

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.

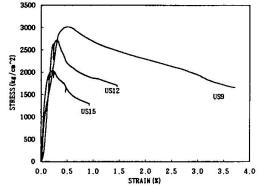
In the case of the 9 series concrete-filled columns, it was observed that local plate buckling occurred in one of the plates of the column just before maximum load was reached. As an increment of width-thickness ratio, the occurrence of local buckling came earlier and that could be seen at steel box 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	ot	unstiffened	steel	box	colu	mns
	1			25		-,		

	$P_{\mathbf{u}}$	σ,,		σ _b (kg	(cm²)	***************************************		σ_u/σ_y	σ_b/σ_y		
Specimen	0.00	(kg/cm ²)	Tost		stic an		Test	Prediction	Test	Inelastic	
	(WII)	(Kg/CIII)	1631	RS 1	RS 2	RS 3	Test	(Eq. 2.6)	Test	analysis	
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	



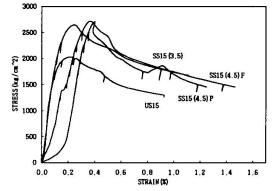


Fig. 4 Stress-strain curves (unstiffened steel column)

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 $40\sim50\%$ and about $30\sim40\%$ 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

	Pu	$\sigma_{ m u}$		$\sigma_b(\mathbf{k}\mathbf{g})$	g/cm^2			σ_u/σ_y		$\sigma_{\rm b}/\sigma_{\rm y}$	
Specimen	(ton)	(kg/cm ²)	Toot	Inela	stic ana	alysis	Test	Prediction	Test	Inelastic	
	(1011)	(Kg/CIII)	rest	RS 1	RS 2	RS 3	rest	(Eq. 2.7)	rest	analysis	
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

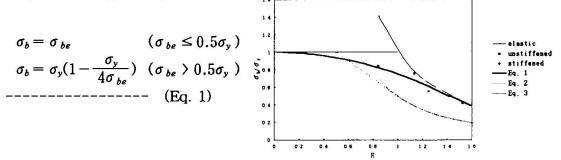


Fig. 6 Comparison of test results with design proposal 1

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 $\lt R \le 1.0$) or $\frac{\sigma_b}{\sigma_y} = \frac{0.5}{R^2}$ (1.0 $\lt R$) 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.

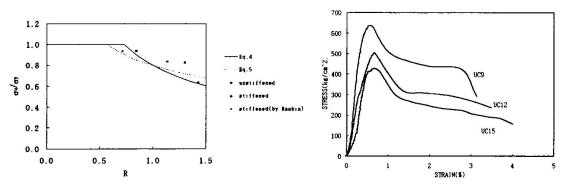


Fig. 7 Comparison of test results with design proposal 2

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)	$P_{y} = \sigma_{y} A_{s} + \beta f_{c} A_{c} \text{ (ton)}$	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.

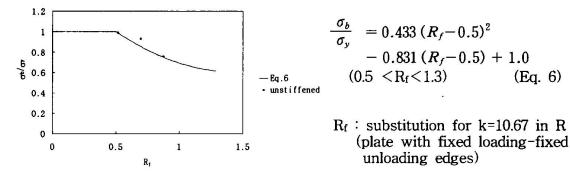


Fig. 9 Comparison of test results with Eq. 6

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

[1] Lau, S.C.W. and Hancook, G.J. "Buckling of Thin Flat-Walled Structures by a Spline Finite Strip Method", Thin-Walled Structures, Vol.4, No.4, pp.269-294, 1986
[2] Ge, Hanbird, and Usami, T. "Development of Earthquake-Resistant Ultimate Strength Design Method for Concrete-Filled Steel Structures", Nagoya University, March, 1994 [3] Kim, W.S., Kwon, W.B., "A study on the Residual stresses in Rolled and Built-up Sections", The conference Report of Korean society of steel construction, pp.95-103, 1994 [4] Chajes, A., Fang, P.J. and Winter, G., "Torsional-Flexural Buckling, Elastic and Inelastic, of Cold Formed Thin-Walled Columns", Research Bulletin, No.66-1, School of Civil Engineering, Cornell University, Ithaca, New York, 1966 [5] "Korean Standard Specifications for Highway Bridges", 1996
[6] Nakai, H., Kitada, T., and Yoshikawa, O. "A Design Method of Steel Plate Element in Concrete-Filled Square Steel Tubular Columns" Proc. of JSCE, I-3, pp.405-413, 1985

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_{i} , A_{c} = crosssectional area of steel and concrete, respectively;

R, R_f = plate width-thickness ratio parameters; $R = \sqrt{\frac{\sigma_y}{\sigma_{cr}}} = \frac{b}{t} \sqrt{\frac{12(1-v^2)}{\pi^2 k}} \sqrt{\frac{\sigma_y}{E}}$

 $k = buckling coefficient(4n^2); n = number of subpanels in each plate panel;$

 σ_b = inelastic buckling stress; σ_{be} = elastic buckling stress;