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Experimental behaviour and strength of concrete-encased composite beam–columns with T-shaped steel section under cyclic loading

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Abstract

Experimental research was conducted to investigate the structural behaviour of concrete-encased composite beam–columns with T-shaped steel section. Specimens were tested under lateral cyclic loading and axial compression. The test parameters included the distribution of longitudinal reinforcement, the spacing of transverse reinforcement, the presence of cross ties, and the axial compressive load level. The test results indicate that the cyclic behaviour and failure modes of the beam–columns are greatly affected by the direction of the bending moment owing to the unsymmetrical cross section. The concrete-encased composite beam–columns can develop stable hysteretic response and large energy absorption capacity by providing cross ties and decreased spacing of transverse ties. The current ACI and AISC-LRFD design provisions were also evaluated by comparing predicted strengths with the test results.

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1. Introduction

Combining structural steel with reinforced concrete, the composite structural systems are often used for medium-rise to high-rise buildings. Two types of composite columns

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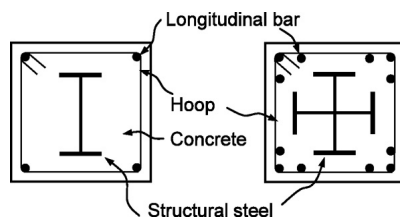


Fig. 1. Types of concrete-encased steel composite column.

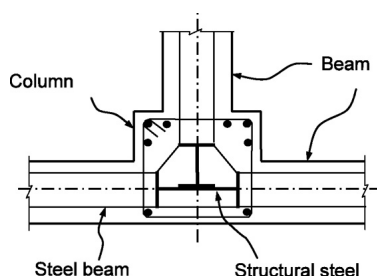


Fig. 2. An exterior composite column.

frequently designed in the buildings are concrete-encased steel columns and concrete-filled steel tube columns. A concrete-encased steel composite column is in the form of a structural steel column encased with structural concrete, and usually further reinforced with longitudinal bars and lateral ties. The structural steel column is fabricated from rolled or built-up shapes. As compared with the use of either a steel column or a reinforced concrete column, the concrete-encased steel composite column has the advantage of increasing axial load-carrying capacity, structural stiffness, and fire resistance.

Fig. 1 shows two types of concrete-encased steel composite column frequently used in composite buildings. A structural steel with H- or cross-shaped steel section is generally used in an inner column of the composite building. For architectural reasons, it is common to design a flush face of the beams and the columns in an exterior frame. Fig. 2 shows an exterior beam-to-column connection. A T-shaped structural steel is usually appropriate to connect the steel beams inside the composite beams, and to achieve a flush face of composite beams and columns.

Many studies have been conducted for concrete-encased H-shaped steel composite columns to investigate the axial load-carrying capacity [1,2], behaviour under uniaxial bending and axial compressive load [3–6], and behaviour under biaxial bending and axial compressive load [7,8]. However, very few experimental data are available for the case of concrete-encased T-shaped steel composite columns. Roik and Bergmann [9] proposed a design method for composite columns with unsymmetrical cross section. The method is based on the simplified approach for composite columns given in the 1984 version of Eurocode 4 [10] that is applicable only for the doubly symmetrical cross section. Results calculated by Roik and Bergmann showed good agreement with their test results which were undertaken to verify the design method.

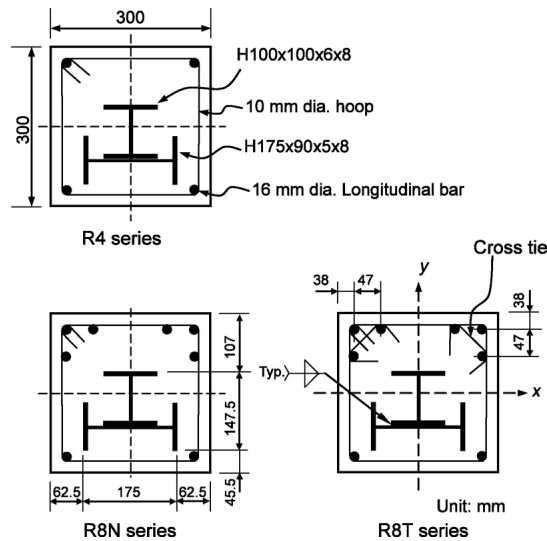


Fig. 3. Cross sections of the specimens.

This study is to elucidate the cyclic behaviour and flexural strength of concrete-encased composite beam–columns with T-shaped steel section and different reinforcement. Test specimens were tested under uniaxial bending moment and constant axial compressive load. Due to the unsymmetrical structural steel section, the cross section of the specimens has only one axis of symmetry. The lateral load to create the bending moment was applied in the plane of the symmetrical axis. The primary objective of this study is to examine the effects of the bending moment and axial compressive load on the cyclic behaviour and ultimate strength.

2. Experimental program

2.1. Test specimens

A total of ten specimens were designed to represent approximately a half-scale model of a prototype column used in medium-rise buildings. The test specimens had a square cross section of 300×300 mm. Fig. 3 shows the three configurations of the cross sections. The test specimens consisted of the structural steel shape, longitudinal reinforcement, transverse reinforcement, and concrete. The variables studied included number of longitudinal bars, spacing of the transverse ties, axial load level, and cross ties. Table 1 summarizes the test specimens. Each specimen was numbered to reflect its variables. The first two characters, R4 or R8, represent the number of the longitudinal bars. The subsequent character, T or N, indicates the presence or absence of the cross tie, respectively. The second group of characters represents the spacing of the transverse hoops and cross ties, where S7 and S12 indicate a 75 mm and a 125 mm spacing, respectively. The last group of characters stands for the applied axial load level, where P2 and P4 indicate

Table 1
Summary of test specimens

Specimen designation	Number of rebar	Cross tie	Spacing of hoop (mm)	Applied axial load (% of P_{squash})
R4-S7-P4	4	–	75	40
R4-S12-P4	4	–	125	40
R8N-S7-P2	8	–	75	20
R8N-S7-P4	8	–	75	40
R8N-S12-P2	8	–	125	20
R8N-S12-P4	8	–	125	40
R8T-S7-P2	8	yes	75	20
R8T-S7-P4	8	yes	75	40
R8T-S12-P2	8	yes	125	20
R8T-S12-P4	8	yes	125	40

that 20% and 40% of the squash load of the composite column was axially applied to the specimens, respectively.

The T-shaped structural steel used in the specimens consisted of two hot-rolled sections, an $H175 \times 90 \times 5 \times 8$ section and an $H100 \times 100 \times 6 \times 8$ section. The flange of the latter section was welded to the web of the former section to create a T-shaped steel section. The ratio of the structural steel area to the gross area was 5%. There was 34 mm eccentricity from the centroid of the structural steel shapes to the geometric center of the column cross section.

As shown in Fig. 3, a longitudinal bar was placed at each corner of the column for the R4 series of specimens. Four additional longitudinal bars, two at each upper corner, were added in the R8 series of specimens. These additional bars were intended not only to increase the axial strength of the column but also to provide a better confinement for the concrete core. All the longitudinal bars were Grade 40 with minimum yield strength of 276 MPa (40 ksi) [11], 16 mm in diameter and deformed. In addition, cross ties of 10 mm in diameter were used at upper corners of the R8T series of specimens to engage the additional longitudinal bars and to enhance the deformation ductility of the column. Following the requirement of the ACI building code for seismic design [12], the cross ties engaged the longitudinal bars through a seismic hook of 135° at one end and a 90° hook at the other end.

Deformed bars of 10 mm in diameter were used as hoop reinforcement. The 75 mm hoop spacing was determined by one-quarter of the least side dimension of the cross section recommended for reinforced concrete compression member by the ACI [12]. The 125 mm hoop spacing was calculated according to latest AISC seismic provisions [13] that require the hoop reinforcement to have a minimum area A_{sh} of:

$$A_{sh} = 0.09h_{cc}s \left(1 - \frac{F_y A_s}{P_n} \right) \frac{f'_c}{F_{yh}} \quad (1)$$

where h_{cc} is the cross-sectional dimension of the confined core measured center-to-center of the hoop reinforcement; s is the spacing of the hoop reinforcement; F_y is the specified yield strength of the structural steel; A_s is the cross-sectional area of the structural steel; f'_c

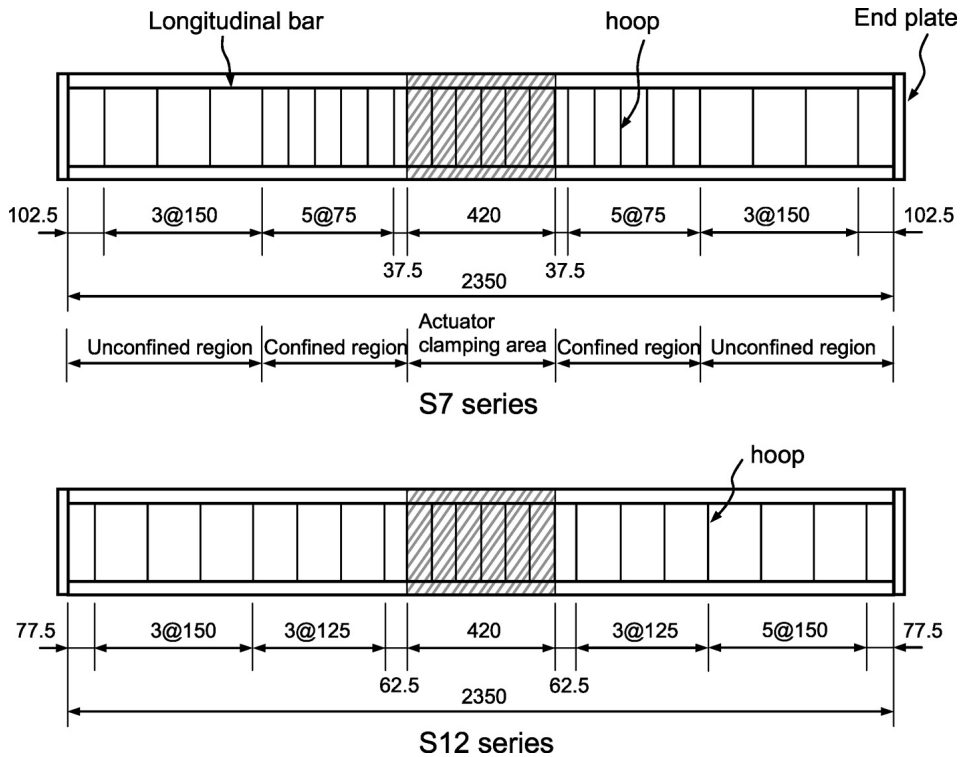


Fig. 4. Spacing of hoop reinforcement.

is the specified compressive strength of the concrete; P_n is the nominal axial compressive strength of the composite column; and F_{yh} is the specified yield strength of the hoops. Fig. 4 illustrates the hoop arrangement for S7 and S12 series of specimens. The hoop spacing was provided at a spacing of 75 mm and 125 mm for S7 and S12 series of specimens, respectively, over confined regions of the column where flexural yielding is likely to occur. A spacing of 150 mm was used for the remainder of the column lengths, which are unconfined regions near both ends of the column.

All the specimens were loaded under a constant axial load then subjected to a cyclic lateral force. The magnitude of the applied axial load for P2 and P4 series of specimens was 20% and 40%, respectively, of the squash load of the column, P_{squash} , which was defined as follows:

$$P_{\text{squash}} = 0.85 f'_c A_c + F_y A_s + F_{yr} A_r \quad (2)$$

where A_c is the concrete area; F_{yr} is the specified yield strength of the longitudinal bar; and A_r is the cross-sectional area of the longitudinal bar. Moreover, P_{squash} was calculated using the measured material strengths, which are given in Table 2. The concrete cylinder strength was 30.4 MPa at 28 days and was 34.5 MPa measured at the time of testing.

Table 2
Measured mechanical properties of structural steel and reinforcement

Material	Yield strength (MPa)	Ultimate strength (MPa)
Steel	358	455
Rebar	406	594
Tie	430	615

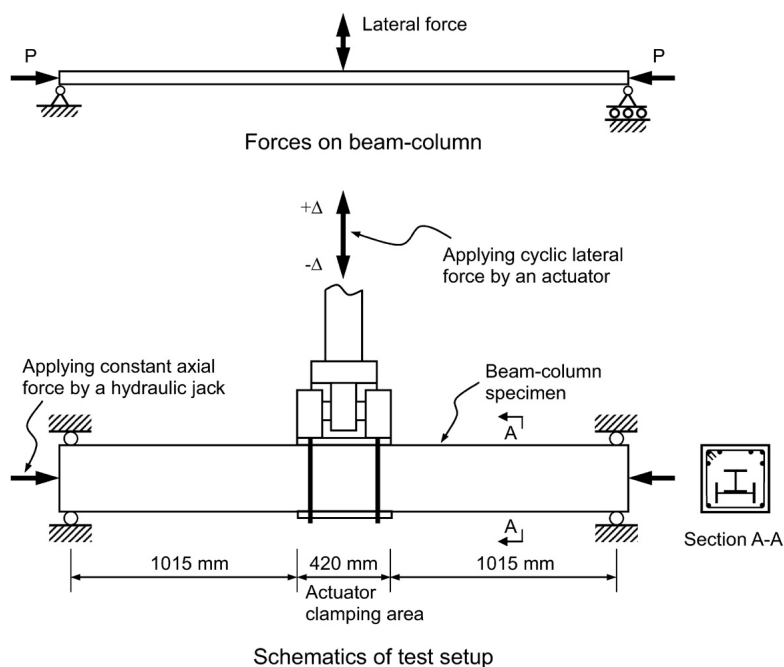


Fig. 5. Test setup of the beam–column.

2.2. Test setup and test procedure

Fig. 5 illustrates the test setup for simulating the loading state of a beam–column. Roller supports were used at both ends of the specimen. With this test setup, the distribution of the bending moment in each half of the specimen is similar to that in a prototype column between the point with maximum moment and inflection point when the column is subjected to a lateral load. The lateral load was applied by an MTS servo-controlled hydraulic actuator to the midpoint of the column, using a displacement-controlled testing at a speed of 10 mm/min. The axial load was exerted through a hydraulic jack installed at the end of the column. A test protocol was used to investigate the cyclic behaviour of the specimens and to establish the ductility capacity. The column was laterally subjected to a predetermined cyclic displacement history as indicated in Fig. 6, which shows that the peak displacements were increased in multiples of the yielding displacement. The

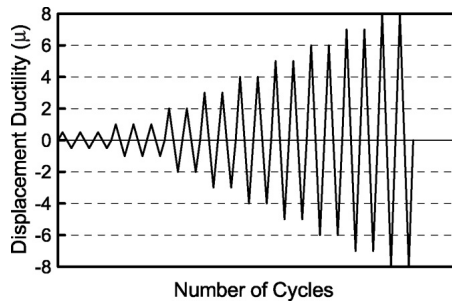


Fig. 6. Lateral displacement history.

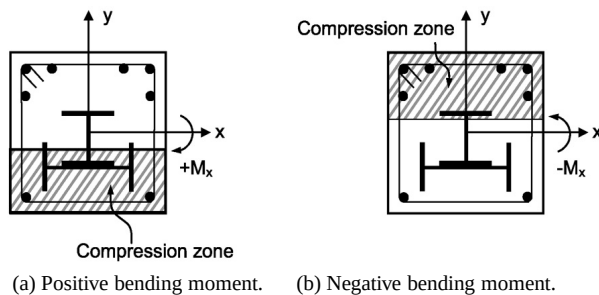


Fig. 7. Direction of the bending moment.

yielding displacement, Δ_y , is the calculated lateral displacement of the column when the extreme longitudinal bar reaches yield. A displacement ductility, μ , is defined as the ratio of the applied lateral displacement, Δ , to the yielding displacement, Δ_y , of the column.

$$\mu = \frac{\Delta}{\Delta_y}. \quad (3)$$

The specimens were subjected to three successive cycles at the displacement ductility of $\mu = \pm 0.5$ and ± 1.0 , and two cycles at the displacement ductility over 1.0.

3. Experimental results and discussion

3.1. Behaviour and failure mode

As indicated in Fig. 7, the direction of the bending moment was defined to describe the cyclic behaviour of the test specimens. Positive bending moment corresponds to positive displacement ductility.

3.1.1. R4 series of specimens

The cyclic behaviour of R4-S7-P4 was similar to that of R4-S12-P4. Flexural cracks initially occurred at the displacement ductility of $\mu = \pm 0.5$; afterwards, the cracks

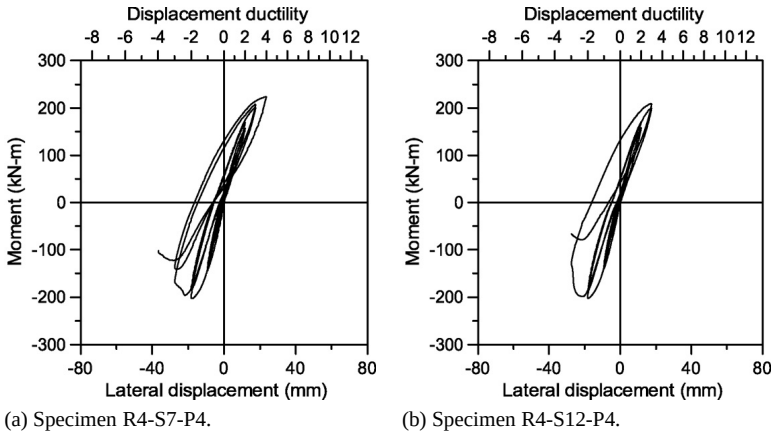


Fig. 8. Moment–lateral displacement relations for R4 series of specimens.

progressively grew. Concrete crushed at cycles of $\mu = -2$ and $+4$, and longitudinal bars buckled at cycles of $\mu = -3$. The lateral load dropped rapidly after the longitudinal bars buckled.

The response of the specimens is presented in the moment–lateral displacement hysteresis curves. In addition to the moment caused by the applied lateral force, the bending moment was calculated also considering the $P-\Delta$ effect caused by the applied axial compressive load acting on the lateral displacement of the column. Fig. 8 shows the moment–displacement relations for specimens R4-S7-P4 and R4-S12-P4. In Fig. 8, the horizontal axis represents the lateral displacement of which 20 mm corresponds to a drift angle of 2.0% rad. The displacement ductility is also given in the horizontal axis at the top of the figure. Due to the unsymmetrical cross section of the column, different yielding displacement, Δ_y , was specified for positive and negative bending moments. Therefore, different scales of the displacement ductility in the positive and negative directions are shown in the same figure for the same amount of the lateral displacement. As indicated in all the figures, both specimens demonstrated poor energy absorption capacity under cyclic loading.

3.1.2. R8N series of specimens

The moment–displacement hysteretic responses for R8N series of specimens, R8N-S7-P2, R8N-S7-P4, R8N-S12-P2, and R8N-S12-P4, are shown in Fig. 9. For the specimens with an axial compressive load of 20% of the squash load, R8N-S7-P2 and R8N-S12-P2, minor concrete crush occurred at cycles of $\mu = -3$. The crushing of the concrete resulted in the pinching in the moment–displacement curves when the specimens were subjected to the negative bending moment. In specimen R8N-S7-P2, the longitudinal bars buckled at cycles of $\mu = -6$ and the hook of the lateral ties failed at cycles of $\mu = -7$, which caused the moment resistance to decrease. Fig. 10(a) shows the appearance of this specimen after failure. However, the moment dropped significantly in specimen R8N-S12-P2 at cycles of $\mu = -5$ due to local buckling of the longitudinal bars and serious spalling of the concrete as shown in Fig. 10(b).

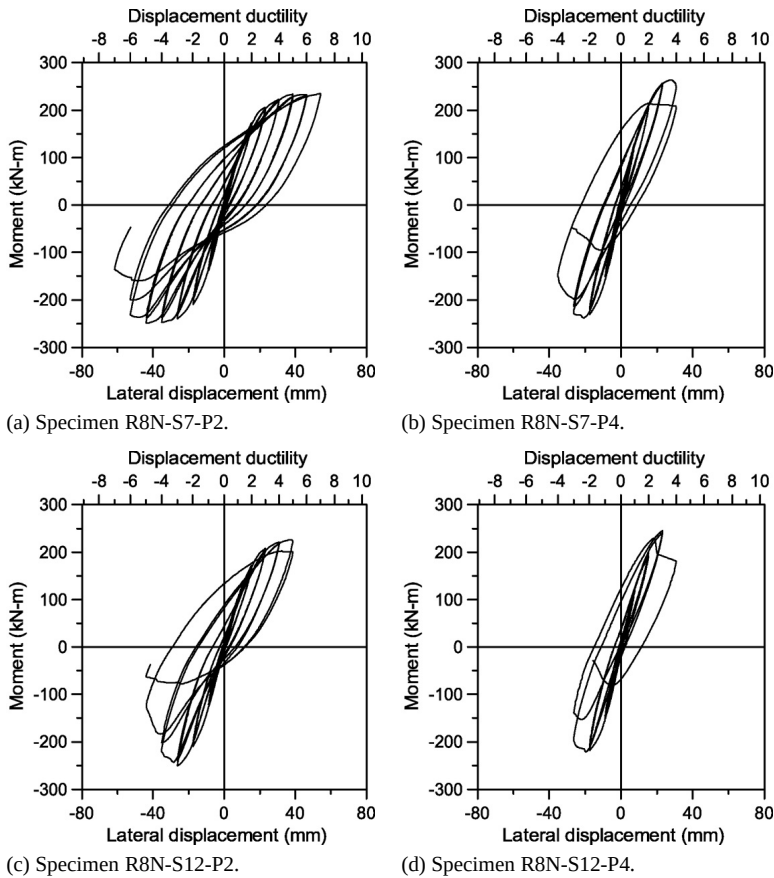
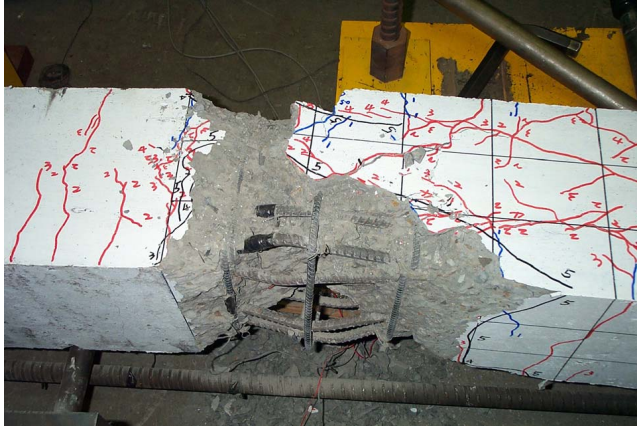


Fig. 9. Moment–lateral displacement relations for R8N series of specimens.

For specimens with an axial load of 40% of the squash load, R8N-S7-P4 and R8N-S12-P4, the moment began to decrease at cycles of $\mu = -2$ to -3 due to cracking and crushing of the concrete. Afterwards, as a result of the buckled longitudinal bars and failure of the hook of the lateral ties, the moment decreased rapidly at cycles of $\mu = -4$ and -3 for R8N-S7-P4 and R8N-S12-P4, respectively. The failure phenomena were similar for both specimens. Crushing of the concrete followed by local buckling of the longitudinal bars and fracture of the hoops caused failure of both specimens. These two specimens showed worse cyclic performance than specimens with low axial load, R8N-S7-P2 and R8N-S12-P2, as shown in Fig. 9.

3.1.3. R8T series of specimens

The moment–displacement hysteresis curves for R8T series of specimens, R8T-S7-P2, R8T-S7-P4, R8T-S12-P2, and R8T-S12-P4, are shown in Fig. 11. Specimens R8T-S7-P2 and R8T-S12-P2 behaved similar to specimens R8N-S7-P2 and R8N-S12-P2, but reached higher displacement ductility owing to the additional cross ties. The cross ties enhanced



(a) Specimen R8N-S12-P2.



(b) Specimen R8N-S12-P4.

Fig. 10. Specimens R8N-S12 at failure.

cyclic behaviour by confining the concrete and preventing buckling of the longitudinal bars as demonstrated in Fig. 12, which presents the appearance of specimen R8T-S7-P2 at failure. However, it is observed that the cross ties had little effect on the cyclic behaviour for specimens with high axial load. With 40% of the squash load as the axial compressive force, specimens R8T-S7-P4 and R8T-S12-P4 demonstrated almost identical behaviour and failure modes as observed in specimens R8N-S7-P4 and R8N-S12-P4, without cross ties.

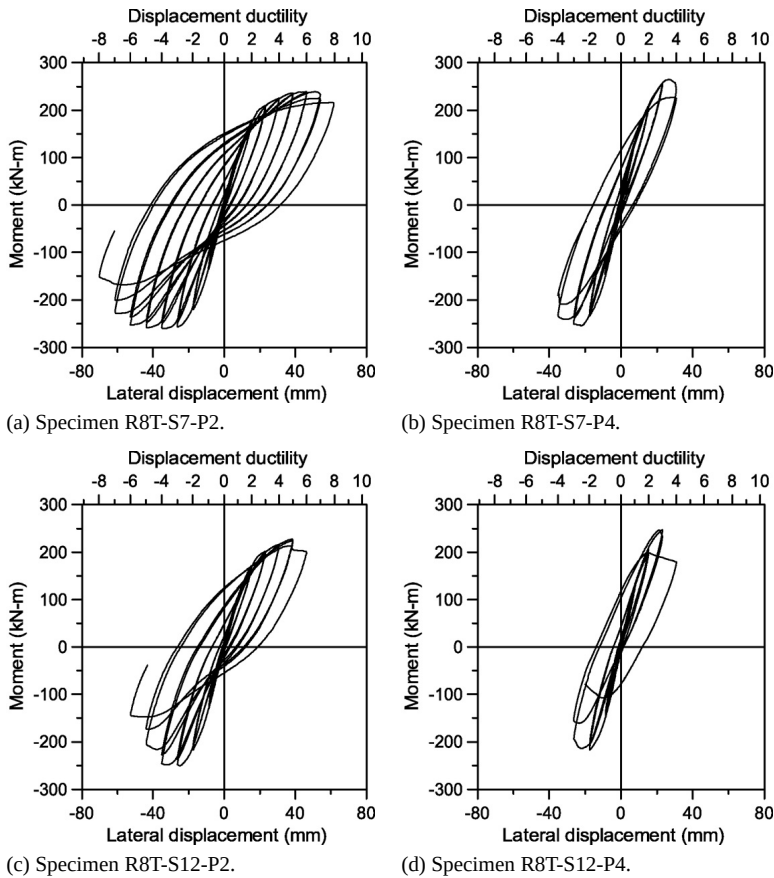


Fig. 11. Moment–lateral displacement relations for R8T series of specimens.

3.2. Characteristics of moment–displacement curves

Because of the unsymmetrical structural steel section and distribution of the longitudinal bars in the cross section of the column, the moment–displacement curves show different hysteresis curves in the positive and negative bending moments. When the cross section of the columns is subjected to a positive bending moment as indicated in Fig. 7(a), the internal compressive force can be resisted by the structural steel located in the compression zone, in addition to the concrete and the longitudinal bars. Moreover, the structural steel section can substantially confine the concrete core in the compression zone. Meanwhile, the longitudinal bars and the structural steel section in the tension zone can provide the corresponding tensile force. Therefore, the hysteresis curves of the columns under a positive bending moment are very stable, especially demonstrated in the specimens with low axial load. Slight strength degradation was observed due to minor crushing of the concrete.



Fig. 12. Specimen R8T-S7-P2 at failure.

When the cross section is subjected to a negative bending moment as shown in Fig. 7(b), the internal compressive force is resisted mainly by the reinforced concrete, with little contribution from the much less structural steel. Moreover, the portion of the structural steel can not effectively confine the concrete core. As a result, the concrete in the compression zone is more prone to crush. The strength and stiffness of the column are obviously deteriorated due to crushing and spalling of the concrete and subsequent buckling of the longitudinal bars. This characteristic leads to increased pinching of the hysteresis curves in the negative bending moment. The behaviour of the specimens under negative bending moment is somewhat similar to that of a reinforced concrete beam–column.

3.3. Ductility capacity

Ductility is required to ensure ductile response of the columns whereby the strength of the column shall not significantly degrade at increasing deflections. To evaluate the deformation capacity of a column beyond the elastic limit, a displacement ductility capacity is defined as the ratio of the displacement corresponding to 20% drop in the strength from the peak value to the yielding displacement. Table 3 shows the displacement ductility capacity in both positive and negative bending moments. The effect of the hoop spacing on the ductility capacity can be seen by the comparison of specimens in S7 series and S12 series. Small hoop spacing (S7 series) can effectively confine the concrete core, and leads to large deformation capacity. Therefore, S7 series of specimens, with low axial load, can increase two displacement ductility factors more than S12 series of specimens. However, hoop spacing has little effect on specimens with high axial load

Table 3
Displacement ductility capacity

Specimen designation	Displacement ductility in positive direction	Displacement ductility in negative direction	Displacement ductility capacity
R4-S7-P4	4	3	3
R4-S12-P4	3	2	2
R8N-S7-P2	7	6	6
R8N-S7-P4	4	3	3
R8N-S12-P2	5	4	4
R8N-S12-P4	3	3	3
R8T-S7-P2	8	7	7
R8T-S7-P4	4	4	4
R8T-S12-P2	6	5	5
R8T-S12-P4	3	3	3

which significantly increases the secondary bending moment to cause early failure of the columns. Furthermore, additional longitudinal bars and cross ties can only slightly enhance the ductility capacity.

3.4. Flexural strengths and code predictions

The experimental ultimate flexural strengths for all specimens under constant axial compression are presented in Table 4. Because all the specimens have the same structural steel shape, R8 series of specimens result in higher flexural strength than R4 series of specimens owing to the four additional longitudinal bars. Due to the confinement effect provided by the close lateral ties, minor increases of the flexural strength, 4%–8%, were observed for S7 series of specimens compared to S12 series of specimens.

A general method is specified in the Eurocode 4 [14] to design a composite column with non-symmetrical or non-uniform cross section. To calculate the flexural strength, plane sections are assumed to remain plane based on the assumption of full composite action up to failure between the steel and concrete components of the column. The ACI Building Code [12] used in the United States essentially has the same design philosophy. Therefore, the ACI provisions are herein used to predict the strengths of the specimens. AISC-LRFD [15] also provides design provisions for composite columns. The provisions used to design composite column in the ACI and the AISC-LRFD are primarily based on the specifications to design for reinforced concrete columns and structural steel columns, respectively. The AISC-LRFD method adopts a bilinear interaction curve between axial compression and flexural strength by the following equations.

$$\text{For } \frac{P_u}{\phi_c P_n} \geq 0.2$$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9} \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0. \quad (4a)$$

Table 4

Test strengths and calculated nominal strengths of composite beam–columns subjected to axial load

Specimen designation	Applied axial load (kN)	Test		ACI		AISC-LRFD		$+\frac{M_{Test}}{M_{ACI}}$	$-\frac{M_{Test}}{M_{ACI}}$	$+\frac{M_{Test}}{M_{LRFD}}$	$-\frac{M_{Test}}{M_{LRFD}}$
		+M (kN m)	−M (kN m)	+M (kN m)	−M (kN m)	+M (kN m)	−M (kN m)				
R4-S7-P4	1656	223.8	−202.9	165	−135	74	−122	1.36	1.50	3.02	1.66
R4-S12-P4	1656	209.2	−202.2	165	−135	74	−122	1.27	1.50	2.83	1.66
R8N-S7-P2	882	234.9	−248.5	187	−191	158	−189	1.26	1.30	1.49	1.31
R8N-S7-P4	1764	263.3	−237.5	180	−169	110	−132	1.46	1.41	2.39	1.80
R8N-S12-P2	882	225.5	−250.8	187	−191	158	−189	1.21	1.31	1.43	1.33
R8N-S12-P4	1764	245.2	−220.8	180	−169	110	−132	1.36	1.31	2.23	1.67
R8T-S7-P2	882	238.8	−260.1	187	−191	158	−189	1.28	1.36	1.51	1.38
R8T-S7-P4	1764	264.6	−254.9	180	−169	110	−132	1.47	1.51	2.41	1.93
R8T-S12-P2	882	227.6	−250.8	187	−191	158	−189	1.22	1.31	1.44	1.33
R8T-S12-P4	1764	247.8	−217.1	180	−169	110	−132	1.38	1.28	2.25	1.64
Average								1.33	1.38	2.10	1.57
COV								0.07	0.07	0.29	0.14

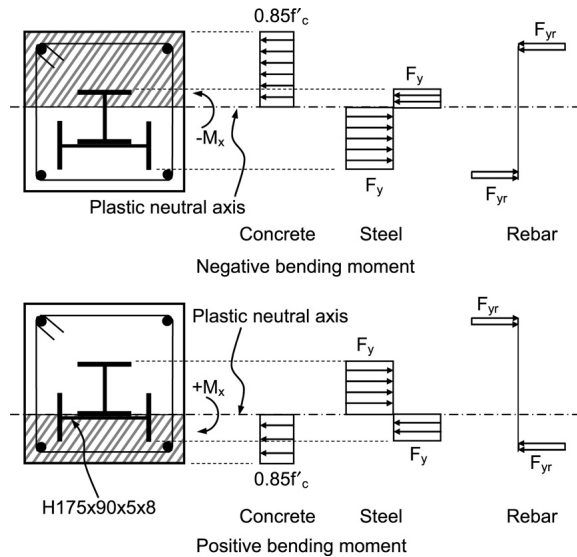


Fig. 13. Plastic stress distribution on composite cross section.

For $\frac{P_u}{\phi_c P_n} < 0.2$

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0 \quad (4b)$$

where P_u is the required compressive strength; P_n is the nominal compressive strength; M_u is the required flexural strength; M_n is the nominal flexural strength; ϕ_c is the resistance factor for compression, 0.85; and ϕ_b is the resistance factor for flexure, 0.90. The nominal flexural strength, M_n , is determined from the plastic stress distribution on the composite cross section. Fig. 13 shows the plastic stress distribution for the R4 specimens. The structural steel and the reinforcing bars are assumed to be fully yielded in tension and compression while the concrete stress in the compression zone is assumed to reach $0.85f'_c$. It should be noted that the plastic stress distribution does not necessarily satisfy the strain compatibility between the concrete and structural steel materials.

A comparison of the test flexural strengths and nominal strengths calculated using the ACI and the AISC-LRFD provisions is presented in Table 4. The nominal strengths are calculated based on the measured yield stress of the materials (Table 2). All the resistance factors in the codes are set to unity. It is observed that the test strengths are all greater than the predicted nominal strengths. In general, the ACI predictions are in reasonable agreement with the test results. The average ratios of the test strengths to the ACI predicted nominal strengths are 1.33 and 1.38 for specimens with positive and negative bending moments, respectively, and the corresponding coefficients of variation are 0.07.

The nominal strengths predicted by the AISC-LRFD method are conservative and result in higher average ratios of test strength to prediction, 2.10 and 1.57 for columns

with positive and negative bending moments, respectively. Especially, the AISC-LRFD method conservatively predicts the positive flexural strengths. The reason is that the plastic neutral axis used to calculate the positive nominal flexural strength, M_n , is very close to the weak axis of the $H175 \times 90 \times 5 \times 8$ steel section, as shown in Fig. 13. Therefore, the contribution of the $H175 \times 90 \times 5 \times 8$ steel section to the nominal flexural strength is obviously insignificant. As a result, conservative nominal flexural strength was obtained. Moreover, the higher ratio of P_u/P_n for P4 series of specimens leads to further conservative predictions because of the consideration of the interaction of flexure and high axial compression.

The nominal strengths predicted by the AISC-LRFD method are more conservative than those predicted by the ACI method, which was also addressed by El-Tawil et al. [16]. It is of interest to compare the test strengths to the design strengths by considering the strength reduction factor or the resistance factor. The current ACI method adopts the same load factors for dead loads and live loads as the AISC-LRFD method, but the strength reduction factor used in the ACI method is 0.65 for a tied column, but is permitted to be increased linearly to 0.90 for small magnitude of the axial compressive load. The resistance factors used in the AISC-LRFD method are 0.85 and 0.90 for axial and flexural strengths, respectively, as shown in Eq. (4). Table 5 presents the comparisons of test strengths and design strengths calculated using the ACI and the AISC-LRFD provisions. The ACI method gives a more consistent prediction in both positive and negative flexural moments than the AISC-LRFD method. The differences in the design strengths of the two design methods are less than those in the nominal strengths because smaller strength reduction factors are adopted in the ACI method than the AISC-LRFD method. For high axial load P4 series of specimens, both codes provide more conservative predictions.

3.5. Energy absorption capacity

Energy absorption capacity is one of the most important indices to evaluate the seismic performance of a structure. Fig. 14 shows the cumulative energy calculated based on the force–displacement curves for R8N and R8T series of specimens because these specimens were subjected to an identical lateral displacement history. The end point of each curve is marked by a symbol. The specimens with low axial load demonstrate much more energy absorption capacity than those with high axial load, as shown in Fig. 14(a) and (b). Moreover, the figure shows that using additional cross ties and small spacing of the hoops results in considerable increase in the cumulative energy absorption. The specimens with 75 mm hoop spacing generally dissipate more energy than those with 125 mm spacing.

4. Conclusions

The experimental results of cyclic load tests on a number of composite beam–columns with T-shaped structural steel lead to the following conclusions.

Table 5
Test strengths and calculated design strengths of composite beam–columns subjected to axial load

Specimen designation	Applied axial load (kN)	Test		ACI		AISC-LRFD		$+\frac{M_{Test}}{\phi M_{ACI}}$	$-\frac{M_{Test}}{\phi M_{ACI}}$	$+\frac{M_{Test}}{\phi M_{LRFD}}$	$-\frac{M_{Test}}{\phi M_{LRFD}}$
		+M (kN m)	−M (kN m)	+ ϕ M (kN m)	− ϕ M (kN m)	+ ϕ M (kN m)	− ϕ M (kN m)				
R4-S7-P4	1656	223.8	−202.9	100	−62	56	−93	2.24	3.27	4.00	2.18
R4-S12-P4	1656	209.2	−202.2	100	−62	56	−93	2.09	3.26	3.74	2.17
R8N-S7-P2	882	234.9	−248.5	123	−118	135	−160	1.91	2.11	1.74	1.55
R8N-S7-P4	1764	263.3	−237.5	97	−85	84	−99	2.71	2.79	3.13	2.40
R8N-S12-P2	882	225.5	−250.8	123	−118	135	−160	1.83	2.13	1.67	1.57
R8N-S12-P4	1764	245.2	−220.8	97	−85	84	−99	2.53	2.60	2.92	2.23
R8T-S7-P2	882	238.8	−260.1	123	−118	135	−160	1.94	2.20	1.77	1.63
R8T-S7-P4	1764	264.6	−254.9	97	−85	84	−99	2.73	3.00	3.15	2.57
R8T-S12-P2	882	227.6	−250.8	123	−118	135	−160	1.85	2.13	1.69	1.57
R8T-S12-P4	1764	247.8	−217.1	97	−85	84	−99	2.55	2.55	2.95	2.19
Average								2.24	2.60	2.68	2.01
COV								0.16	0.18	0.33	0.19

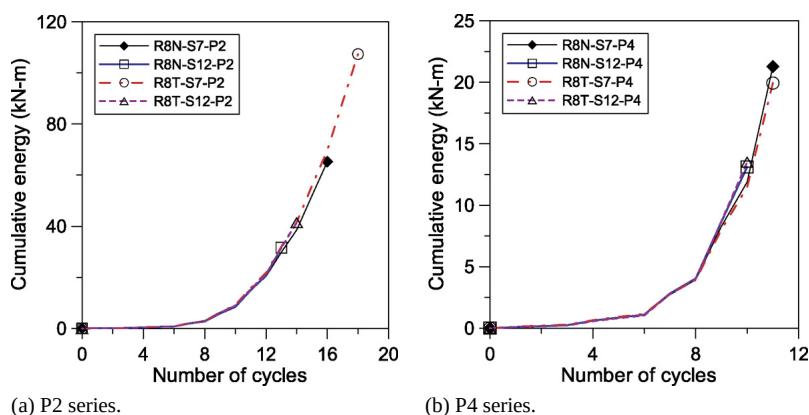


Fig. 14. Cumulative energy absorption.

- (1) The behaviour of an unsymmetrical composite section is considerably different depending on whether the cross section is subjected to a positive or negative bending moment. More structural steel in the compression zone of the cross section subjected to a bending moment leads to a stable hysteresis curve. The crushing and spalling of the concrete and buckling of the longitudinal bars in the compression zone with less structural steel are the cause of pinching of the hysteresis curve.
- (2) The hysteresis curves are highly affected by the level of the applied axial compressive load. With 40% of the squash load as the axial load, the columns demonstrated much less displacement ductility and energy absorption capacity compared to the columns with 20% of the squash load, owing to an appreciable increase in secondary bending moment resulted from the applied axial load.
- (3) The ultimate flexural strength, ductility and energy absorption capacity can be enhanced by providing the cross ties and decreased spacing of the hoops. This is attributed primarily to the increased confinement provided by the transverse reinforcement.
- (4) The comparisons of the experimental flexural strengths with code predictions indicate that the ACI method, based on an assumption of linearly varying strain distribution, leads to a reasonable strength prediction for composite columns with unsymmetrical cross section. The nominal strengths determined by the AISC-LRFD method are more conservative than by the ACI method.

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