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Behaviour of concrete-filled square section stub columns fabricated from mild steel plates and incorporating cold-formed tubes at the vertices

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ABSTRACT
Recent research has investigated the behaviour of fabricated section stub columns made through welding facet plates but also incorporating small diameter circular hollow sections at the vertices. In 2000, Japanese researchers investigated the behaviour of empty fabricated triangular section stub columns made up of normal strength steel facet plates and normal strength steel tubes. The steel used for the facet plates and the tubes had a nominal yield strength 235MPa. Australian researchers in 2004 studied the behaviour of both empty triangular and square fabricated stub columns through the welding of very high strength steel tubes at the vertices to high strength steel plates. The high strength steel facet plates had nominal yield strength of 350MPa and tubes at the vertices with a nominal yield strength of 1350MPa. Further research in Australia extended the previous research on empty fabricated triangular and square sections through the determination of behaviour after concrete filling the fabricated stub columns. This paper investigates the behaviour of concrete-filled fabricated square section stub columns manufactured using mild steel plates, 250MPa yield stress, welded to cold-formed steel tubes, of 350MPa yield stress, at the vertices. Experimental results are analyzed and the strength compared to existing standards.

KEYWORDS: *Columns, Fabricated section columns, Concrete filled steel tubes, Static strength*

1. INTRODUCTION

Previous research has investigated the behaviour of fabricated section stub columns made through welding facet plates but also incorporating small diameter circular hollow sections at the vertices. Aoki and Ji (2000) investigated the behaviour of empty fabricated triangular section stub columns made up of normal strength steel facet plates and normal strength steel tubes. The steel grade of the facet plates and the tubes used in Aoki and Ji (2000)'s investigation had a nominal yield strength 235MPa.

Following Aoki and Ji (2000)'s research, Zhao et al (2004) studied the behaviour of both empty triangular and square fabricated stub columns through the welding of very high strength (VHS) steel tubes at the vertices to high strength steel plates. The research by Zhao et al (2004) used high strength steel facet plates with nominal yield strength of 350MPa and tubes at the vertices with a nominal yield strength o f 1350MPa. The interest in the behaviour of fabricated columns can also be seen through recent research on fabricated long hollow columns made up of both mild steel facet plates and mild steel tubes under axial compression (Javidan et al 2015).

Mashiri et al (2010, 2011) and Alatshan et al (2012) extended previous research on empty fabricated VHS-plate triangular and square sections through the determination of behaviour after concrete filling the fabricated stub columns. This paper investigates the behaviour of concrete-filled fabricated square section stub columns manufactured using mild steel plates and welded to cold-formed steel circular hollow section (CHS) tubes at the vertices. Cold-formed steel tubes are high strength steel tubes that are more commonly available on the Australian steel market compared to VHS steel tubes. The experimental results from this study will be compared to previous research reported by Mashiri et al (2010, 2011) and Alatshan (2012). In this study, the peak loads from the experimental tests are compared to the section capacities determined by different standards in an effort to determine the most appropriate existing standards for estimating sections capacity in fabricated square CHS-plate stub columns.

2. TEST PROGRAMME

2.1. Test Specimens and Materials Properties

Six fabricated square steel tube-to-plate stub columns were manufactured for testing, see Figure 2.1. The fabricated square CHS-plate stub columns are manufactured by welding mild steel facet plates to cold-formed tubes at the vertices as shown in Figure 2.1(a). The resultant stub columns for testing are shown in Figure 2.1(b). The geometric and material properties of the specimens are summarized in Table.2.1. Table 2.1 shows the diameter of tube (d), the thickness of tube (t), the width of plate (B), the thickness of plate (T), and the height of the stub column (H). In addition, the nominal yield stress of steel tubes (f_{ynt}), the nominal yield strength of side plates (f_{ynp}) and the corresponding averages of the measured yield stress of the tube (f_{yt}) and measured yield stress of the plate (f_{yp}) are also listed in Table.2.1. The average measured yield stresses for the plate and the tube were obtained from tensile coupon results.

The tubes used in the fabrication of the stub columns are cold-formed high strength steel with a nominal yield stress (f_{ynt}) of 350MPa and a nominal ultimate strength (f_{unt}) of 430MPa. The facet plates are mild steel of grade 250, and 3 mm in thickness. The end plates are of grade 250, and 10mm in thickness, and the top end plate have a hole of diameter 80mm to enable concrete filling of the fabricated columns. The cold-formed tubes, the end plates and facet plates are welded to form the stub columns using the pulsed metal arc welding process.

Three tensile coupons were used for the facet plates as well as the cold formed tubes to determine the average nominal yield stresses according to AS1391-2007 (SAL 2007). The average nominal yield stress of cold-formed tubes was found to be 410 MPa and the average yield strength of the mild steel side plates was found to be 269 MPa.

Concrete was vertically cast into steel tubular specimens, and was compacted by using a vibrator, see Figure.2.1(c). The concrete was cured inside the steel columns for 28days. The concrete that was used for filling core of the fabricated stub columns, had a specified compressive strength of grade 32MPa. A mean concrete compressive strength of 34MPa was determined from concrete cylinder tests.

(a) (b) (c)

Figure 2.1 Concrete-filled fabricated square section stub columns (a) cross-section and (b) stub-column

Table 2.1 Test Specimens and Material Properties

Specimens	d (mm)	t (mm)	B (mm)	T (mm)	H (mm)	f_{ynt} (MPa)	f_{ynp} (MPa)	f_{yt} (MPa)	f_{yp} (MPa)	Concrete Material Properties
N1NP1(A)	21.3	3.2	90	3	300	350	250	410	269	f_c'=32 MPa f_{cm}=34 MPa
N1NP1(B)	21.3	3.2	90	3	300	350	250	410	269	
N1NP2(A)	21.3	3.2	120	3	400	350	250	410	269	
N1NP2(B)	21.3	3.2	120	3	400	350	250	410	269	
N1NP3(A)	21.3	3.2	150	3	500	350	250	410	269	
N1NP3(B)	21.3	3.2	150	3	500	350	250	410	269	

2.2. Test Set-Up

The specimens were tested 28 days after the casting of concrete. A 3000 kN capacity Instron 8506 testing machine in the Structures Laboratory at the University of Western Sydney was used to test the specimens, see Figure 2.2. As shown in Figure 2.1(b), strain gauges were fixed to the external surface of the steel tubes and facet plates. Linear variable displacement transducers (LVDTs) were also placed symmetrically around the fabricated section in order to measure the deformations of the facet plates and CHS tubes of the specimen. The specimens were loaded using displacement control at a cross head speed of 0.01mm/min.

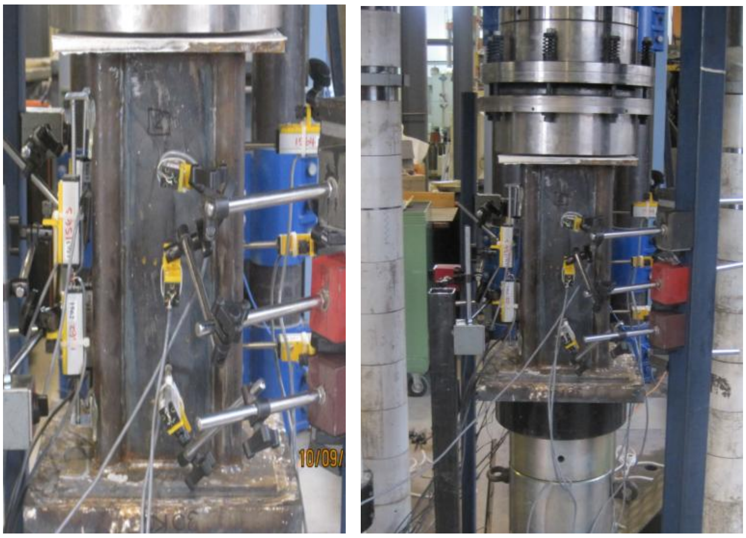


Figure 2.2 Test Set Up for Fabricated Stub Columns

3. TEST RESULTS AND DISCUSSION

3.1. Failure of Specimens

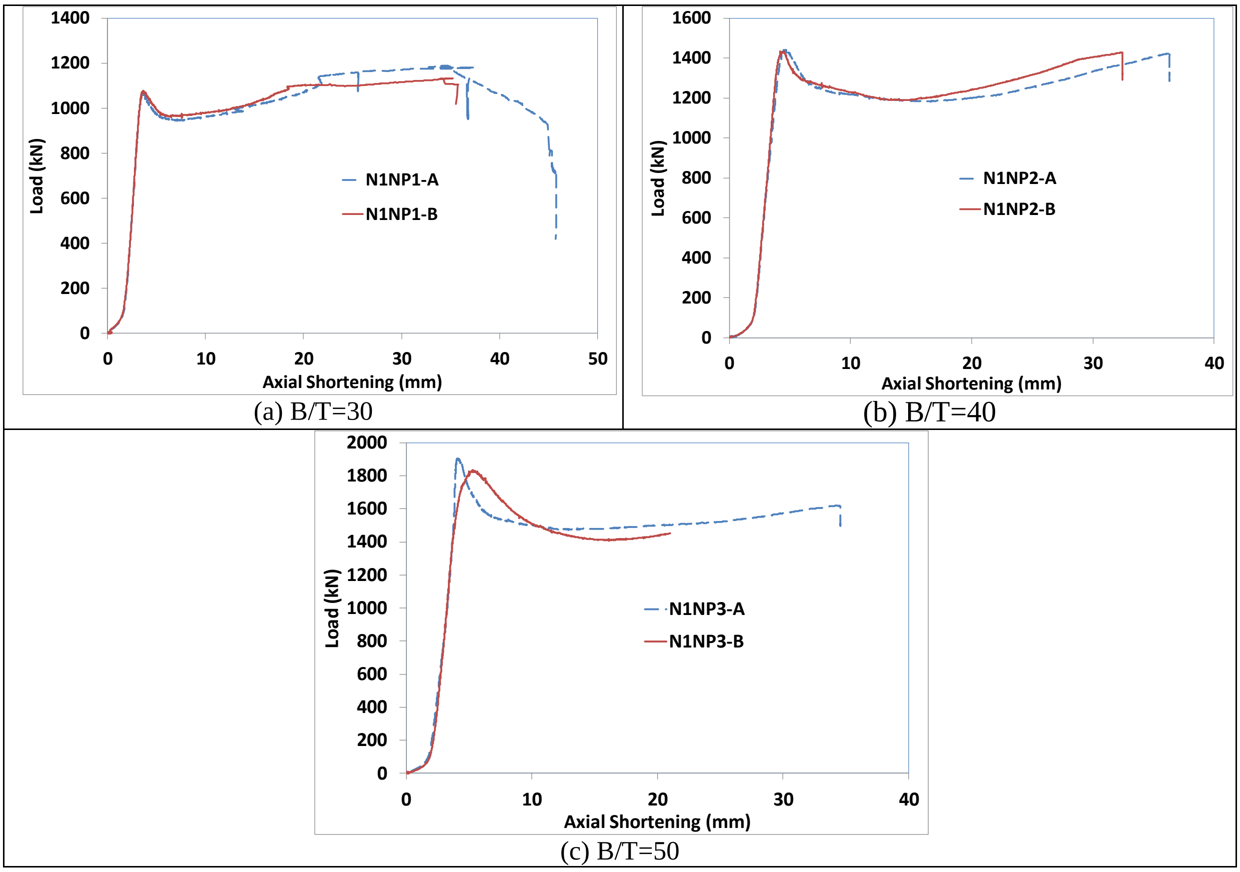
The failure modes of the fabricated square stub columns made up of mild steel plates and cold-formed high strength steel tubes are shown in Figure 3.1(a). The failure was characterized by significant outward buckling and deformation showing relative ductility of the specimens compared to those reported by Mashiri et al (2011, 2014) which were made from very high strength steel tubes and mild strength steel facet plates. As shown in Figure 3.1(a), following significant outward buckling and deformation, fracture of the weld area adjoining the facet plates and the cold-formed tubes was also observed as shown in Figure 3.1(b).

(a) (b)

Figure 3.1 Failure of fabricated square hollow section stub columns

3.2. Load deformation curves

The load-deformation curves for fabricated CHS-plate square stub columns are characterized by strain hardening after the peak load following first yield as shown in Figure 3.2. Figure 3.2 also shows that the load-deformation curves exhibit ductility, with the specimens able to carry load without a significant drop as deformation occurs past the peak load. Ductility seems to be more pronounced for the specimens with a smaller facet plate *B/t* ratio compared to those with comparatively larger values of *B/t*. This load-deformation behavior shows increased ductility to that of fabricated VHS-plate square stub columns reported by Mashiri at al. (2011, 2014) where the very high strength steel tubes were used at the vertices. The specimens fabricated using VHS tubes did not exhibit strain hardening behavior.



(a) B/T=30

(b) B/T=40

(c) B/T=50

(a) (b)

Figure 3.2 Load-axial shortening curves for fabricated square hollow section stub columns

3.3. Comparison of Test Capacity and Design Standards

The peak load for the stub column tests (N_{ue}) were obtained from the load-axial shortening curves shown in Figure 3.2.

The peak load is compared to section capacities predicted using different standards. The standards used in the comparison are from Australia, AS5100.6 (SAL 2004), USA, ACI 318 (2002), Europe, EC4 (1994) and China, GJB 4142 (2000). The capacities for the fabricated section stub columns are estimated using the standards formulae for rectangular composite columns which are summarized in the following sections.

The section capacity of a concentrically loaded rectangular composite compression member according to AS5100.6 (SAL 2004) is given by:

$$N_{uo} = A_s f_y + A_c f'_c \quad (3.1).$$

where, A_s is the cross-sectional area of the structural steel section, f_y is the nominal yield strength of the structural steel, A_c is the area of concrete in the cross-section and f_c' is the characteristic compressive strength of the concrete at 28 days. The use of Equation 3.1 in AS5100.6 is limited to steels with a maximum yield stress of 350MPa. The plate element slenderness (λ_e) should be less than yield slenderness limit (λ_{ey}) which is equal to 45 for hot-rolled plates supported on both edges.

According to the American Concrete Institute, ACI 318 (2005), the section capacity of a concentrically loaded rectangular composite compression member is given by:

$$N_{uo} = A_s f_y + 0.85 A_c f'_c \quad (3.2).$$

where, A_s is the cross-sectional area of steel tube, f_y is the yield stress of steel, A_c is the cross-sectional area of concrete core and f_c' is the cylinder strength of concrete. The use of Equation 3.2 in AS5100.6 is limited to *B/t* ratios such that, $B/t \leq \sqrt{3E_s/f_y} = 50.2$ for f_y=250.

The section capacity of a concentrically loaded rectangular composite compression member according to Eurocode 4, EC4 (2004) is given by:

$$N_{uo} = A_s f_y + 0.85 A_c f'_c \quad (3.3).$$

where, A_s is the cross-sectional area of the structural steel section, f_y is the design value of the yield strength of structural steel, A_c is the cross-sectional area of concrete and f_c' is the design value of the cylinder compressive strength of concrete. In using Equation 3.3 for concrete filled sections the coefficient 0.85 may be replaced by 1.0, as was adopted in this investigation. The use of Equation 3.3 in EC4 (2004) is also limited to *B/t* ratios such that, $B/t \leq 52\sqrt{235/f_y} = 50.4$ for f_y=250.

The Chinese standard, GJB 4142 (2000) for composite structures uses the following formula to estimate section capacity.

$$N_{uo} = f_{scy} A_{sc} \quad (3.4)$$

where, $f_{scy} = (1.212 + B\xi + C\xi^2) f_{ck}$;

$B = 0.1381 \frac{f_{sy}}{215} + 0.7646$;

$C = -0.0727 \frac{f_{ck}}{15} + 0.0216$;

$\xi = \frac{A_s f_y}{A_c f_{ck}}$ =Confinement Factor

A_{sc} = Total cross section area of steel and concrete = A_s+ A_c; f_{ck} = 0.67 f_{cu} and

f_{cu} = characteristic compressive strength of the concrete at 28 days.

The peak loads (N_{ue}) from the load-axial deformation curves shown in Figure 3.2 are shown in Table 3.1 for the fabricated CHS-plate square stub columns. The section capacities (N_{uo}) estimated from the various standards using nominal material properties are shown in Table 3.1. Table 3.1 also shows the strength indices based on the different standards. The Strength Index (*SI*) is defined as the ratio of the peak experimental load (N_{ue}) to the section capacity (N_{uo}):

$$SI = \frac{N_{ue}}{N_{uo}} \quad (3.5).$$

Table 3.1 Peak experimental load (N_{ue}) and estimated section capacity (N_{uo}) based on different standards and relevant Strength Indices (*SI*) using nominal material properties

Specimens	*B* (mm)	*T* (mm)	*B/T*	N_{ue} (kN)	$N_{uo\text{-}SAL}$ (kN)	SI_{SAL}	$N_{uo\text{-}ACI}$ (kN)	SI_{ACI}	$N_{uo\text{-}EC4}$ (kN)	SI_{EC4}	$N_{uo\text{-}GJB}$ (kN)	SI_{GJB}
N1NP1(A)	90	3	30	1067	889	1.201	834	1.279	889	1.201	684	1.559
N1NP1(B)	90	3	30	1079	889	1.214	834	1.294	889	1.214	684	1.577
N1NP2(A)	120	3	40	1440	1215	1.185	1125	1.280	1215	1.185	945	1.523
N1NP2(B)	120	3	40	1432	1215	1.178	1125	1.272	1215	1.178	945	1.515
N1NP3(A)	150	3	50	1895	1600	1.185	1466	1.293	1600	1.185	1252	1.514
N1NP3(B)	150	3	50	1836	1600	1.148	1466	1.253	1600	1.148	1252	1.466
					Mean	1.185		1.278		1.185		1.526
					SD	0.021		0.014		0.021		0.035
					COV	0.017		0.011		0.017		0.023

Table 3.1 shows that the values of the strength index (*SI*) are greater than 1.0 for the Australian Standard

AS5100.6 (SI_{SAL}), the American Concrete Institute (SI_{ACI}), Eurocode 4 (SI_{EC4}) and the Chinese Standard (SI_{GJB}). This shows that the existing standards provide a conservative estimate of the section capacity. The mean, standards deviation and coefficient of variation of the strength index based on various standards are also given in Table 3.1.

The mean strength indices of the Australian Standard AS5100.6 (SAL 2004) and Eurocode 4 (EC4 2004) are closest to1.0, when nominal material properties are used as shown in Table 3.1. The American Concrete Institute standard (ACI 2005) as well as the Chinese standard show even greater conservatism in the estimation of section capacity with strength indices greater than 1.2 and 1.5 respectively. Based on these observations the AS5100.6 and EC4 are recommended for determining section capacity of fabricated CHS-plate square stub columns.

Table 3.2 is similar to Table 3.1 but uses measured material properties in the estimation of section capacity for the fabricated CHS-plate square stub columns. Table 3.2 shows that there is a closer agreement between the estimated section capacity values and peak experimental loads when measured materials are adopted for the estimation of section capacity compared to nominal material properties. Table 3.2 equally shows that the Australian Standard AS5100.6 and EC4 both give the best estimate for section capacity of fabricated CHS-plate square stub columns.

Table 3.2 Peak experimental load (N_{ue}) and estimated section capacity (N_{uo}) based on different standards and relevant Strength Indices (*SI*) using measured material properties

Specimens	*B* (mm)	*T* (mm)	*B/T*	N_{ue} (kN)	$N_{ue\text{-}SAL}$ (kN)	SI_{SAL}	$N_{ue\text{-}ACI}$ (kN)	SI_{ACI}	$N_{ue\text{-}EC4}$ (kN)	SI_{EC4}	$N_{ue\text{-}GJB}$ (kN)	SI_{GJB}
N1NP1(A)	90	3	30	1067	976	1.094	918	1.163	976	1.094	707	1.509
N1NP1(B)	90	3	30	1079	976	1.106	918	1.176	976	1.106	707	1.526
N1NP2(A)	120	3	40	1440	1324	1.088	1228	1.172	1324	1.088	972	1.481
N1NP2(B)	120	3	40	1432	1324	1.082	1228	1.166	1324	1.082	972	1.473
N1NP3(A)	150	3	50	1895	1734	1.093	1591	1.191	1734	1.093	1282	1.478
N1NP3(B)	150	3	50	1836	1734	1.059	1591	1.154	1734	1.059	1282	1.432
					Mean	1.087		1.170		1.087		1.483
					SD	0.014		0.012		0.014		0.029
					COV	0.013		0.010		0.013		0.020

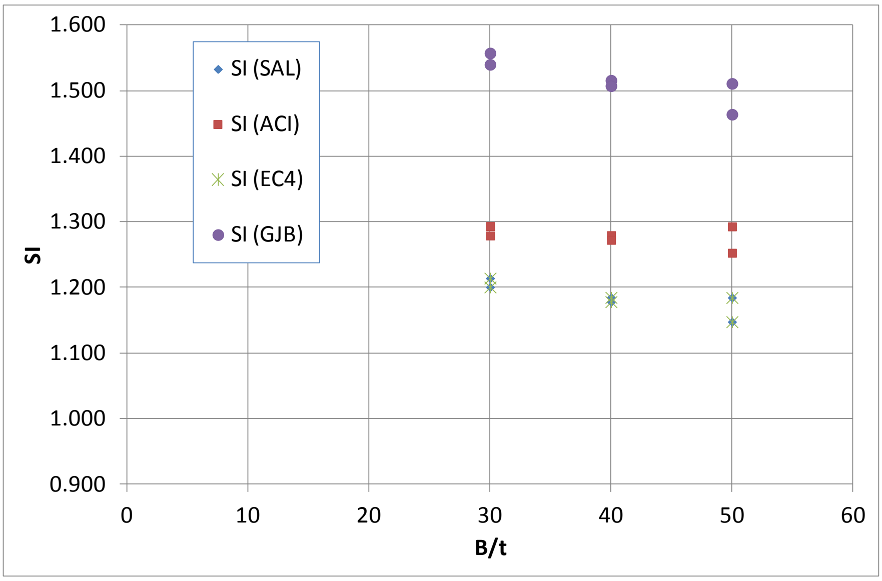


Figure 3.3 Strength Index (SI) versus B/t ratio

Figure 3.3 shows the relationship between strength index (*SI*) and the ratio, *B/t*. In Figure 3.3, as the value of *B/t* increases, the value of *SI* decreases. This trend shows that as *B/t* increases, there is less confinement of the concrete core. This is because as the *B/t* ratio becomes large, the facet plates in the fabricated columns become more susceptible to local buckling. As *B/t* increases, the constraining factor as defined by the Chinese Standard also decreases (Han et al 2005).

The observation in Figure 3.3 is different to that observed in similar fabricated VHS-plate square stub columns where there was no noticeable decrease in *SI* as *B/t* increased as reported by Mashiri et al. (2011). This was attributed to a greater reinforcing factor at the vertices when very high strength steel tubes are used in the fabrication.

4. CONCLUSIONS

Six (6) fabricated CHS-plate square stub columns made up of cold-formed circular hollow section (CHS) tubes and mild steel plates were tested under axial compression. The following observations were made:

(a) The load-deformation curves for fabricated CHS-plate square stub columns show strain hardening effects with the load increasing again post peak and remaining stable over a significant amount of deformation. This behavior points to better ductility of CHS-plate square stub columns compared to previous tests of fabricated VHS-plate square stub columns.
(b) The CHS-plate square stub columns failed through outward buckling of the facet plates and local buckling of the CHS tubes in the vicinity of the facet plate buckle. Fracture of the weld area at the interface of the facet plates and the CHS tubes is also evidenced at large deformations of the stub columns.
(c) Based on the standards that were studied, the Australian Standard, AS5100.6 and Eurocode 4 were found to be the most suitable predictors of peak experimental load for the fabricated CHS-plate square stub columns under axial compression.
(d) A plot of strength index (*SI*) versus *B/t* shows that the strength index decreases as *B/t* increases. This shows that at larger values of *B/t*, there is a reduction in confinement of the concrete core and hence a reduction in composite action between the steel facet plates and the confined concrete.

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