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Simulations of partially encased composite column under concentric loading using equivalent steel section

D. Ghosh
Bangladesh University of Engineering and Technology, Dhaka, Bangladesh

M. Begum
Bangladesh University of Engineering and Technology, Dhaka, Bangladesh

ABSTRACT: Partially encased composite column (PEC) consists of thin walled welded H- shaped steel section with transverse links provided at regular intervals between the flanges to inhibit the occurrence of local buckling in the thin flange plates. The space between the flanges and the web plate are filled up with concrete. Extensive experimental investigations have been conducted by several research groups to understand the behavior of this relatively new composite column under both concentric and eccentric loading conditions along with sophisticated non-linear finite element analysis. However, the separation between concrete and steel initiates the unstable condition in the finite element analysis near the ultimate point when flange plate buckles. To avoid the expensive and cumbersome modeling of the behavior at the interface of two dissimilar materials, finite element analysis is conducted using the equivalent steel section of Partially Encased Composite columns under concentric gravity loading.

1 BACKGROUND

Steel-concrete composite columns can replace the use of steel-only columns in the construction of mid-rise and high-rise buildings to improve the behavior and cost efficiency of the structure significantly. A welded H–shaped steel section figures the partially encased composite (PEC) section with concrete infill between the flanges (Fig. 1). In Europe, in the early 1980s, PEC columns and beams were introduced using standard-sized rolled steel sections. In 1996, the Canam Group in North America proposed a PEC column section constructed from a thin–walled built–up steel shape with transverse links provided at regular intervals to restrain local buckling as shown in Figure 1. Extensive experimental research has been performed in Ecole Polytechnique de Montréal (Tremblay et al. 1998;, Chicoine et al. 2002 and Bouchereau and Toupin 2003) on small-scale and large-scale PEC column specimens under various conditions of loading. The influences of high performance materials on the behavior of these columns have also been investigated *e*xperimental by Prickett & Driver (2006) at the University of Alberta. The results of these experimental investigations indicated that the behavior of this composite column is significantly affected by the local instability of the thin steel flanges. Begum et al. (2007) were able to overcome these challenges in the finite element model through the implementation of a dynamic explicit formulation along with a damage plasticity model for concrete and a contact pair algorithm at the steel-concrete interface. The developed model was applied successfully to reproduce the behavior of 34 PEC columns from five experimental programs. However, despite of the accuracy, the composite finite element model developed by (Begum et al. 2007) is very sophisticated due to the presence of two dissimilar materials. Moreover, modeling of the interfacial behavior between steel and concrete requires extensive calculations as well as skilled and experienced users. Due to these complexities most of the structural analysis and design software do not handle such composite members. In this paper an attempt has been made to simulate the behavior of this steel-concrete composite section developing a fictitious steel section of the partially encased composite column. This equivalent steel section has added a new dimension to the analysis of partially encased composite column renovating the numerical procedure.

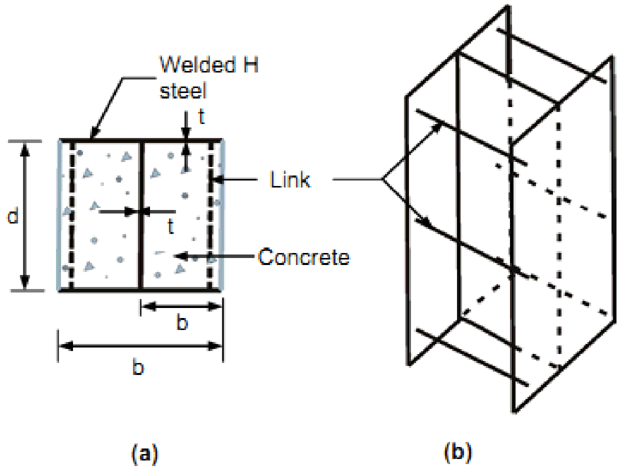


Figure 1. Partially encased composite column , (a) Column Cross-Section and (b) 3D view of the Steel Configuration

2 OBJECTIVES AND SCOPES

The main objective of this study is to numerically simulate the behavior of partially encased composite column made of thin plates, using equivalent steel column. The methodology of Marinopoulou et al. (2007) was followed to develop of the equivalent steel column for composite columns with compact steel sections. The method is to be modified to incorporate the local instability of the thin flanges in PEC columns. To access the accuracy and reliability of the proposed steel section a finite element analysis using the fictitious section is to be performed. Two series of test specimens from the published literature with varying geometric properties are selected for the finite element analysis.

3 REFERENCE TEST COLUMNS

Eight test PEC columns from the published literature are selected for current study. The lists of these specimens, along with their properties, are given in Table 1. Figure 2 shows the cross-sections and steel side elevations of typical test columns. Specimens C-2, C-4, C-5 and C-7 were tested during the initial phase of the research program by Tremblay et al. (1998) to study the behaviour of these columns under concentric gravity loading. Specimens C-2 to C-7, had square cross-sections of 300 mm□x300 mm and 450 mmx450 mm, and a length equal to 5d, where d is the depth of the cross-section. Round mild steel bars of 12.7 mm diameter were used as transverse links in these columns, except specimen C-5 had larger bars of 22.2 mm diameter. Two different link spacing—0.5d and 1.0d—were used in these columns. Specimens C-8 to C-11, tested by Chicoine et al. (2002), also under axial compression, were larger in their cross-sectional dimensions (600 mmx600 mm) as compared to the previous test specimens. As shown in Table 1, most of the geometric properties for these specimens were similar, except specimen C-10, which had a link spacing of 0.5d and specimen C-11, which had a b/t ratio of 31.

All the test columns were fabricated from CSA-G40.21-350W grade steel plate. Normal strength concrete (nominally 30 MPa) was used in the test region of these columns. To strengthen the end regions of these test specimens, high strength concrete of 60 MPa nominal strength was used along with the closer link spacings provided in these zones.

Table 1. Properties of Reference Test Specimens

Reference	Specimen	Plate size b_f x d x t	Link spacing s	Link diameter Φ	Length L	Compressive Strength of concrete f_c	Yield strength of plates f_y
		(mm)	(mm)	(mm)	(mm)	(MPa)	(MPa)
Tremblay et al. (1998	C-2	450x450x9.70	225	12. 7	2250	32. 7	370
	C-4	450x450x9.70	450	12. 7	2250	31. 9	370
	C-5	450x450x9.70	225	22. 2	2250	34. 3	370
	C-7	300x300x6.35	300	12. 7	1500	31. 9	374
Chicoine et al. (2002	C-8	600x600x12.88	600	15. 9	3000	34. 2	360
	C-9	600x600x12.91	600	15. 9	3000	34. 2	360
	C-10	600x600x12.81	300	15. 9	3000	34. 2	360
	C-11	600x600x9.70	600	15. 9	3000	34. 2	345

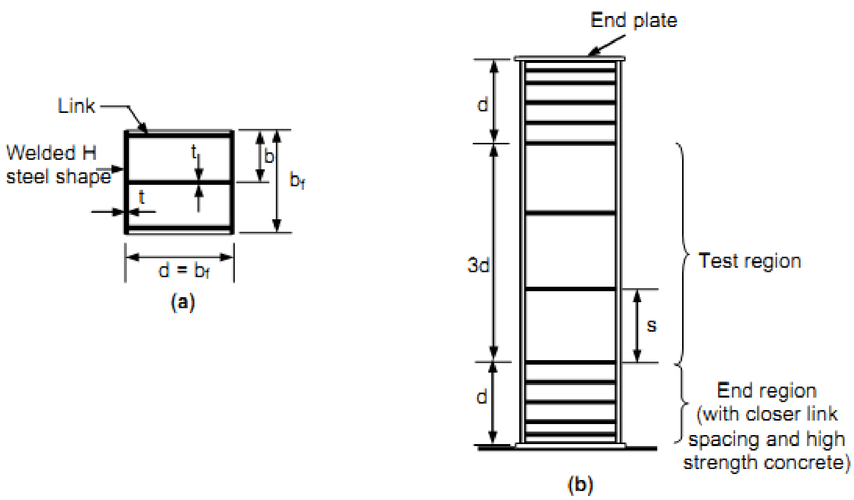


Figure 2. Typical PEC Test Column, (a) Cross-section, (b) Elevation

4 EQUIVALENT STEEL SECTION OF REFERENCE PEC COLUMNS

The equivalent steel section of PEC column is formulated using the methodology proposed by Marinopoulou et al. (2007) with some modifications. The method developed by Marinopoulou et al. (2007) was mainly for partially encased composite sections with fabricated shapes, typically used in Europe. The methodology by Marinopoulou et al. (2007) has been modified in the current study to consider the effect of local buckling in the thin flanges. The formulation of equivalent cross- section of double-symmetrical partially encased composite steel-concrete column is based on three equivalence criteria: compression resistance and bending stiffness about the two principal axes. The fictitious steel cross-section consists of the actual steel cross-section and two additional pairs of plates, one perpendicular to the web at mid-height and one perpendicular to the flanges at mid-width as shown in Figure 3. Plate dimensions are chosen to match the compression resistance and principal bending stiffness of the composite section. Furthermore, the fictitious steel section is constrained to contain the entire actual steel section. For noncompact partially encased composite columns, used in the current study the actual steel section is replaced by an effective steel section (Loove, 1996) to take into account the local buckling of the thin flanges.

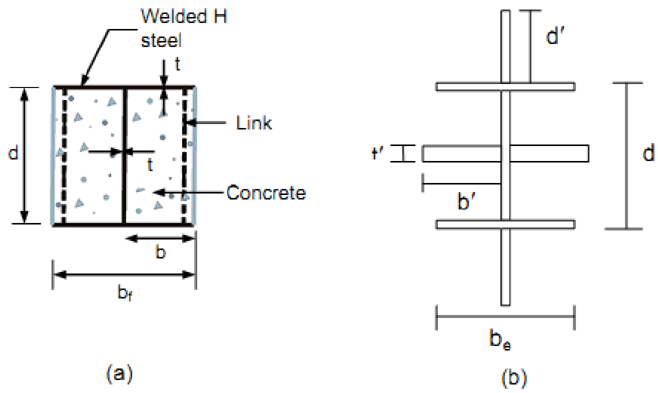


Figure 3: Equivalent steel section for PEC column, (a) Actual cross-section, (b) Equivalent steel section

5 FINITE ELEMENT MODELING

5.1 *Geometric Properties*

The equivalent steel columns of the reference test specimens are analyzed under concentric loading conditions using ABAQUS finite element code (HKS 2007). In order to capture the local buckling behaviour, S4R shell elements were used to model the steel plates. Each node of the S4R shell element has six degrees of freedom—three translations and three rotations. Fixed boundary conditions are applied in the finite element model at the bottom of the column which is similar as that observed in the test specimens. The axial load was applied using displacement control technique at the top surface of the column.

5.2 *Material Properties*

The steel material properties for the plate are defined with a simple elasto-plastic model using a trilinear hardening curve obtained from tensile tests on steel coupons (Tremblay et al. 1998 and Chicoine et al. 2002). The stress and strain data obtained from the uniaxial tension tests are converted to true stress and logarithmic plastic strain using the relationships proposed by Lubliner (1989).

5.3 *Solution Strategy*

In the finite element model geometric nonlinearities are included along with the nonlinear material behavior. Geometric nonlinearities can occur due to the large displacement resulting from the local buckling of the flange plates. Newton-Raphson solution strategy is implemented to trace the nonlinear behavior of the equivalent steel column under concentric axial loading.

6 PERFORMANCE OF THE FINITE ELEMENT MODEL

6.1 *Axial Load and strain behavior*

The evaluation of the performance of finite element model is based on the comparison of the obtained results from the experiments and the finite element model considering entire composite section of the column. Table 2 presents the comparisons between the ultimate axial capacities and corresponding average axial strains obtained from the numerical models and from the experiments. The ratio of the peak load of the equivalent steel column to the test column varied from 0.89 to 1.06 with an average value of 0.97. However, the average axial strain values corresponding to the peak point, of the steel model is found to be much lower than the experimental peak strain values as well as the peak strains obtained from the composite model (2007). This is due to the fact that in formulating the equivalent steel section the linear elastic material behavior is assumed for steel plates. Material nonlinearity was not considered at all in the formulation, for keeping the model simple. The composite model provided more representative strain values as compared to the steel model at the cost of computational time and complexity.

The numerical and experimental load versus average axial strain curves for two representative specimens are shown in Figure 4. The initial portions of the numerical load vs. strain curves obtained from the steel model matched very well with the experimental ones as well as with that obtained from the composite model. However some discrepancies are observed in the strain values near and after the peak point of the load versus strain curves.

6.2 *Failure behavior*

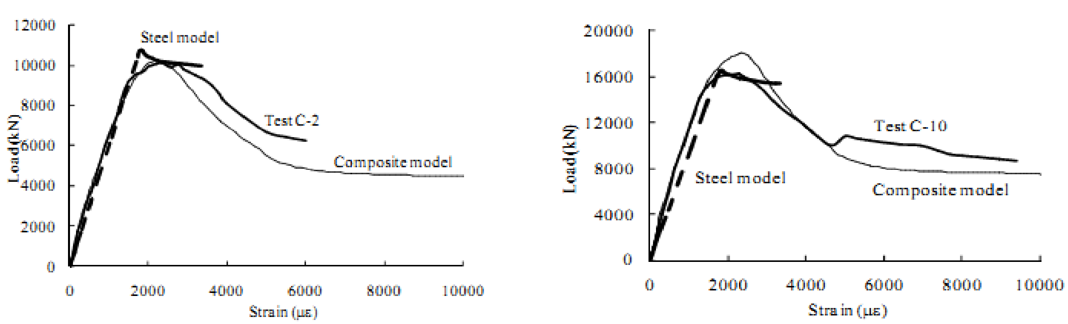
The ultimate capacity of the steel column was observed to attain by the occurrence of local buckling of the steel plates (Fig.4) accompanied by yielding of steel at the same location. Similar behavior was also observed in the steel plates of the composite section at failure.

Table 2. Performance of finite element model of PEC column using equivalent steel section

Specimen design	Peak load			Peak Strain		
	Exp	Numerical		Exp		
	P_{num} (kN)	P_{exp} (kN)	P_{exp}/P_{num}	ξ_{num} (μξ)	ξ_{exp} (μξ)	ξ_{exp}/ξ_{num}
C-2	10700	10100	0.94	1840	2306	1.25
C-4	10560	9390	0.89	1850	1695	0.92
C-5	10610	10000	0.94	1810	2330	1.23
C-7	4640	4280	0.92	1800	2142	1.19
C-8	16620	16470	0.99	1810	1845	1.02
C-9	16620	16610	1.00	1810	1770	0.98
C-10	16530	16240	0.98	1800	2256	1.25
C-11	14050	14930	1.06	1760	1810	1.03
Mean	0.97				1.01	
SD	0.05				0.05	

7 CONCLUSIONS

The composite cross-section of partially encased composite column is replaced by a fictitious section made entirely of steel. The fictitious steel section is restricted to consist of the entire steel section, with two additional steel plates representing the contribution of concrete. The equivalent steel section was used to simulate the behavior of partially encased composite columns under concentric gravity loading only. The effect of local buckling of the thin flanges was accounted through the implementation of effective width of the flange plates. The axial capacity obtained from the finite element simulation is compared to that obtained from the experiments on the reference test columns. The finite element simulations of the composite columns with equivalent steel sections are found to predict the experimental behavior of PEC columns with very good accuracy up to the limit point. However, after the limit point the steel model renders lower strain values as compared to the experimentally obtained values due to the exclusion of material nonlinearities while formulating the fictitious steel section.



Specimen C-2 Specimen C-10

Figure 4: Numerical and experimental load versus strain behavior

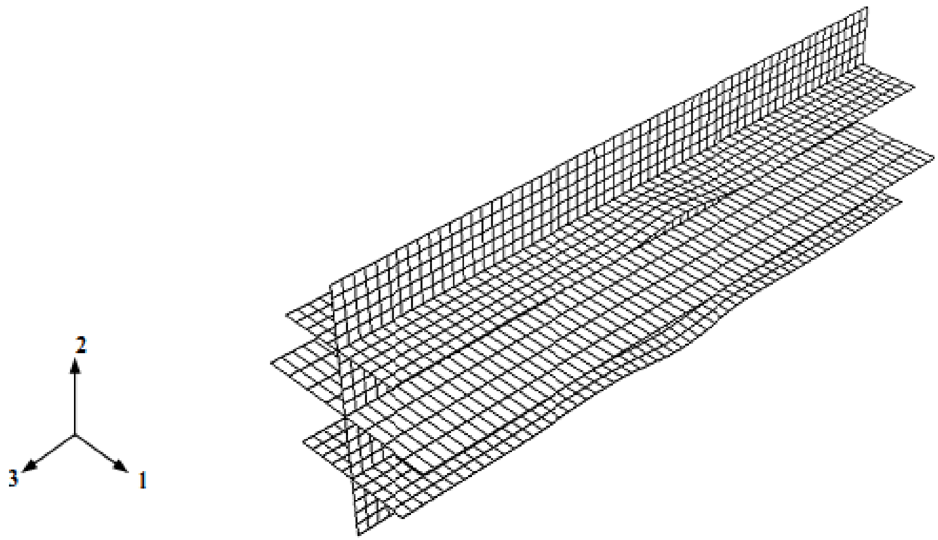


Figure 5: Failure behavior of equivalent steel PEC column

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