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Experimental investigation of inelastic buckling of built-up steel columns

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Abstract. This paper experimentally investigated the buckling capacity of built-up steel columns mainly, Cruciform Columns (CC) and Side-to-Side (SS) columns fabricated from two Universal Beam (UB) sections. A series of nine experimental tests comprised of three UB sections, three CC sections and three SS sections with different lengths were tested to failure to measure the ultimate axial capacity of each column section. The lengths used for each category of columns were 1.8, 2.0, and 2.2 m with slenderness ratios ranging from 39–105. The measured buckling loads of the tested specimens were compared with the predicted ultimate axial capacity using Eurocode 3, AISC LRFD, and BS 5959-1. It was observed that the failure modes of the specimens included flexural buckling, local buckling and flexural-torsional buckling. The results showed that the ultimate axial capacity of the tested cruciform and side-by-side columns were higher than the code predicted design values by up to 20%, with AISC LRFD design values being the least conservative and the Eurocode 3 design values being the most conservative. This study has concluded that cruciform column and side-to-side welded flange columns using universal beam sections are efficient built-up sections that have larger ultimate axial load capacity, larger stiffness with saving in the weight of steel used compared to its equivalent universal beam counterpart.

Keywords: built-up sections; steel columns; cruciform; side-to-side columns; buckling; compression axial load.

1. Introduction

Columns are usually referred to as vertical compression members supporting roofs and floors in structural frames. In many cases, these members are subjected to both axial and bending effect. An Axially loaded column in compression buckles about the axis of least moment of inertia. Steel columns in steel structures are generally slender. The most common mode of failure of such columns is likely to be elastic or inelastic buckling. Many steel design standards provide comprehensive design aids for analyzing and designing standard steel columns. For built-up sections, such design aids is not readily available and many design codes do not provide specific design provisions for built-up compression members. As the result, many investigators have studied the buckling behavior of built-up steel compression members and compared the experimental compressive strengths with the code predicted design values to make recommendation on design provisions of built-up columns.

Uniaxial compression tests of built-up sections made of plates or drawn from cold-formed steel

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sheets have also been carried out by many investigators. Weng *et al.* (1992) developed a column-strength curve for predicting the maximum strength of cold-formed steel columns based on experimental results. In their study, the effect of residual stress on local buckling strength as well as the effect of initial imperfection were all taken into consideration directly. Migita *et al.* (1992) carried out experimental investigation on polygonal section steel columns and derived a formula to predict their local buckling strength and interaction strength between local and overall buckling strengths based on the experimental results.

Liu *et al.* (2003) and Young *et al.* (2003) carried out series of compression tests on cold-formed stainless steel square hollow sections of various lengths and with fixed-ends. They compared their experimental results with several code design values and they concluded that code design values generally underestimate the test columns capacities.

Kwon *et al.* (2007) conducted compression tests on welded built-up H-section and channel section columns fabricated from a mild steel plate. They experimentally and analytically investigated the ultimate strength and performance of these columns and concluded that the interaction between local and overall buckling has significant effect on the ultimate strength of welded section columns.

Young (2008) carried out several experimental and numerical investigations of cold-formed steel columns of different open and close cross-section shapes. The axial capacities of the tested columns were compared with the design strengths calculated using various steel design codes of cold-formed steel structures and several recommendations were made.

Megnounif *et al.* (2008) proposed a procedure for predicting the elastic buckling strength of built-up, cold-formed steel columns based on the effective width method and the direct strength method. They compared their results with experimental data on plain and lipped, built-up columns and showed that effective width approach was more accurate than the Direct Design Method.

Gao *et al.* (2009) carried out experimental compression tests on thin-walled box-section stub columns fabricated from high strength steel with pinned-connected ends. The experimental results were compared with those predicted by the AISI (1996) and they concluded that the code predicted values are too conservative.

Tahir *et al.* (2009) investigated the axial capacity of cruciform columns made of universal beams. They showed that the capacity of the tested columns is higher than the BS 5950-1 and Eurocode 3 predicted design values with ratios of up to 21% and 47%, respectively. They also concluded that cruciform columns are stiffer and economical in terms of strength to weight ratio as compared to two universal beams. Liu *et al.* (2009) and Lue *et al.* (2006) carried experimental investigation on built-up sections fabricated from hot-rolled standard channel (back-to-back welded, back-to-back bolted and face-to-face welded) sections with the aim of verifying the AISC (2005) and other prominent steel design codes slenderness ratio formulas for built-up compressive members. They concluded that the 0.75 slenderness rule, which states that the slenderness ratio of a component element of a built-up member should not exceed 75% of the governing slenderness ratio of that member, is justified according to the experimental investigation.

Several researchers analytically investigated the buckling behavior of cruciform compression members (Chen and Trahair 1994, Dabrowski 1988, Damkilde 1985). As indicated, Tahir *et al.* (2009) experimentally investigated the buckling behavior of four cruciform columns of the same length and made of different Universal Beam sections with slenderness ratio in the range of 26–35.

Chang *et al.* (2010) carried out experimental and theoretical investigations on the response and collapse of 310 stainless steel tubes with different diameter-to-thickness ratios subjected to cyclic bending. They proved that the theoretical formulations effectively simulate the experimental data. In this paper, three cruciform sections of different lengths and made of the same Universal Beams were investigated, in addition to three side-to side and three universal beams sections. The tested columns have slenderness ratio in the range of 39-105. In addition, BS 5950-1 (2000), EC3 (1992), and AISC LRFD (2005) predictions for the ultimate load capacity of the tested columns has also been computed and compared with the experimental results.

2. Research Objectives

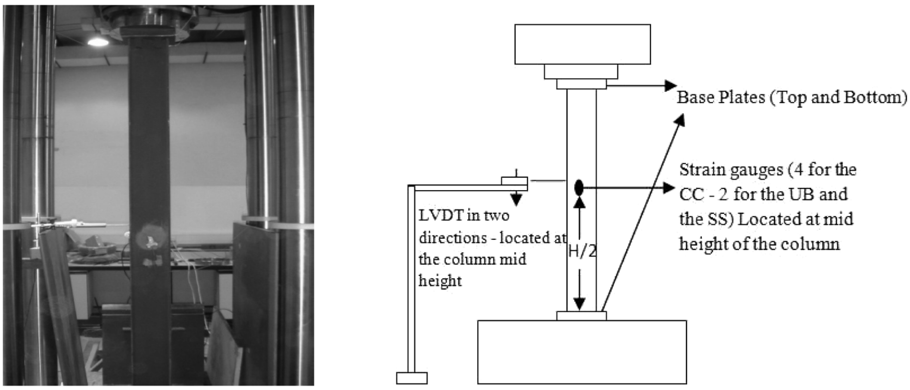
The objectives of this research are to:

1) Investigate experimentally the axial capacity of cruciform steel column sections (CC) and side to side welded flange column sections (SS) with three different lengths.

2) Compare the experimental results for the ultimate load capacity with the predicted values computed according to the guidelines of EC3 (1995), BS 5950-1(2000) and AISC LRFD (2005) codes, respectively.

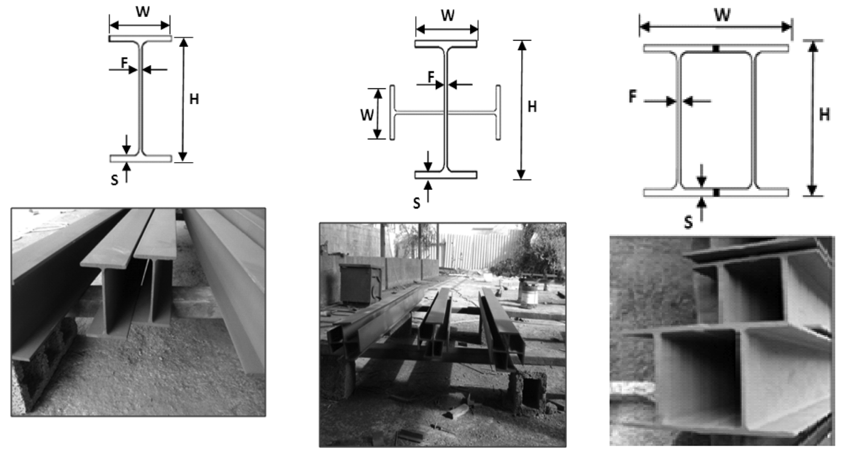
3. Experimental Program

The test program presented in this paper provides experimental ultimate loads and failure modes of three different steel sections tested under pure axial compression. A total of nine specimens were tested using a Universal Testing Machine (UTM) shown in Fig. 1(a) that has a capacity of 2500 kN. Fig. 1(b) displays a schematic of the experimental setup. The samples were shaped into three different cross sections with three different lengths for each section. The investigated cross sections are shown in Fig. 2 consisting of the regular universal beam section (UB) shown in Fig. 2(a), cruciform column section (CC) shown in Fig. 2(b) and side-to-side welded flange column section (SS) shown in Fig. 2(c), respectively. Table l lists the cross sectional properties and dimensions of the tested specimens. The CC sections were fabricated by cutting one universal beam at the centre of the web along the longitudinal direction into two T-sections, and then welding it to another UB of the same cross section using fillet welding from the middle of the web making sure that the length is the same in both directions. The SS



(a) Universal Testing Machine (b) Schematic for the column test setup

Fig. 1 Universal testing setup



(a) Universal Beam (UB) (b) Cruciform Columns (CC) (c) Side-to-Side welded flanges (SS)

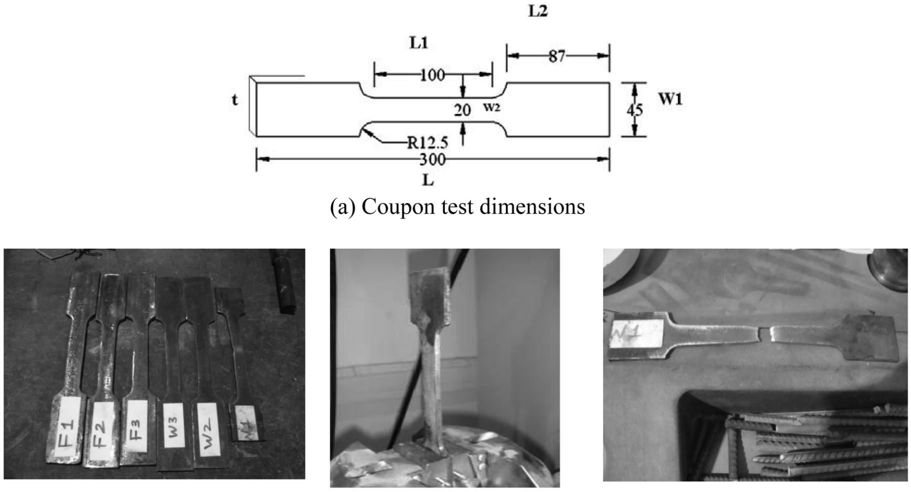
Fig. 2 Types of column sections

Table 1 Cross-sectional properties of the tested specimens

Designation	Description	L (mm)	H (mm)	W (mm)	S (mm)	F (mm)	A (mm^2)	I_x ($\times 10^4$ mm^4)	I_y ($\times 10^4$ mm^4)
UB 1	152×89×16	1800	152	88.7	4.5	7.7	2030	834	89.6
CC 1	152×89×32	1800	152	88.7	4.5	7.7	4060	923.7	923.7
SS 1	152×178×32	1800	152	177.4	4.5	7.7	4060	1668	959.1
UB 2	152×89×16	2000	152	88.7	4.5	7.7	2030	834	89.6
CC 2	152×89×32	2000	152	88.7	4.5	7.7	4060	923.7	923.7
SS 2	152×178×32	2000	152	177.4	4.5	7.7	4060	1668	959.1
UB 3	152×89×16	2200	152	88.7	4.5	7.7	2030	834	89.6
CC 3	152×89×32	2200	152	88.7	4.5	7.7	4060	923.7	923.7
SS 3	152×178×32	2200	152	177.4	4.5	7.7	4060	1668	959.1

sections were fabricated by welding two UB sections side to side at the flanges along the longitudinal direction using butt welding**.** The three different cross sections of columns were cut into three different lengths of 1.8, 2.0 and 2.2 m. In order to have equal load distribution along the cross section, both ends of the columns were grinded and then supported by a 30 mm thick end plate as shown in Fig 1. In addition, the specimens were painted to prevent any types of corrosion. The columns test specimens were designated as UB, CC and SS for universal beam, cruciform and side to side cross sections followed by numerical number of 1, 2, and 3 representing columns heights of 1.8 m, 2 m, and 2.2 m, respectively.

Prior to testing the column specimens, a load of 100 kN was applied manually and then removed (unloaded) to reduce any residual stresses encountered in each specimen. Afterwards, strain gauges and LVDT were mounted at mid height of the column to measure the axial strain and lateral deflection of the column specimens. A compression axial load was then applied at a rate of 40 kN/min and continued



(a) Coupon test dimensions

(b) Coupon specimens (c) Configuration (d) Failure mode of specimens

Fig. 3 Coupon Tests

until failure of the specimens. In this study, failure of the tested specimens is identified when large deformations has occurred or at the onset when the applied axial load started to decrease significantly.

The material properties of all specimens were determined by carrying out longitudinal tensile coupon tests to identify the yield stress of the tested specimen materials and also to determine the elastic-plastic behavior of the material through the stress-strain curves. Six longitudinal coupon specimens, taken on the longitudinal direction of the columns from both flange and web were used. The coupon specimens were tested according to the British Standard BS EN 10002-1 (2000) using a 200 kN capacity Instron testing machine. In order to ensure that fracture occurred within the middle portion of the constant gauge length, the test coupons were dimensioned with a more gradual change in cross section from the constant gauge width to the grip. The configuration and dimensions of the coupon specimens are shown in Fig. 3. The specimens failed as expected at the middle part of the specimens by yielding as shown in Fig. 3(d). The measured mechanical material properties and axial tensile stress-strain curve for the tested specimens will be provided in the following section.

4. Experimental Results, Observations and Discussion

4.1 Monotonic test

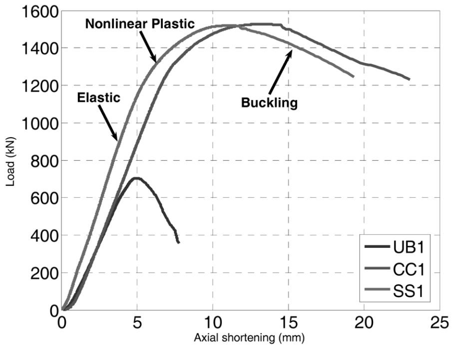
The stress–strain curves were obtained from the coupon tests for the web and the flange of the UB steel columns. The average and measured test results of both flange and web coupon specimens are summarized in Table 2. The yield and ultimate stresses were obtained based on the measured load divided by the initial cross-sectional area of the coupon specimens computed as width multiplied by thickness, and the strain is the change in length divided by the original gauge length of 100 mm shown in Fig. 3(a).

Table 2 Averaged and measured properties of coupon tests

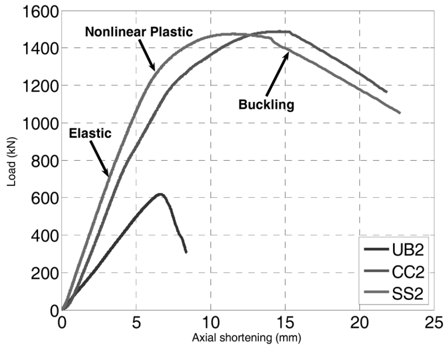
Specimen	E (GPa)	F_y (MPa)	F_u (MPa)	Ultimate Strain (mm/mm)
F1	198	450	595	0.143
F2	201	410	551	0.130
F3	201	400	561	0.127
W1	199	411	590	0.130
W2	198	455	600	0.143
W3	203	411	551	0.150
Average	200	423	575	0.137

4.2 Load-axial shortening relation

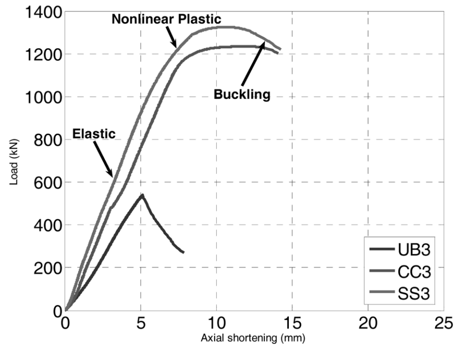
The axial compression tests for the UB and built-up sections were conducted using the UTM machine that has a capacity of 2500 kN and connected to a data acquisition system that records all stages of loading. The load and shortening were recorded using the data acquisition system. The variation of axial load and axial shortening of the columns are shown in Fig. 4. There are several plots in Fig. 4. For illustration purpose, linear elastic, nonlinear plastic, and buckling portions are shown for SS specimens only. It is observed that the axial load-axial shortening curves have three portions – linear elastic,



(a) Column length 1.8 m



(b) Column length 2.0 m



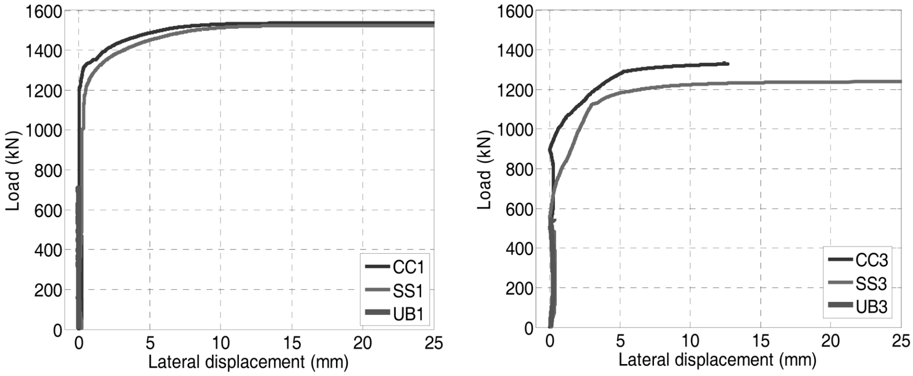
(c) Column length 2.2 m

Fig. 4 Load-axial shortening for UB, CC and SS columns nonlinear plastic and buckling portions. Initially the columns exhibited approximately a linear elastic response in which the slope associated with axial stiffness remained approximately constant. Prior to buckling and failure of the tested specimens, as demonstrated by the lateral deflections, the axial stiffness was reduced significantly until the ultimate load was reached.

The side-to-side columns consistently have higher load capacity than the cruciform columns for the same axial shortening at both the elastic and inelastic portion of the loading. In other words, for the elastic and inelastic portion, at a given load the SS column has smaller axial shortening. However, after reaching the ultimate load capacity such behavior is reversed and the cruciform column showed more axial shortening for a given load compared with the side-to-side columns for the shorter and intermediate columns as shown in Figs. 4(a) and 4(b). This could be attributed to the different initial imperfections, amounts of cutting and welding, and boundary supports. Both cruciform and side-to-side columns achieved similar ultimate axial load for the 1.8 and 2.0 m lengths, however for the 2.2 length columns, the side-to-side column (SS3) performed better than the cruciform column (CC3) all through the three portions (elastic, plastic and buckling) of the loading as shown in Fig. 4(c). The Universal Beam showed similar behavior to that of the side-to-side and cruciform columns, however, with less ductility and very short plastic plateau as compared to the cruciform and side-to-side column. The long Universal Beam (UB3 with length 2.2 m), has no plastic portion, i.e., only elastic and buckling portion which indicates that the buckling is elastic.

4.3 Load-lateral deflection relationship

LVDTs were mounted on each specimen to measure the lateral deflections in two perpendicular directions and the strains on the tension and compression sides of the specimens. Figs. 5(a) and 5(b) show samples of the axial load and the corresponding lateral deflection of the short column (length 1.8 m), and long column (length 2.2 m), respectively. It is observed that the lateral displacement of the columns is very small (close to zero) up to about 80% of the buckling load and the relationship between the axial load and the lateral displacement is linear for this portion. At around 80% of the buckling load the inelastic behavior starts and continues until the column buckles at a lateral deflection of around 10 mm. The UB columns buckle immediately after the elastic part without plastic behavior as was observed in the axial load-axial shortening behavior discussed in the previous section.



(a) Column length 1.8 m (b) Column length 2.2 m

Fig. 5 Load-lateral deformation and load-strain for UB, SS and CC columns

4.4 Comparisons of Experimental and Codes

The steel design codes, namely the Eurocode 3, American Institute of Steel Construction (LRFD) and the British Standard 5950-1 provide formulas for predicting the ultimate axial compressive load capacity of standard and built-up steel columns. According to Eurocode 3, the design buckling resistance of the compression member is given by Eq. (1).

$$N_{b,Rd} = \chi A \frac{f_y}{\gamma_{M1}} \tag{1}$$

where, $\chi = \dfrac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \leq 1$, $\bar{\lambda} = \sqrt{\dfrac{Af_y}{\sigma_{cr}}}$, $\phi = 0.5[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2]$ and $\sigma_{cr} = \dfrac{\pi^2 E}{\lambda^2}$,

$$\lambda = \frac{L_{eff}}{r}$$

According to AISC LRFD, the design compressive strength is given by Eq. (2)

$$\phi P_n = 0.9 F_{cr} A_g \tag{2}$$

where, $F_{cr} = 0.877 F_{cr}$ for $\lambda \geq 4.71\sqrt{E/F_y}$ and $F_{cr} = [0.658^{F_y/F_e}]F_y$ for $\lambda \leq 4.71\sqrt{E/F_y}$ and

$F_e = \pi^2 E/(KL/r_{min})^2$ for UB section column and SS section column

and $F_e = \left[\dfrac{\pi^2 EC_w}{(K_z L)^2} + GJ\right]\dfrac{1}{I_x + I_y}$ for CC section column.

According to BS 5950-1, the design yield strength is given by Eq. (3)

$$P_c = \frac{P_E P_y}{\phi + (\phi - P_E P_y)^{0.5}} \tag{3}$$

where, $\phi = \dfrac{P_y + (\eta + 1)P_E}{2}$, $\eta = 0.001a(\lambda - \lambda_0) \geq 0.0$, $\lambda_0 = 0.2\left(\dfrac{\pi^2 E}{P_y}\right)^{0.5}$, $P_E = \dfrac{\pi^2 E}{\lambda^2}$ and $\lambda = \dfrac{L_E}{r_{min}}$

Table 3 shows a summary of the experimentally measured ultimate axial compressive loads of all columns and the corresponding predicted values. It is clear that almost all codes are conservative in predicting the ultimate compressive load capacity of the columns with a range of up to 22% for the cruciform and side-to-side columns and up to 30% for the universal beam columns. For the UB sections all codes agreed very much in predicting the buckling load of the columns with the different lengths. The least conservative prediction is observed for long cruciform column (CC3) on which the three codes showed the least conservative estimate in the range of -1 to 3% of the experimentally measured values.

Fig. 6 shows the code prediction of the ultimate buckling load of the Universal Beam, cruciform and side-to-side columns as compared with the experimentally measured ultimate loads. It is observed from

Table 3 Summary of the compression capacity of the members (kN)

		Predicted			Ratio		
	Experimental	EC 3	AISC	BS 5950	(Exp / EC3)	(Exp / AISC)	(Exp / BS)
UB1	704	540	551	546	1.30	1.28	1.29
UB2	619	492	500	495	1.26	1.24	1.25
UB3	540	426	436	428	1.27	1.24	1.26
CC1	1526	1247	1285	1287	1.22	1.19	1.19
CC2	1487	1222	1262	1257	1.22	1.18	1.18
CC3	1235	1195	1242	1223	1.03	0.99	1.01
SS1	1503	1257	1338	1298	1.20	1.12	1.16
SS2	1474	1233	1301	1268	1.20	1.13	1.16
SS3	1340	1206	1261	1236	1.11	1.06	1.08

Fig. 6(a) that almost all codes showed similar prediction of the buckling load of the Universal Beams column irrespective of their length (short, intermediate, long) and they are all less than the experimentally measured values. However, it is observed from Figs. 6(b) and 6(c) that the three codes have different predictions of the buckling load with the AISC prediction closer to the experimentally measured buckling load as compared to the Eurocode 3 and BS 5950-1. However, as the column length increases, they tend to converge to the same values of the buckling load and also to the same experimentally measured values as well. This attributed to the fact that the buckling becomes elastic and the elastic

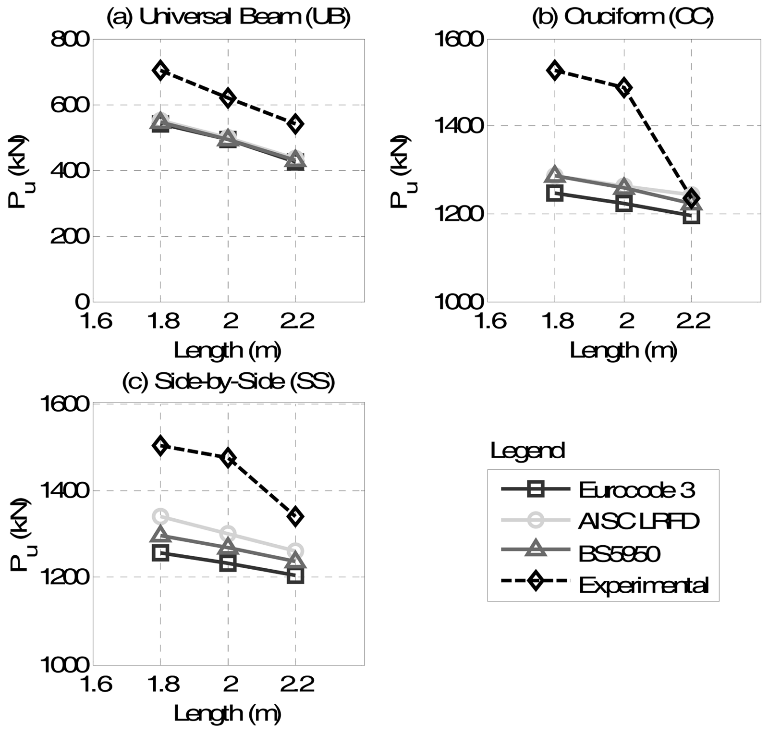


Fig. 6 Variation of buckling load with length for UB, CC and SS columns

buckling formula become applicable. It is clear that the predictions of the buckling load by the three codes are closer to the experimentally measured ones for cruciform and side-by-side columns than that of the Universal Beams columns.

Fig. 7 shows the compressive stress-slenderness ratio for universal beam, cruciform and side-to-side columns for the three codes. It is clear that all codes produce very similar stress-slenderness ratio curves for these columns. It is observed also from Fig. 7(a) that the universal beam columns fall on the elastic region or at the border line between elastic and inelastic, and therefore Euler elastic buckling formula can be applied to predict the buckling load. This has been proved in the load-axial shortening (Fig. 4) and load-lateral deflection (Fig. 5) curves shown before. However, cruciform and side-to-side columns fall in the inelastic buckling region with the third column (SS3 and CC3) closer to the border of elastic-inelastic buckling than the other two columns and therefore, prediction of axial stress is closer to the experimentally measured one as observed in this case. At nearly plastic buckling, i.e., at very small slenderness ratio, AISC prediction of axial load capacity is higher than both Eurocode 3 and BS predictions and therefore closer to the experimentally measured values.

4.5 Accuracy of Codes Prediction & Design Considerations

The accuracy of predicting the buckling load using the three code formulas reveals that the prediction of buckling load of Universal Beam columns is less accurate than the prediction of buckling load of cruciform and side-to-side sections. Prediction of buckling loads for all columns (short, intermediate and long) is within 22% of the experimentally measured values for cruciform and side-by-side columns

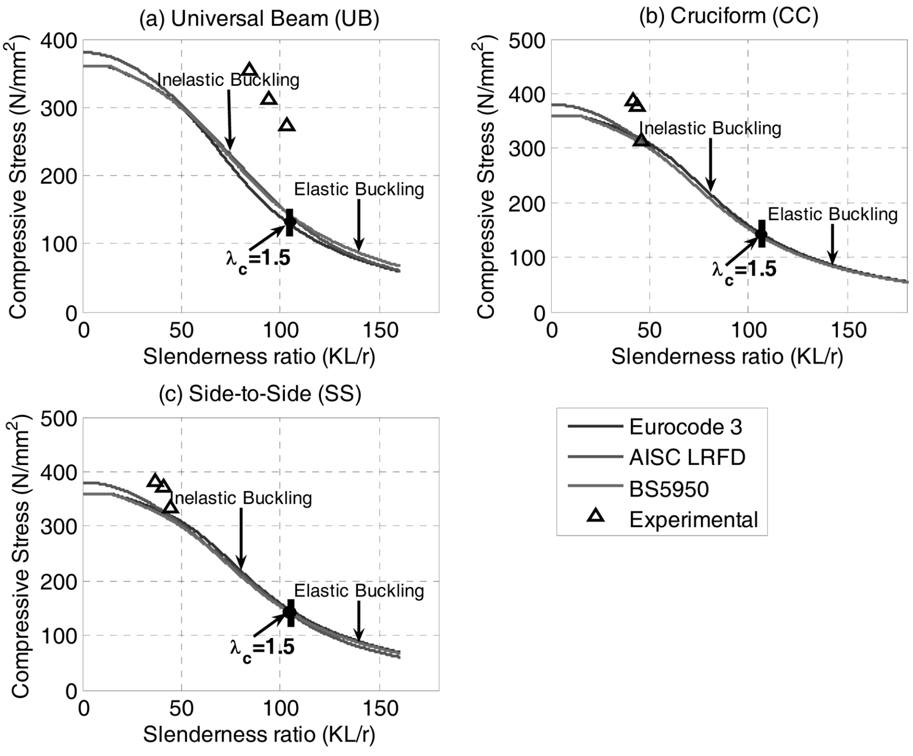


Fig. 7 Compressive stress-slenderness ratio for UB, CC and SS columns

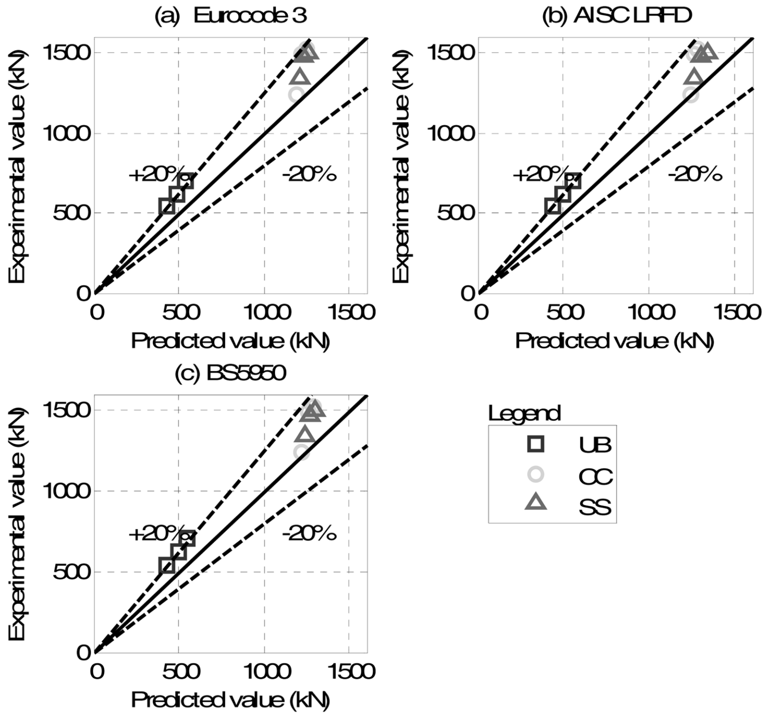


Fig. 8 Accuracy of Eurocode 3, AISC LRFD and BS5950 in Predicting Ultimate Axial Load

for all codes. In addition, as the column length increases, the deviation between the predicted and measured values decreases. This could be related to the assumed boundary conditions of the loaded columns which might not be as accurate for shorter columns. However, it is between 24~34% for Universal Beam columns. Figs. 8(a), 8(b) and 8(c) show the prediction of each code of the ultimate buckling load of the three types of columns UB, CC and SS. It is also clear that the three codes predictions are very close to each other whether they accurately predicts the experimental values or not.

Economy is one of the key issues when selecting steel cross-sections. In order to measure the efficiency and cost of the investigated built-up sections, three Universal Beam sections with comparable load carrying capacity to that of the CC and SS columns were used as a bench mark to compare the saving in steel weight and increase in strength. The prediction is based on the provisions of Eurocode 3 and the results are summarized in Table 4. It should be noted that the cost provided in Table 4 does not include the cost of cutting and welding. It is evident from Table 4 that the saving in weight ranges between 15.63~30.94% while the increase in buckling load capacity ranges approximately between 3.9~9.9%.

5. Summary and conclusion

This paper presented the result of experimental investigation of the buckling capacity of built-up cruciform and side-to-side steel columns fabricated from two Universal Beam sections. The measured ultimate axial load capacities of the tested specimens were compared with the code predicted buckling

Table 4 Summary of ultimate load and saving in weight of steel

Section description	Section	Weight (kg/m)	Compression capacity (kN) EC 3	weight saving %
Controlling	CC 152×89×32	32	1247.14	-
Controlling	SS 152×178×32	32	1256.96	-
Equivalent	UB 254×146×37	37	1370.54	15.63
Equivalent	UB 305×127×42	41.9	1340.94	30.94
Equivalent	UB 406×140×39	39	1307.00	21.88

loads based on Eurocode 3, AISC LRFD, and BS 5959. It can be concluded from this study that:

Code predictions of ultimate axial buckling loads of such columns are conservative compared to the experimentally measured values.

The actual capacity of the tested cruciform columns was higher than the theoretical values with the ratio in the range of 1.01~1.19 compared to BS 5950-1, of 1.03~1.22 compared to EC 3 and of 0.99~1.19 compared to AISC.

The actual capacity of the tested side-to-side columns were higher than the theoretical values with the ratio in the range of 1.08~1.16 compared to BS 5950-1: 2000, of 1.11~1.20 compared to EC 3 and of 1.06~1.13 compared to AISC.

Although the buckling load of cruciform and side-by-side columns is comparable, however, their elastic, plastic and buckling behavior is different.

The Cruciform and side-to-side built-up sections resulted in larger buckling capacity and larger stiffness compared to their counterpart standard UB hot-rolled sections of the same weight. On average the saving in weight is about 20%.

Further experimental and analytical investigation is needed to create a full image about the behavior of the proposed built-up sections in this investigation. Different lengths and sizes of the columns needed to be investigated. In addition, beam-column connections for the proposed column sections should be investigated. The authors are planning to develop finite element (FE) models that predict the axial capacity and capture the behavior of the proposed built-up sections in a future research investigation.

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Notations

The following symbols are used in this paper:

p_y = Design yield strength (kN)
p_E = Euler Strength (kN)
p_c = Design strength (kN)
η = Perry factor for flexural buckling under load
λ = Slenderness Ratio
λ_0 = Limiting Slenderness Ratio
E = Modulus of Elasticity (MPa)
a = Robertson constant, which has the following values (Table 24, of BS5950): a = 2; b = 3.5; c = 5.5 and d = 8.
L_E = Column effective length
$N_{b,Rd}$ = Design buckling resistance of the compression member (kN)
χ = Reduction factor for the relevant buckling mode

α = Imperfection factor which has the following values for the corresponding buckling curve: $a_0 = 0.13$; $a = 0.21$; $b = 0.34$; $c = 0.49$ and $d = 0.76$.
f_y = Design yield strength (MPa)
σ_{cr} = Elastic buckling stress (MPa)
$\bar{\lambda}$ = Non dimensional Slenderness Ratio
γ_{M1} = global partial factor for the buckling resistance
ϕ = AISC LRFD resistance factor
r_x = Radius of gyration on x-axis, in (cm)
r_y = Radius of gyration on y-axis, in (cm)
C_w = Wrapping constant (mm^6)
J = Torsional constant (mm^4)
G = Shear modulus 77200 MPa (11200 ksi)
F_e = Elastic buckling stress according to the Euler equation (MPa)
F_{cr} = Critical buckling stress (MPa)
F_y = Yield stress (MPa)
P_n = Nominal compressive strength (kN)
W = Section flange width (mm)
H = Section depth (mm)
F = Section web thickness (mm)
S = Section flange thickness (mm)
A = Area of cross-section (mm^2)
A_g = Gross area (mm^2)
ϕP_n = Design compressive strength (kN)
P_y = yield compressive strength (kN)
P_e = elastic compressive strength (kN)

CC