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Experimental Testing Of Partially Encased Composite Beam Columns

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Abstract
The past few decades have seen outstanding advances in the use of composite materials in structural applications. There can be little doubt that, within engineering circles, composites have revolutionized traditional design concepts and made possible an unparalleled range of new and exciting possibilities as viable materials for construction.
In addition to the well-known advantages of composite columns, partially encased composite columns offered simplified beam-to-column connection as well as reduced or omitted shuttering thus achieved more cost effective construction. Some companies have patented these new types of partially encased composite column made of light welded steel shapes; moreover, the Canadian Institute of Steel construction CISC has recognized and codified this type of columns.
In This paper, Partially Encased Composite Beam Columns is introduced; experimental studies are made on five partially encased beam columns to investigate the behavior of eccentrically loaded partially encased composite columns using different parameters.
Database subject headings: Composite, columns, encased, partially, equation, steel, slender, plates.

I. Background

The partially encased composite beam columns (PECBC) configuration described in this paper is a relatively recent development in composite structures consisting of thin walled, welded H-shaped steel section with concrete infill cast between the flanges as shown in Figure 1. Transverse links are provided between the flanges at regular intervals to improve the resistance to local buckling. This new composite system was developed to overcome the limitations related to erection, connection design, and economy of more commonly used composite beam column. Studies about this new beam column system were limited; Brent S. Pricket and Robert G. Driver (2006) studied the behavior of partially encased composite columns made with high performance concrete some of the test specimens were loaded under eccentric loading; Mahboba Begum, Robert G Driver and Alaa E Elwi (2007) studied finite element modeling of partially encased composite columns using dynamic explicit method, eccentrically loaded specimens were modeled with different link spacing and load eccentricities and either with or without longitudinal reinforcement.; Saima Ali & Mahboba

Begum (2011) studied the behavior of partially encased slender composite columns in eccentric load; and Christine La Casse (2011) made an experimental and analytical study for PECC using high performance concrete and fiber concrete seven specimens were tested under axial compression and flexure; the failure has been initiated either by the simultaneous compressive crushing of the concrete and the local buckling of the steel flanges or by compressive crushing of the concrete followed by the local buckling in the post-peak range.

II. Objective and Scope of this study

The main objective of this study is to check the behavior and strength of PECBC, using experimental approach along with nonlinear finite element modeling using Cosmos / M program; to investigate the behavior of these eccentrically loaded columns through different parameters.

Three series were examined; series I used to examine the PECBC while series II to examine bare steel beam column and series III to examine reinforced concrete columns. All three series had the same geometric and strength characteristics.

Then a comparison of the ultimate load carrying capacity of the three types of columns was made to know the advantages of the new introduced PECBC against the traditional types of steel or reinforced concrete beam columns.

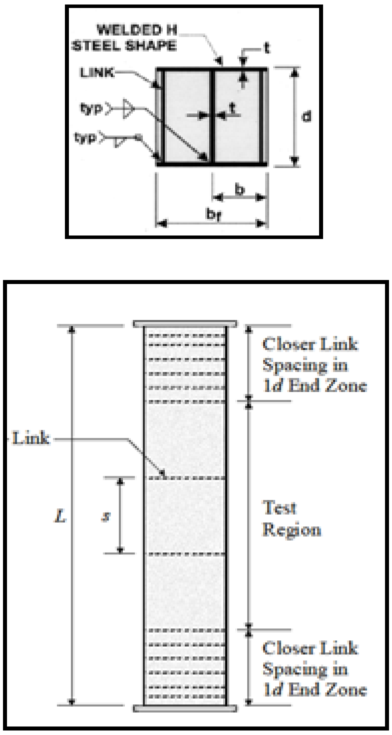

Figure (1) Typical PECBC Illustration

III. Test Program

Five PECBC measuring 100x100 in cross section were constructed as shown in Table 1; three of them with height 500mm; one of height 800mm and one of height 1000mm. H- Steel plates used with thickness 2mm; the flange width-to-thickness ratio for the columns was 25. Two different link spacing were used to study the effect of link spacing on column behavior. Link diameter of 6mm was used. The links are set back form the flange tips so that there is 20mm of clear concrete cover between the link and the concrete face. Four columns were cast with concrete of strength (350 kg/m2) and one was cast with concrete strength (300 kg/m2). The end zone of the columns (100mm of column length at each end) was strengthened to prevent possible failure at these locations due to uneven loading. Additional links were added so that the link spacing was 50mm for the first 100mm of the end zone as shown in Figure (1).

Test specimens P1 through P5 were designed to examine the behavior of eccentrically loaded PEC columns. Three Parameters were tested 1- Links spacing. 2- Concrete compressive strength. 3- Column height to depth ratio. P1 was the datum specimen P2 was used to examine the effect of link spacing as it had link spacing equal to half column depth while the other specimens with ink spacing equals column depth. P3 was used to examine the effect of concrete strength as it had strength of 300 Kg/cm2 while the other specimens, had concrete strength of 350 Kg/cm2. P4 & P5 were used to examine the effect of column height parameter and the effect of height to depth ration as P4 had height to depth ratio of 8 while P5 had height to depth ratio of 10, while P1 through P3 had height to depth ratio of 5.

Four strain gauges and two linear potentiometers are connected to each column as marked below in Figure (2).

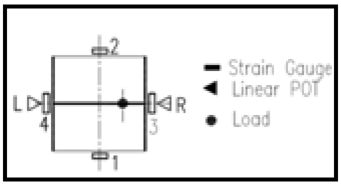

Figure (2) Strain gauges and linear potentiometers numbering

Columns P-1

Specimen P-1 is used to study the effect of the eccentric loading on the behavior of PECBC. Loading was applied and increased gradually from 5 tons up to 20 tons; meters readings and observations are recorded every 5 ton.

Table 1. Specimens Properties

Specimen	2b (mm)	d (mm)	T (mm)	S (mm)	E (mm)	H (mm)	fcu (Kg/cm2)	Fy (Kg/cm2)
P-1	100	100	2.0	100	25	500	350	2350
P-2	100	100	2.0	50	25	500	350	2350
P-3	100	100	2.0	100	25	500	300	2350
P-4	100	100	2.0	100	25	800	350	2350
P-5	100	100	2.0	100	25	1000	350	2350

Then load is increased from 23 tons to 40 tons gradually; readings and observations are recorded every 2 tons; when reaching 30 tons traces of buckling of column flange is observed at the rear side 140 mm from column top (0.3H) with traces of cracks at the column top; at 32 tons; buckling of column flange in the same location increases; and concrete crakes are observed at the column top. The Column buckling starts at 30 tons Load; and 0.75 m.t bending moments; accordingly the failure load is 30 tons and the failure mode is buckling of flange plates associated with concrete crushing; similar to Tremblay et al. (1998). Buckling happened right after the link spacing confinement; and when buckling happens the steel plate leaves the concrete surface thus the confinement of the concrete no longer exists thus the concrete acts alone and cracks appear. By increasing the load the buckling increases and appears in different location of the flange as shown in Figure (3) and the cracks of the concrete appear everywhere as shown in Figure (4); when removing the cracked concrete the web plate also has buckled as shown in Figure (5).

Columns P-2

Specimen P-2 is used to study the effect of the first parameter closer link spacing (s=d/2). Loading was applied and increased gradually from 5 tons up to 20 tons; readings and observations are recorded every 5 ton; then load increased from 22 tons to 36 tons gradually; readings and observations are recorded every 2 tons. Up to 30 ton loading there are no observations of any flange buckling or concrete crushing. At 32 ton load traces of buckling of column flange is observed at the front side 250 mm from column top (0.5H) at the right side and at the bottom from the left side; along with traces of concrete cracks at column top. Column buckling starts at 32 tons Load; and 0.8 m.t bending moments; accordingly the failure load is 32 tons and the failure mode is buckling of flange plates associated with concrete crushing; similar to Tremblay et al. (1998).

The failure load is 2 tons higher than specimen P-1 and this is due to the closer link spacing used; thus closer link spacing enhances the column load capacity and failure mode. Flanges buckling and concrete cracks happened at column mid height Figure (6) & (7).

The column failure zones indications is marked out on the column showing flange plate buckling zone and concrete failure zones Figures (8).

Columns P-3

Specimen P-3 is used to study the effect of the second parameter the different concrete compressive strength (300 Kg/cm2). Loading was applied and increased gradually from 2 tons up to 38 tons; readings and observations are recorded every 2 ton. Up to 26 ton loading there are no observations of any flange buckling or concrete crushing. At 28 ton load separation between the concrete face and the flange plate at the right side at mid height of column is observed. At 30 ton; column flange buckling occurred at height of 330 mm (0.66H) from column top at the right side. Buckling continued to increase at this location up to 34 ton. The column buckling starts at 28 tons load; and 0.7 m.t bending moments; accordingly the failure load is 28 tons and the failure mode is buckling of flange plates associated with concrete crushing. The failure load was 2 tons less than specimen P-1and the failure in both columns is due to buckling of flange plates; but it is obvious that the failure load decreased because of using lower concrete compressive strength thus lower confinement of the concrete to the steel flange plates thus faster buckling; also the failure load was less by 4 tons than specimen P-2 and this is due to the using larger link spacing and less concrete compressive strength. The separation between the concrete and the flange plates happened at the column mid height at the right side of the column Figure (9) (the same side where the load is applied) and the buckling happened at the same side but at 0.66 of the column height Figure (10); this separation is due to the using of lesser concrete compressive strength; this separation made the buckling happed faster than column P-1 or P-2 accordingly lesser column capacity. When buckling happens the steel plate leaves the concrete surface thus the confinement of the concrete no longer exists thus concrete acts alone and cracks appears Figure (11); in this specimen after appearance of flange buckling and of concrete cracks; test continued up to 34 tons and then stopped.

Columns P-4

Specimen P-4 is used to study the effect of the third parameter the column height to depth ratio (H/d = 8) on the behavior of partially encased composite column under the effect of eccentric loading.

Load was applied and increased gradually from 5 tons up to 15 tons; readings and observations are recorded every 5 ton. Then loads was increased from 15 tons to 25 tons gradually; readings and observations are recorded every 2 tons. Up to 23 ton loading there are no observations of any flange buckling or concrete crushing. At 25 ton load; column flange buckling occurred at height of 300 mm (0.375H) from column top at the right side; along with concrete cracks at height of 120mm from column top; then test stopped. The column buckling starts at 25 tons Load; and 0.625 m.t bending moments; accordingly the failure load is 25 tons and the failure mode is buckling of flange plates associated with concrete crushing. The failure load is decreased by 5 tons than specimen P-1 and this is due to using longer column (H/d=8); the flange buckling happened faster due to the usage of longer column Figure (12); the failure mode is similar to previous studies failure mode ; column flange buckling starts and hence concrete cracks appears. When buckling occurred the flange steel plate leaves the concrete surface thus the confinement of the concrete no longer exists thus the concrete acts alone and cracks appears Figure (13).

Columns P-5

Specimen P-5 is used to study the effect of the third parameter the column height to depth ratio but with (H/d = 10) on the behavior of PECBC. Load was applied and increased gradually from 5 tons up to 15 tons; readings and observations are recorded every 5 ton. Then loads increased from 15 tons to 24.5 tons gradually; readings and observations are recorded every 1 tons. Up to 24 ton loading there are no observations of any flange buckling or concrete crushing. At 24.5 ton load; column flange buckling occurred at height of 150 mm (0.15H) from column bottom at the right side Figure (14); along with concrete cracks at height of 200 mm from column top Figure (15) and bottom; then test stopped. The column buckling starts at 24.5 tons Load; and 0.61 m.t bending moments; accordingly the failure load is 24.5 tons and the failure mode is buckling of flange plates associated with concrete crushing; the failure load is decreased by 5.5 tons than specimen P-1 and this is due to using longer column (H/d=10); which is similar to P-4 with only difference of 0.5 t which is due to increasing the column height to depth ratio from 8 to 10. The flange buckling happened faster due to the usage of longer column; the failure mode is similar to previous studies failure mode; column flange buckling starts and hence concrete cracks appears. When buckling occurred the flange steel plate leaves the concrete surface thus the confinement of the concrete no longer exists thus the concrete acts alone and cracks appears.

Bare Steel and Reinforced Concrete Specimens

Four Bare steel columns measuring 100x100 in cross section were constructed two of them with height 500mm, one of 800mm height and one of 1000mm height; with specimen properties as shown in table 2 same properties as the PECBC; and five reinforced concrete columns measuring 100x100 in cross section were constructed, three of them with height 500, one of 800 height and one of 1000mm height; with specimen properties as shown in table 3 below. The longitudinal reinforcement used was with same yield strength and area as of the steel plates used for the PECBC and the steel columns. All the specimens were tested using the same machine and under the same conditions as the PECBC specimens; loading was applied and results were recorded. Figure (16) & (17) shows the steel and RC specimens after testing.

Table 2. Bare Steel Specimens

Specimen	2b (mm)	d (mm)	t (mm)	S (mm)	e (mm)	H (mm)	F_y (Kg/cm^2)
S-1	100	100	2.0	100	25	500	2350
S-2	100	100	2.0	50	25	500	2350
S-4	100	100	2.0	100	25	800	2350
S-5	100	100	2.0	100	25	1000	2350

Table 3. Reinforced Concrete Specimen

Specimen	b (mm)	d (mm)	RFT (mm)	S (mm)	e (mm)	H (mm)	f_{cu} (Kg/cm^2)	F_y (Kg/cm^2)
C-1	100	100	12Dia 8.0	100	25	500	350	2400
C-2	100	100	12Dia 8.0	50	25	500	350	2400
C-3	100	100	12Dia 8.0	100	25	500	300	2400
C-4	100	100	12Dia 8.0	100	25	800	350	2400
C-5	100	100	12Dia 8.0	100	25	1000	350	2400

PECBC Testing Results

Failure mode for the five specimens is nearly the same; Column flange buckling happens thus separation between the steel flange plate and the concrete surface happens; thus concrete is no more confined by the steel plates and resist the load alone thus concrete cracks appears and hence concrete crushing; the failure mode is typical to previous experimental studies happened for concentric PEC columns testing. Column's flanges buckling and concrete crushing. Column load capacity differs for each specimen and it is obvious that the highest column capacity is specimen P-2 with closer link spacing S=d/2 and column height = 5 d and concrete compressive strength of fcu = 350 kg/cm2; while specimen P-1 comes the second one as the link spacing s=d with column height = 5 d and concrete compressive strength of fcu= 350 kg/cm2; then comes specimen P-3 with link spacing = d with column height = 5 d but with concrete compressive strength of fcu= 300 kg/cm2 Figure; then specimen P-4 with link spacing s=d and concrete compressive strength of fcu= 350 kg/cm2 but with column height = 8d; and the last one is specimen P-5 with link spacing s=d and concrete compressive strength of fcu= 350 kg/cm2 but with column height = 10d. Table 4 below shows the specimen failure load.

Table 4. PECBC Specimens Failure Load

Specimen No.	**PECBC Specimen Failure Load**	
	Normal Load (t)	Bending Moment (m.t)
P-1	30	0.75
P-2	32	0.8
P-3	28	0.7
P-4	25	0.625
P-5	24.5	0.6125

Conclusion

The behavior of partially encased composite beam column was assessed using experimental testing

The Column load capacity differs for each specimen; the highest column capacity is specimen P-2 with closer link spacing S=d/2 and column height = 5 d and concrete compressive strength of fcu = 350 kg/cm2; while specimen P-1 comes the second one as the link spacing s=d with column height = 5 d and concrete compressive strength of fcu= 350 kg/cm2; then comes specimen P-3 with link spacing = d with column height = 5 d but with concrete compressive strength of fcu= 300 kg/cm2but; then specimen P-4 with link spacing s=d and concrete compressive strength of fcu= 350 kg/cm2 but with column height = 8d; and the last one is specimen P-5 with link spacing s=d and concrete compressive strength of fcu= 350 kg/cm2 but with column height = 10d.

PECBC capacity is almost triple the steel columns specimens; hence the addition of concrete to the steel columns to cover the web plate; enhanced the column behavior due to the confinement of the steel column creating the PECB; the concrete and the steel section acted compositely due to the existence of the steel links which acted as shear connectors. The column gained the higher capacity while still gaining the advantage of steel structure column as the flanges are exposed for easier beam column connection; on the other hand using slender steel plates gives the advantage of using light sections thus more lesser crane capacity and less project cost.

PECBC capacity is less than the reinforced concrete beam column capacity by about 17 to 27 %; hence for the same column price (as both series had the same concrete and steel volume) the RC columns had more column capacity by average 22% over the PECBC; but still had the disadvantage for longer construction durations and more complicated fabrication process.

The 22% less capacity can be gained by either using higher concrete strength or thicker steel plates for the PECBC which can be done with minimum cost and accordingly have the same RC column capacity and the advantages of the PECB Columns.

Notations

PECBC = Partially Encased Composite Beam Columns
RC = Reinforced Concrete
b = Half flange width
d =Specimen depth
t = Plate thickness
s = Vertical link spacing
fcu= Concrete cubic compressive strength
Fy= Steel yield strength
H = Column height
Puexp.= Experimental failure load
CISC= Canadian Institute for Steel Construction
L= Load
SG= Strain Gauge
Lin Pot= Linear Potentiometer

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Figure (3) P-1 Flange Bucklin


Figure (4) P-1 Concrete Crack


Figure (5) P-1 Web Buckling


Figure (6) P-2 Flange Buckling


Figure (7) P-2 Concrete Cracks


Figure (8) P-2 Failure zones

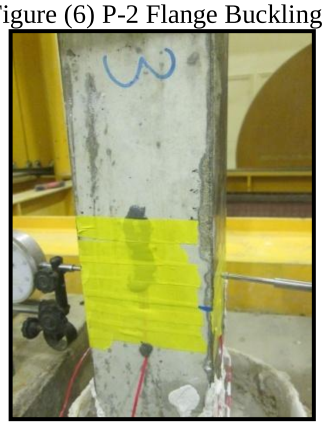
Figure (9) P-3 Flange separation


Figure (10) P-3 Flange Buckling


Figure (11) P-2 Failure zones

Figure (12) P-4 Flange Buckling

Figure (13) P-4 Concrete Cracks

Figure (14) P-5 Flange Buckling

Figure (15) P-5 Concrete Cracks

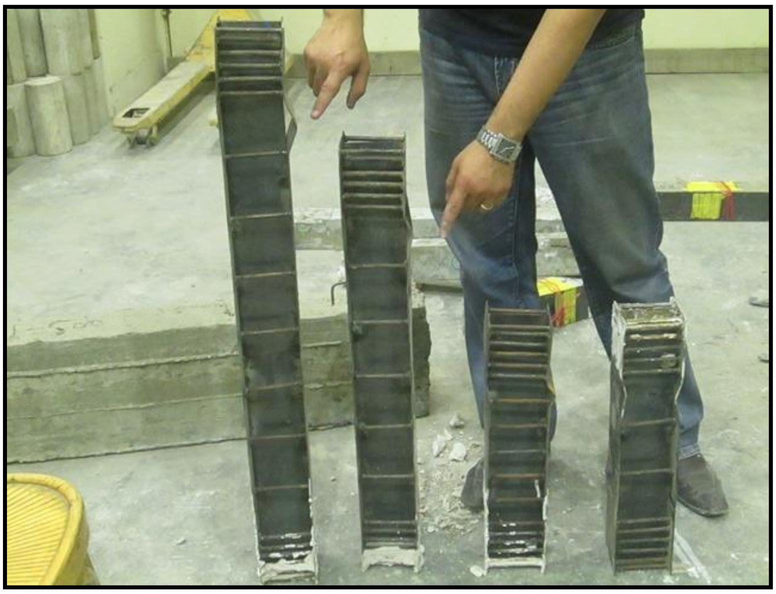

Figure (16) Bare Steel Specimens after Failure

Figure (17) Reinforced Concrete Specimens after Failure