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Research Article

A comparative study between buckling behaviour and statistical analysis of axially loaded fully encased composite columns made with high strength concrete

Sasikumar P[1]

[1] JCT College of Engineering and Technology, Anna University, Coimbatore (India); sasiserene@gmail.com
*Correspondence: sasiserene@gmail.com

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Abstract: This research presents a buckling behaviour of axially loaded fully encased composite columns made with high strength concrete (HSC). Additionally, the research includes experimental study, numerical analysis, and statistical analysis. Three fully encased composite columns (FECC) cast with different cross sections are 200mm x 200mm, 230mm x 200mm, 250mm x 200mm, and 2800mm, respectively. The FECC specimens were made with 80MPa grade of concrete and ISMB 100 x 50 x 7 x 4.2 steel section, including the experimental, analytical, and numerical analyses. The buckling resistance of FECC specimen results were compared to Euro code – 1994 and American Institute of Steel of Construction 360 – 2010 codes. Finite element analysis (FEM) studied the FECC specimens and predicted the buckling resistance compared to the experimental test results. The FEM analysis closely correlated to the experimental results. The statistical analysis was performed on the FECC specimens with current design codes and previous experimental research work. The statistical study compared the experimental test results of buckling resistance and flexural stiffness of FECC specimens to the predicted values from codes. The numerical and statistical analysis results were compared to the experimental test results. Also, it is highly correlated and helps the performance of columns.

Keywords: High strength concrete, fully encased composite columns, buckling behavior, finite element analysis.

1. Introduction

Generally, concrete encased steel (CES) composite columns are typically provided in high-rise buildings, deep base foundations, and metro railway buildings. The CES columns are made with high-strength concrete and steel sections. Commonly, CES columns were constructed with one or more steel sections and mode composite with the surrounding concrete, longitudinal and lateral reinforcement. In some cases, beam-column joints or connections provide the shear studs to transfer the reaction force from the beam to the column section (EN 1994-1-1). Many research works investigated the experimental behaviour of concrete-encased composite long columns under axial compression (Chen & Yen 1996; Lai et al., 2019; Tsai et al., 1996; Han 2004; Zhu et al., 2014), eccentric loading (Claeson & Gylltoft 1998; Kim et al., 2012; Kim et al., 2013; Matsui et al., 1979; Wang 2007; Yu & Lu 2009), biaxially loading (Munoz et al., 1997; Tokgoz & Dundar 2008), monotonic and cyclic loading (Campian et al., 2014; Campian et al., 2015), partially encased composite columns (Begum et al., 2013; Pererala et al., 2016), numerical simulation finite element analysis was conducted (Begum et al., 2007; Ello-body & Young 2011).

In the current project, three high-strength concrete fully encased composite columns were investigated with 80MPa and ISMB100. The compressive strength of concrete is beyond the designed concrete compressive strength. The main rebar and tie bar were used in 12mm and 8mm, respectively, and spacing of transverse reinforcement was provided throughout the FECC specimens is 100mm c/c, the 3D model shown in Figure 1. The experimental test results of FECC specimens were compared with the analytical and numerical prediction. The statistical study predicts and estimates the buckling resistance of high-strength concrete FECC specimens.

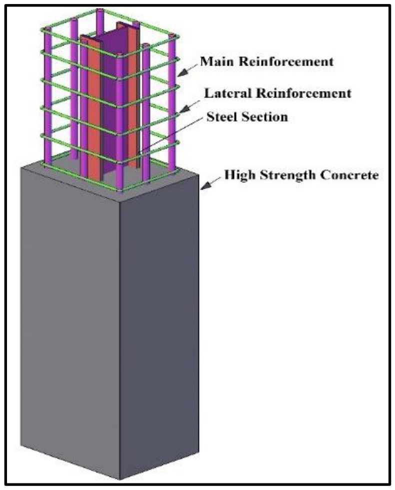


Figure 1. 3D model of fully encased composite column specimen.

2. Experimental study

2.1. *Details of the FECC specimens and details*

The FECC specimens were cast with different cross-sections, as illustrated in Figure 2. The length of the columns is 2800 mm, and end conditions are hinged at both ends. 4, 6 and 8 numbers of 12mm deformed bars were used in the primary reinforcement in FECC-1, FECC-2 and FECC-3, respectively, and 8mm lateral reinforcement provided 100mm c/c throughout the columns. The FECC specimens were cast and tested after curing periods of 28 days. Three cubes were cast and tested on the same day after curing periods. The compressive strength is 79MPa. Geometry details and physical proper-ties of materials are listed in Table 1.

Table 1. Geometric and physical properties of materials.

Specimens ID		FECC-1	FECC-2	FECC-3
Geometric properties	Cross section (mm)	200 x 200	230 x 200	250 x 200
	Length (mm)	2800	2800	2800
	Slenderness ratio	14	14	14
	Steel section (mm)	100 x 50 x 7 x 4.2	100 x 50 x 7 x 4.2	100 x 50 x 7 x 4.2
	Tie spacing (mm)	100	100	100
	Steel area ratio (%)	2.85	2.48	2.28
	Reinforcement ratio (%)	1.13	1.47	1.81
Physical properties	f_c (MPa)	79	79	79
	f_{ss} (MPa)	250	250	250
	f_{sl} (MPa)	500	500	500
	E_c (GPa)	30	30	30
	E_{ss} (GPa)	210	210	210
	E_{sl} (GPa)	200	200	200

Note: f_c, f_{ss}, and f_{sl} are the compressive strength of concrete and yield tensile strength of steel section and rebar; E_c, E_{ss} and E_{sl} are the modulus of elasticity of concrete, steel section and rebar.

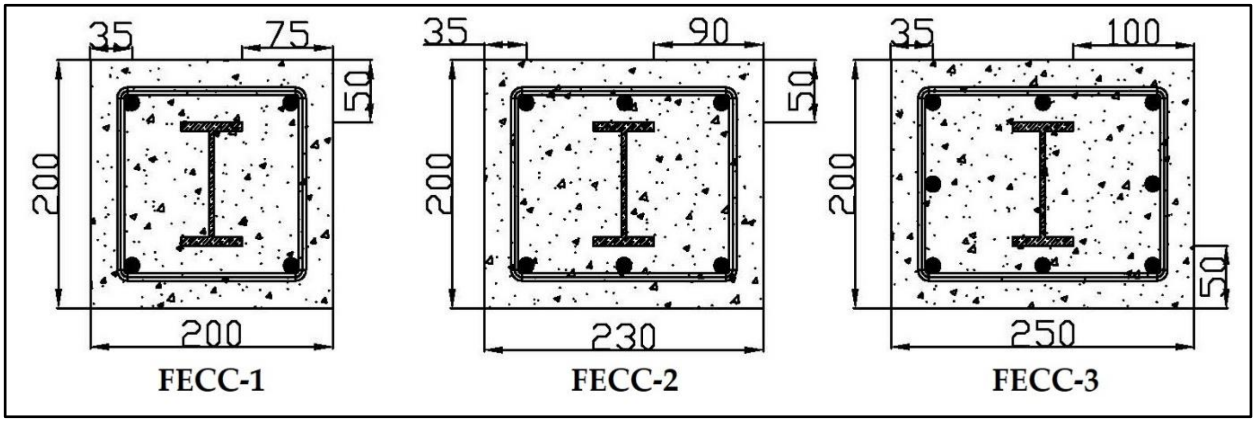


Figure 2. Cross-sectional details of fully encased composite column specimens.

2.2. *Experimental setup and results of the specimens*

The column specimens were tested under pure compression in loading the frame capable 500 T; also, the columns were tested under displacement control mode. A total of eight Linear Variable Displacement Transducers (LVDTs), two LVDTs were placed on the top end plate to measure the vertical displacements and the remaining six LVDTs were placed on the two sides to measure lateral displacement, as shown in Figure 4. The axial compression load is applied in a displacement control mode. The loading rate is 0.5mm/min until the failure of specimens.

All FECC specimens failed the same behaviour. All columns' lateral and vertical displacements are plotted as shown in Figure 3. The column axial load-vertical displacement curve is linear behaviour up to 70% of axial load; after that, the curve will be elastic to the plastic stage. After reaching the ultimate load, the axial load gradually decreases, and it fails in brittle behaviour. The experimental peak load is presented in Table 2. The highest axial load-carrying capacity is noted in the FECC-3 specimen, and the remaining two specimens had a low axial load. The failure stage of columns, concrete cover spalling and buckling of longitudinal is noted. The axial load carrying capacity and prevention of concrete cover spalling of columns depend on the transverse reinforcement.

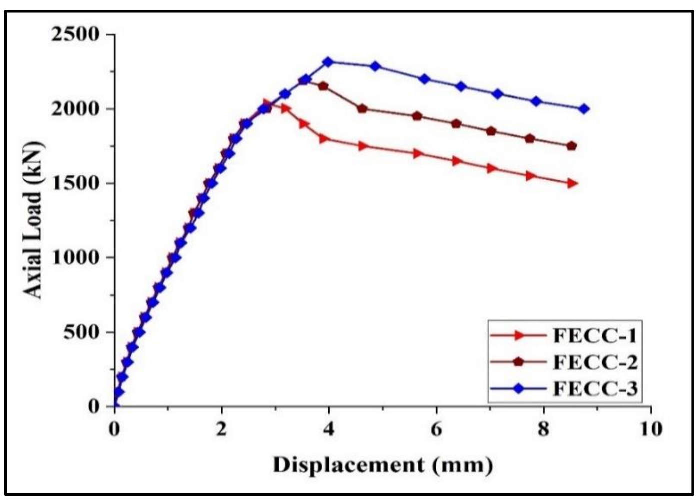


Figure 3. Axial load versus displacement curve for FECC specimens.

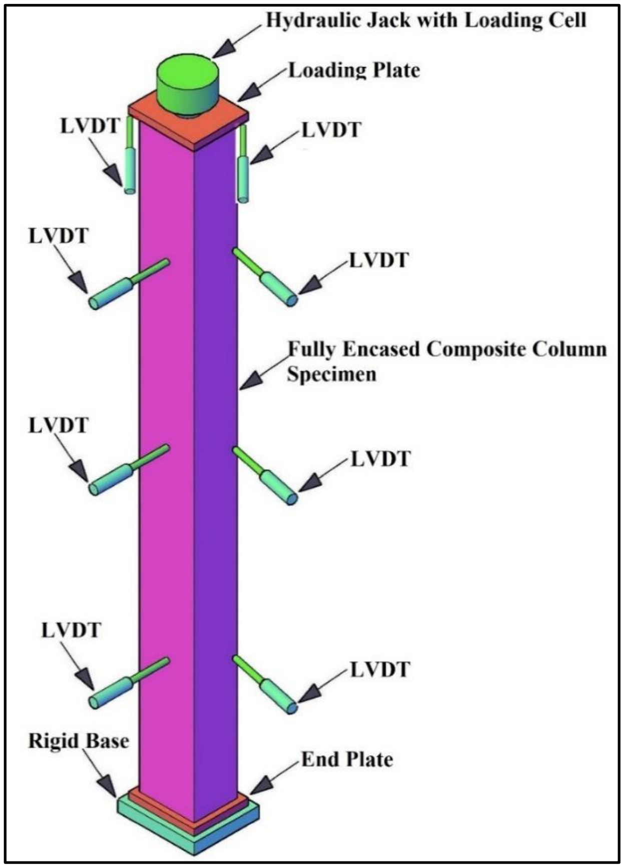


Figure 4. Experimental setup model of FECC specimen.

3. Analytical study

The analytical study used two codes (EC4 & AISC) to predict the buckling resistance of fully encased composite columns. While the two methods for buckling resistance involve multiplying a reduction factor to the axial cross-section resistance, the detailed treatments of the column slenderness effect differ Lai et al. (2019). Additionally, there is composite action between steel and concrete. The buckling resistance of FECC specimens was calculated as per EC4 and AISC using Eqs. (1) - (4) (EC4) and Eqs. (5) - (7) (AISC). An analytical study evaluated the experimental results is presented in Table 2. The buckling resistance of FECC specimens is compared to the EC4 and AISC codes; EC4 is given a better correlation than the AISC.

$$\lambda = \sqrt{N_{pl}/N_{cr}} \quad (1)$$

$$\varphi = 0.5*[1 + \acute{\alpha}\,(\lambda - 0.2) + \lambda^2] \quad (2)$$

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \lambda^2}} \quad (3)$$

$$N_{bl} = \chi N_{pl} \quad (4)$$

where,
$N_{pl} = 0.85A_c f_c + A_{ss} f_{ss} + A_{sl} f_{sl}$

$N_{cr} = \frac{\pi^2 EI_{eff}}{(KL)^2}$
$EI_{eff} = 0.6E_cL_c + E_{ss}L_{ss} + E_{sl}L_{sl}$

$$\lambda = \sqrt{P_{no}/P_e} \tag{5}$$

$$N_{bl} = P_{no}(0.658)^{\lambda} \tag{6}$$

$$N_{bl} = 0.877P_e \tag{7}$$

where,
$P_n = 0.85A_cf_c + A_{ss}f_{ss} + A_{sl}f_{sl}$
$P_e = \frac{\pi^2 EI_{eff}}{(KL)^2}$
$EI_{eff} = C_1E_cL_c + E_{ss}L_{ss} + 0.5E_{sl}L_{sl}$
$C_1 = 0.1 + 2\,(A_s / (A_c + A_s)) \leq 0.3$

Table 2. Comparison between experimental and analytical results

Specimen ID	Experiment load (kN)	EC4 (kN)	P_{expt}/EC4	AISC (kN)	P_{expt}/AISC
FECC-1	2034	1870	1.09	1663	1.22
FECC-2	2187	2010	1.09	1860	1.18
FECC-3	2315	2099	1.10	2006	1.15
	Mean		1.09		1.18
	CV (%)		0.81		2.98

4. Numerical study

4.1. Finite element method (FEM)

The Finite Element Analysis (FEA) was performed in ANSYS software to analyse the buckling resistance of fully encased composite column specimens. The FEA was conducted by various mesh sizes (coarse, medium and fine). Based on the mesh-sensitive analysis, the fine mesh performs better than the remaining two mesh sizes. The FECC specimens are analysed with hinged end conditions, as shown in Figure 5.

4.2. Comparison of experimental study and numerical analysis

Numerical results were compared with experimental test results, and the numerical results were highly correlated with the experimental test results. The experimental FECC 1-3 specimens' axial load-carrying capacity is low compared to the numerical analysis results given in Table 3. The axial load versus displacement response of experimental and numerical are the same behaviour, up to 70% of axial load. After peak load, the specimens gradually failed, as displayed in Figure 6.

Table 3. Comparative study of experimental and numerical results.

Specimen ID	Experiment load (kN)	Numerical load (kN)	P_{expt} / P_{Num}
FECC-1	2034	2046	0.99
FECC-2	2187	2198	0.99
FECC-3	2315	2329	0.99
	Mean		0.99
	CV (%)		0.55

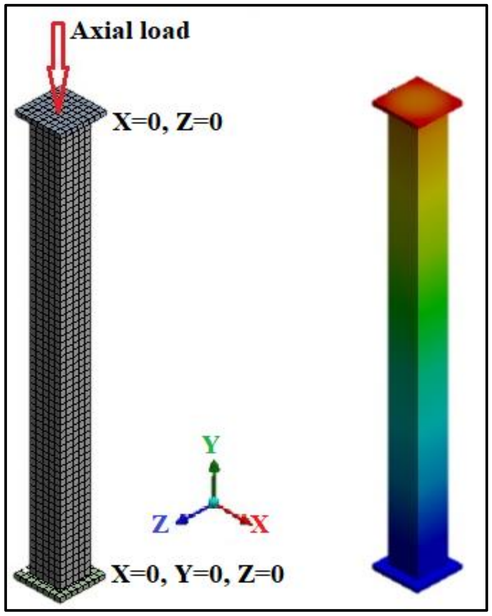
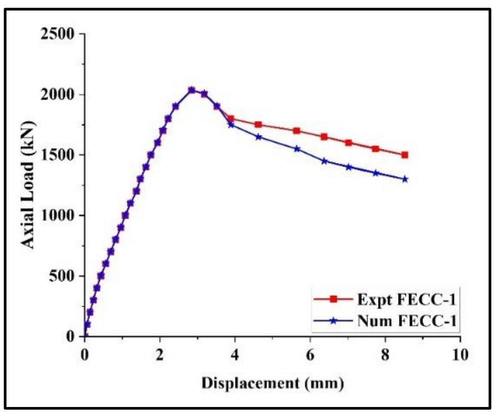
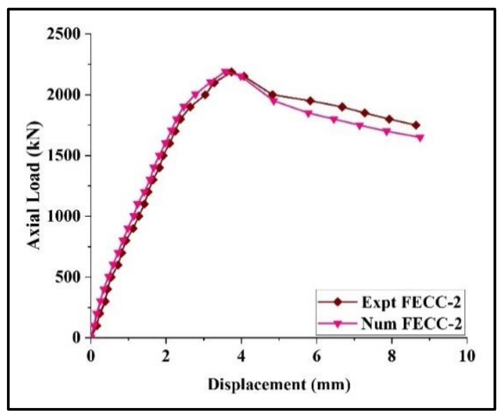


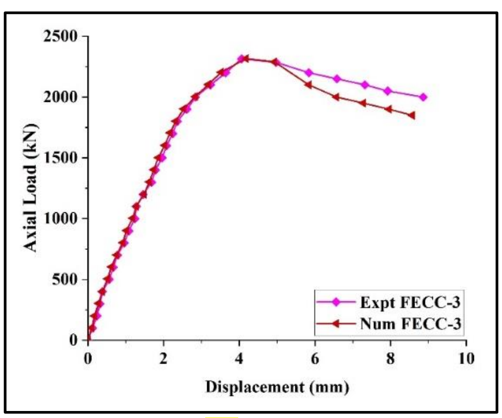
Figure 5. Finite element analysis of FECC specimens.



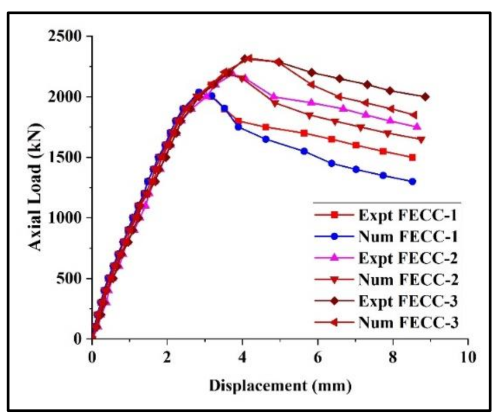
(a). FECC-1



(b). FECC-2



(c). FECC-3



(d). FECC 1-3

Figure 6. Axial load versus displacement of FECC specimens.

5. Statistical analysis

The statistical analyses were conducted with previous literature and compared with the current research work. The statistical analysis included the buckling resistance, flexural stiffness, concrete compressive strength, yield strength of steel, slenderness ratio and steel ratio. The statistical analyses reported are in Table 4 - 6; the current research work is highly correlated with the previous literature survey illustrated in Figures 7 and 8. The flexural stiffness is calculated by following Eqs. (8) – (10):

$$\text{EC4 } EI_{eff} = 0.9\,(0.5E_cI_c + E_{ss}I_{ss} + E_{sl}I_{sl}) \quad (8)\ (\text{EC4})$$
$$\text{AISC } EI_{eff} = C_1E_cI_c + E_{ss}I_{ss} + 0.5E_{sl}I_{sl} \quad (9)\ (\text{AISC})$$
$$\text{ACI } EI_{eff} = 0.2E_cI_c + E_{ss}I_{ss} + E_{sl}I_{sl} \quad (10)\ (\text{ACI})$$

The presented experimental results were compared with the previous literature data (Zhu et al., 2014; TG20 1979; Matsni 1979; Wang 2007; Yu & Lu 2009; Kim et al., 2012; Kim et al., 2013; Binglin et al., 2019) and various codes (EC4 and AISC). The statistical analysis provides a detailed interpretation of the results compared with the results of previous studies. Based on the statistical study, the EC4 code shows superior correlation prediction results compared to the AISC-360 code. The mean and coefficient of variation for EC4 are 1.39 and 0.28, while for AISC-360, they are 1.65 and 0.65%, respectively.

Table 4. Geometric details and physical properties from the literature survey.

ID	B x D x L (mm)	Steel section (mm)	f_c (MPa)	f_{ss} (MPa)	f_{sl} (MPa)	Ref.
SRHC-1	160 x 180 x 2800	110	65.6	379	358	Zhu et al (2014)
SRHC-2	160 x 180 x 2800	110	59.8	379	358	
SRHC-3	160 x 180 x 2800	110	55.7	379	358	
SRHC-4	160 x 180 x 2800	110	50.7	379	358	
SRHC-5	160 x 180 x 2800	110	53.6	379	358	
SRHC-6	160 x 180 x 2800	110	67	379	358	
1	165.1 x 177.8 x 1168	127 x 114 x 29.76	18	248	376	TG20 (1979)
2	165.1 x 177.8 x 2083	127 x 114 x 29.76	18	248	376	
3	165.1 x 177.8 x 2997	127 x 114 x 29.76	18	248	376	
4	165.1 x 177.8 x 3886	127 x 114 x 29.76	18	248	376	
5	203.2 x 254.0 x 2134	203 x 152 x 52.09	18	248	376	
6	254.0 x 304.8 x 2134	203 x 152 x 52.09	18	248	376	
7	340.5 x 355.6 x 2134	203 x 152 x 52.09	18	248	376	
A-1	160 x 160 x 924	100 x 100 x 8 x 6	18.5	306	376	Matsui (1979)
B-2	160 x 160 x 2309	100 x 100 x 8 x 6	21.4	298	376	
C-3	160 x 160 x 3464	100 x 100 x 8 x 6	22.5	304	376	
D-4	160 x 160 x 4619	100 x 100 x 8 x 6	20.6	304	376	
SRC-E1	200 x 200 x 1400	112	47.9	400	350	Wang (2007)
SRC-E2	200 x 200 x 1800	112	54.1	400	350	
SRC-E3	200 x 200 x 1400	110	47.5	400	350	
SRC-E4	200 x 200 x 1800	110	61.9	400	350	
SRC-E5	200 x 200 x 1400	110	49.8	400	350	
SRC-E6	200 x 200 x 1800	110	51.5	400	350	
SRC-E7	200 x 200 x 1400	112	53.1	379	361	
SRC-E8	200 x 200 x 1800	112	45.7	379	361	
SRC-1	160 x 180 x 2600	110	42.6	280.5	380.3	Yu & Lu (2009)
SRC-2	160 x 180 x 2600	110	42.6	280.5	380.3	
SRC-3	160 x 180 x 3200	110	42.6	280.5	380.3	
SRC-4	160 x 180 x 3200	110	42.6	280.5	380.3	

SRC-5	160 x 180 x 4100	110	42.6	280.5	380.3	
SRC-6	160 x 180 x 4100	110	42.6	280.5	380.3	
SRC-7	160 x 180 x 3200	110	42.6	280.5	380.3	
SRC-8	160 x 180 x 3200	110	42.6	280.5	380.3	
C1	260 x 260 x 2620	100 x 150 x 17.6 x 17.6	96	913	525	Kim et al. (2012)
C2	260 x 260 x 2620	100 x 150 x 17.6 x 17.6	96	913	525	
C3	260 x 260 x 2620	100 x 150 x 17.6 x 17.6	96	913	525	
C4	260 x 260 x 2620	100 x 150 x 17.6 x 17.6	96	913	525	
C10	260 x 260 x 2620	150 x150 x 15 x 15	104	812	512	Kim et al. (2013)
C11	260 x 260 x 2620	150 x150 x 15 x 15	104	812	512	
C12	260 x 260 x 2620	150 x150 x 15 x 15	104	812	512	
CES-1	250 x 250 x 2800	130 x 115 x22 x 14	96	380	520	Binglin et al. (2019)
CES-2	240 x 320 x 2800	126 x 113 x 20 x 13	96	380	520	
CES-3	200 x 300 x 2800	100 x 100 x 10 x 6	96	380	520	

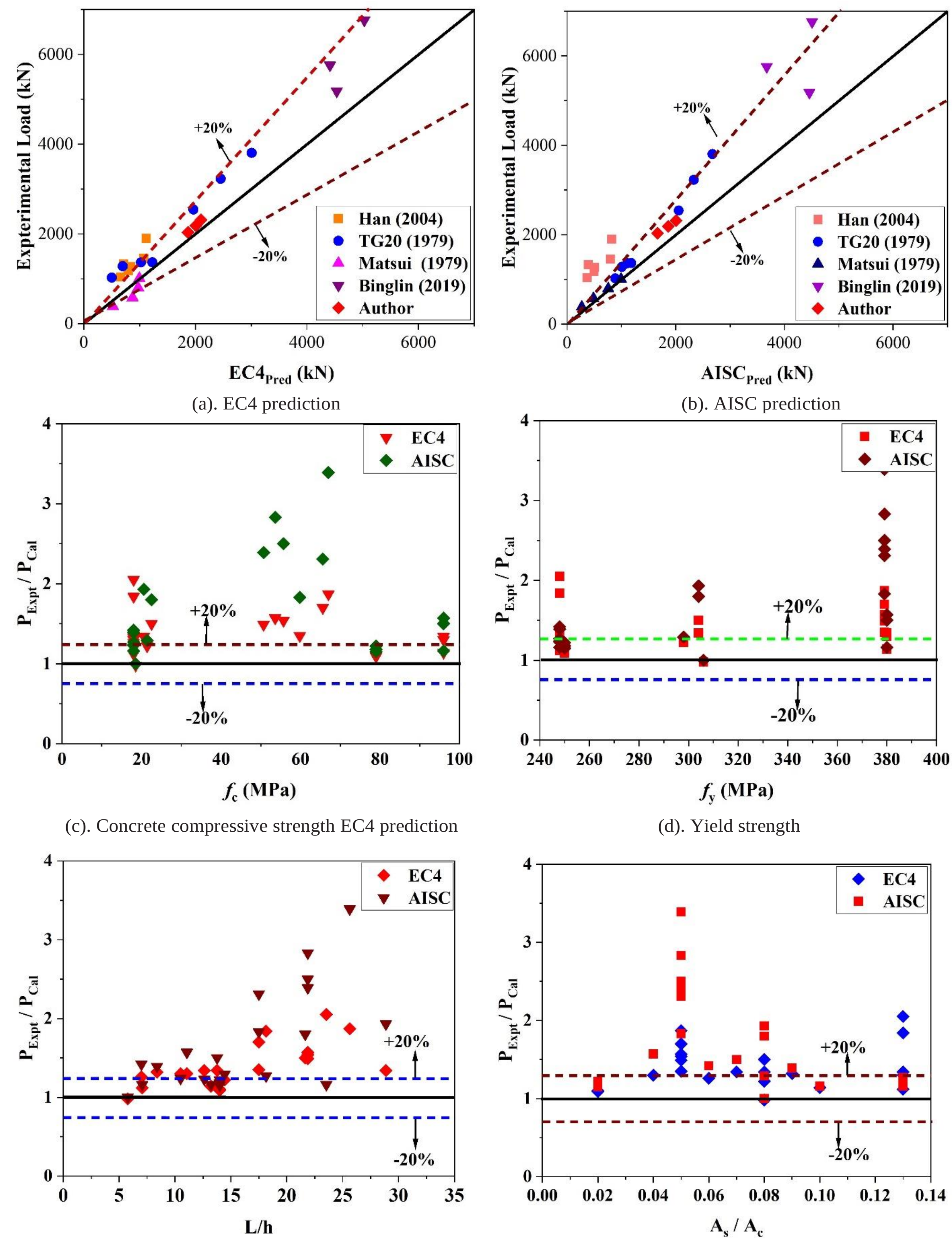
Table 5. Evaluation of the experimental and analytical results from the literature survey.

ID	f_c (MPa)	f_y (MPa)	L/h	A_s/A_c	P_{Expt} (kN)	P_{EC4} (kN)	P_{Expt} /P_{EC4}	P_{AISC} (kN)	P_{Expt} /P_{AISC}	Ref.
SRHC-1	65.6	379	17.50	0.05	1900	1119.46	1.70	824.12	2.31	Han (2004)
SRHC-2	59.8	379	17.50	0.05	1457	1075.70	1.35	797.25	1.83	
SRHC-3	55.7	379	21.88	0.05	1270	823.37	1.54	508.85	2.50	
SRHC-4	50.7	379	21.88	0.05	1183	796.12	1.49	495.09	2.39	
SRHC-5	53.6	379	21.88	0.05	1040	660.80	1.57	367.06	2.83	
SRHC-6	67	379	25.63	0.05	1330	709.97	1.87	391.94	3.39	
1	18	248	7.07	0.13	1370	1228	1.12	1182	1.16	TG20 (1979)
2	18	248	12.62	0.13	1366	1023	1.34	1110	1.23	
3	18	248	18.15	0.13	1281	697	1.84	1008	1.27	
4	18	248	23.54	0.13	1027	500	2.05	886	1.16	
5	18	248	10.50	0.13	2544	1964	1.30	2054	1.24	
6	18	248	8.40	0.09	3229	2453	1.32	2331	1.39	
7	18	248	7.00	0.06	3807	3010	1.26	2674	1.42	
A-1	18.5	306	5.78	0.08	996	1013.99	0.98	993.80	1.00	Matsui (1979)
B-2	21.4	298	14.43	0.08	974	800.77	1.22	757.87	1.29	
C-3	22.5	304	21.65	0.08	874	581.13	1.50	486.12	1.80	
D-4	20.6	304	28.87	0.08	522	388.54	1.34	270.34	1.93	
CES-1	96	380	13.22	0.10	5180	4534.06	1.14	4458.80	1.16	Binglin et al. (2019)
CES-2	96	380	13.77	0.07	6760	5028.24	1.34	4509.40	1.50	
CES-3	96	380	11.07	0.04	5758	4413.15	1.30	3674.70	1.57	
FECC-1	79	250	14	0.02	2034	1870	1.09	1663	1.22	Author
FECC-2	79	250	14	0.02	2187	2010	1.09	1860	1.18	
FECC-3	79	250	14	0.02	2315	2099	1.10	2006	1.15	
Mean							1.39		1.65	
CV (%)							0.28		0.65	

Table 6. Evaluation of prediction of flexural stiffness to literature survey

ID	El_{Expt} /El_{EC4}	El_{Expt} /El_{AISC}	El_{Expt} /El_{ACI}	Ref.
SRHC-1	0.91	0.97	0.86	Han (2004)
SRHC-2	0.89	0.97	0.89	
SRHC-3	0.90	0.97	0.95	
SRHC-4	0.94	0.97	0.87	
SRHC-5	0.92	0.97	0.92	

SRHC-6	0.95	0.97	0.92	
SRC-E1	0.95	0.95	0.93	Wang (2009)
SRC-E2	0.91	0.93	0.90	
SRC-E3	0.85	0.98	0.91	
SRC-E4	0.90	0.93	0.90	
SRC-E5	0.93	0.99	0.84	
SRC-E6	0.82	0.95	0.93	
SRC-E7	0.91	0.95	0.93	
SRC-E8	0.98	0.95	0.82	
SRC-1	0.91	0.97	0.94	Yu & Lu (2009)
SRC-2	0.95	0.96	0.95	
SRC-3	0.88	0.96	0.94	
SRC-4	0.92	0.98	0.97	
SRC-5	0.91	0.97	0.96	
SRC-6	0.92	0.97	0.94	
SRC-7	1.01	0.95	0.93	
SRC-8	0.94	0.97	0.94	
C1	0.98	0.95	1.25	Kim et al. (2012)
C2	0.97	0.85	1.16	
C3	0.99	0.99	1.21	
C4	0.99	1.01	0.99	
C10	0.97	0.80	1.02	Kim et al. (2013)
C11	0.97	0.78	1.04	
C12	0.99	0.79	1.08	
CES-1	0.98	1.31	2.04	Binglin et al. (2019)
CES-2	0.99	2.42	3.63	
CES-3	0.98	2.80	3.71	
FECC-1	1.67	0.83	4.88	Self-reference
FECC-2	1.64	0.82	4.96	
FECC-3	1.67	0.83	5.01	
Mean	0.97	1.04	1.49	
CV (%)	0.27	0.41	1.26	



(a). EC4 prediction

(b). AISC prediction

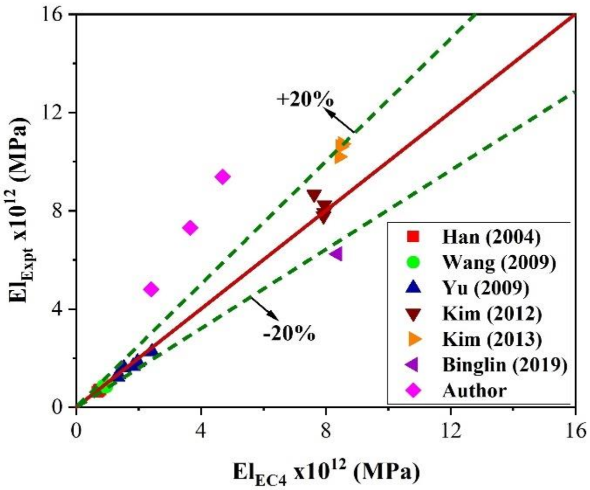
(c). Concrete compressive strength EC4 prediction

(d). Yield strength

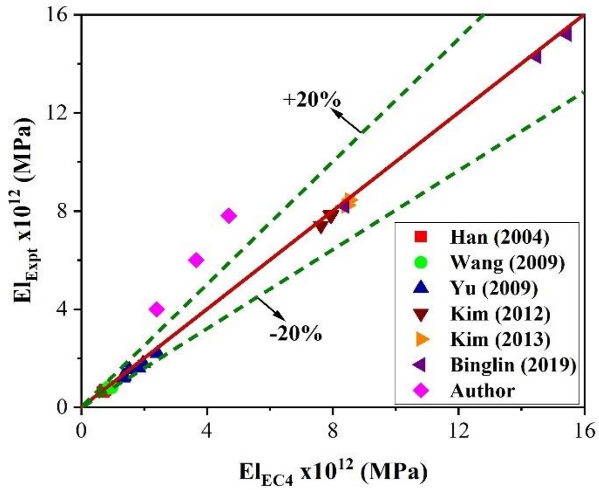
(e). Slenderness ratio

(f). Steel ratio

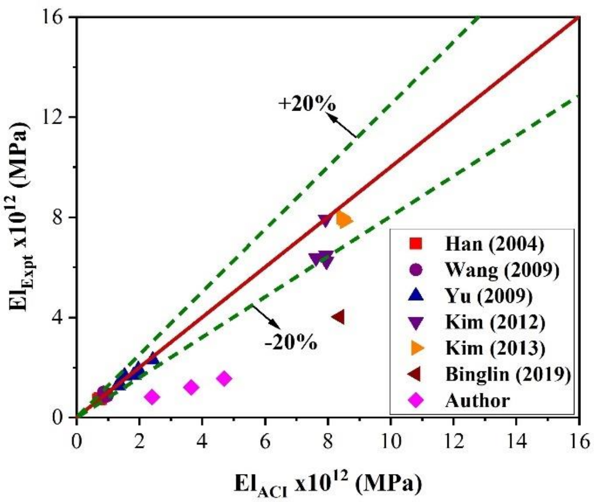
Figure 7. Comparison between analytical and experimental results from literature survey.



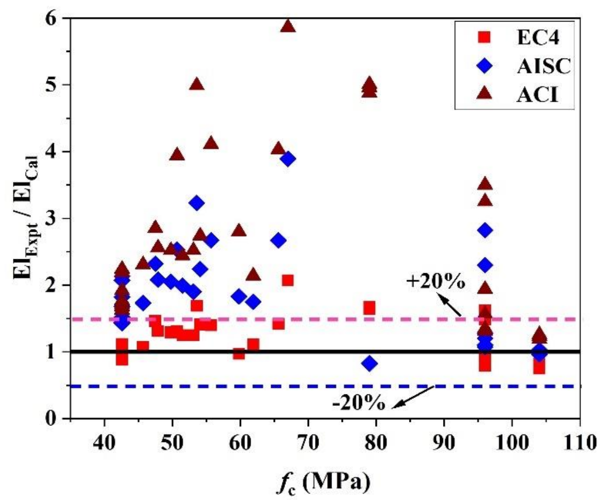
(a). EC4 prediction



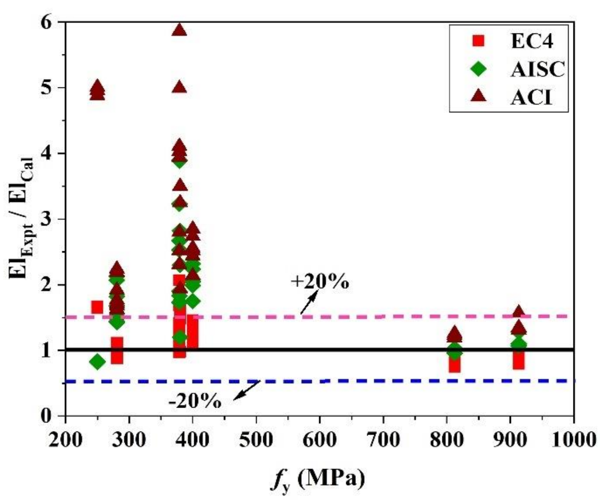
(b). AISC prediction



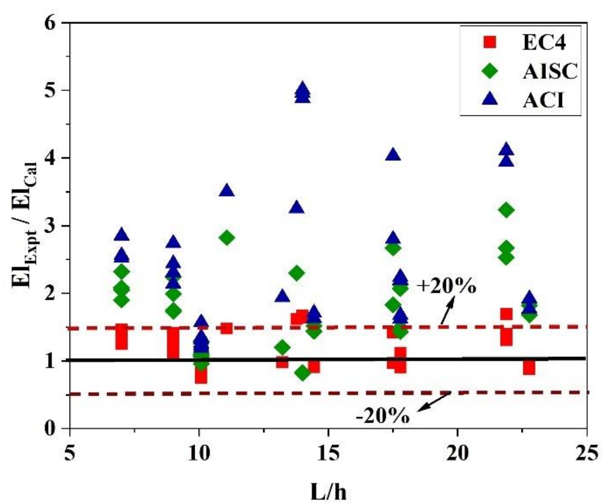
(c). ACI prediction



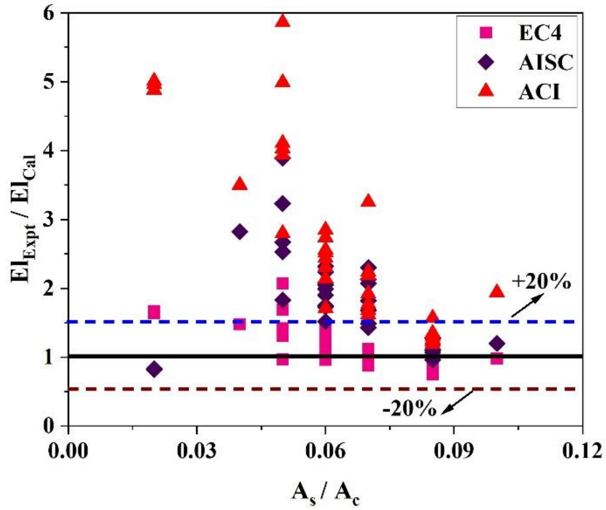
(d). Concrete compressive strength



(e). Yield strength



(f). Slenderness ratio



(g). Steel ratio

Figure 8. Comparison between analytical and experimental results of flexural stiffness.

6. Conclusions and comments

The present study investigated the buckling behaviour of fully encased composite columns in experimental, analytical and numerical analysis. Based on the experimental, analytical and numerical analysis results, the following conclusions were drawn:

1. The experimental study investigated various cross-sections of fully encased composite columns under axial load. The FECC-1, FECC-2 and FECC-3 fail similarly, but load carrying capacity is increased in FECC-3 compared to the remaining two column specimens.
2. Transverse reinforcement leads to increasing column specimens' axial load-carrying capacity. Initially, the columns failed with linear behaviour; furthermore, the load is applied to the columns going to elastic to plastic region, as the columns failed after peak ultimate load.
3. EC4 and AISC codes helped predict the buckling resistance of FECC specimens; EC4 prediction results in better correlations than the AISC code. The experimental results were compared to the previous literature by using statistical analysis. The statistical analysis results helped to predict the current test results.
4. The buckling resistance of FECC specimens was compared to the numerical analyses. The experimental test results were highly correlated to the numerical analysis.
5. The flexural stiffness was evaluated using various codes, and the flexural stiffness of current test results was compared to the previous literature test results. Based on the statistical prediction analysis, the EC4 performs better than the remaining codes.

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