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Finite Element Analysis of Fully Encased Composite Columns

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Abstract

Composite structures are a combination of structural steel shapes and reinforced concrete. These two materials are combined in such a way that both of them are strengthened. The advantages of composite construction include speed, light weight and improved strength of steel, as well as the higher mass, stiffness, damping properties and economy of reinforced concrete. The most common composite columns are concrete filled steel tubes and partially or fully encased steel profiles. Fully encased composite column (FEC) provides compressive strength, stability, stiffness, improved fire proofing and better corrosion protection. This paper investigates the behaviour and strength of axially loaded concrete encased steel composite columns. A nonlinear 3-D finite element (FE) model has been developed to analyse the inelastic behaviour of steel, concrete, and longitudinal reinforcement, as well as the effect of concrete confinement on FEC columns. The FE models have been validated against the current experimental study conduct in the laboratory and published experimental results under concentric and eccentric load. It has been found that the FE model is capable of predicting the nonlinear behaviour of the FEC columns including its failure. The capacities of each constituent of FEC columns, such as steel-I section, concrete and rebars were also determined from the numerical study. Concrete is observed to provide around 54% of the total axial capacity of the column whereas the steel I-sections contribute to the rest of the capacity as well as ductility of the overall system. The nonlinear FE model developed in this study is also used to explore the effect of concrete strength on the behaviour of FEC columns under concentric loads. The axial capacity of FEC columns has been found to increase significantly by increasing the strength of concrete.

Keywords: *Composite Columns, Experimental Finite Element, Fully Encased Numerical Strength*

INTRODUCTION

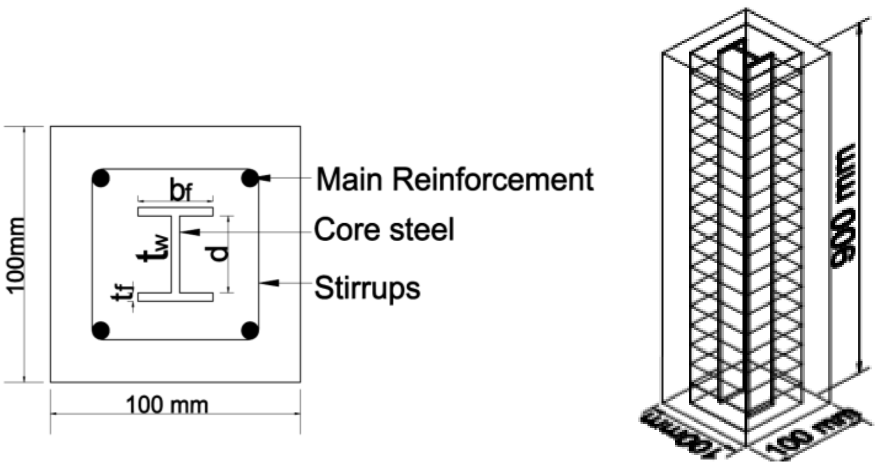
Composite construction system first appeared in the United States in 1894 but its design guidelines were established in 1930. During the past few decades, steel concrete composite structural systems have been used in many tall buildings all over the world. Extensive experimental investigations on FEC columns have been conducted by Sherif and Deierlein [1], Virdi and Dowling [2] with the extensive review of most of these researches given by Shanmugam and Lakshmi [3]. These tests were carried out on concentrically loaded and eccentrically loaded FEC columns having different slenderness ratios, different steel sections and different concrete and steel strength. Analytical and theoretical studies on concentrically loaded and eccentrically loaded FEC columns have been performed by Tokgoz and Dundar [4], and Chen and Lin [5]. It is observed that very limited number of studies are available on numerical simulation of FEC columns. Recently, Ellobody and Young [6] developed a nonlinear 3-D finite element model to investigate the behaviour of concentrically and eccentrically loaded FEC columns. FE model is able to isolate the contribution of the individual element of FEC columns. But, it is observed that studies on determining the capacity of individual elements of FEC columns are very few. Attempts have been made in this study to develop a full scale 3D FE model for FEC columns to explore the behaviour and strength of FEC columns encompassing a wide variety of geometry and material properties. The model will be verified against the experiments conducted in the laboratory and experimental studies carried out by other authors. The validated FE model will eventually be used to explore the failure behaviour of FEC columns.

PROPERTIES OF TEST SPECIMENS

The test program consisted of six (06) numbers of FEC columns with varying percentages of core steel. These composite column specimens had square cross section (100mmx100mm, as shown in Table 1) and constructed with normal strength concrete. The concrete compressive strength (f'c) for normal strength concrete was 27 MPa. The yield strength of reinforcement and core steel were 415 MPa and 350 MPa respectively. The length (L) of all FEC test columns was 900 mm. The typical cross section of these columns is illustrated in Figure 1.

Table 1: Geometric Properties of Test Specimens With Normal Strength Concrete

Sl. No.	Specimen Designation	Steel Plate Size $b_f x d x t_f x t_w$ (mm)	Reinforcement		Steel Ratio	
			Longitudinal Rebar	Tie Rebar (mm)	Plate (%As)	Rebar (%Asr)
1	SCN4A-1	20x20x5x5	4-φ8mm	φ6mm@50mm	3	2
2	SCN4A-2	20x20x5x5	4-φ8mm	φ6mm@50mm	3	2
3	SCN4A-3	20x20x5x5	4-φ8mm	φ6mm@50mm	3	2
4	SCN4B-1	25x25x5x5	4-φ8mm	φ6mm@50mm	3.75	2
5	SCN4B-2	25x25x5x5	4-φ8mm	φ6mm@50mm	3.75	2
6	SCN4B-3	25x25x5x5	4-φ8mm	φ6mm@50mm	3.75	2



(a) (b)

Figure.1. Typical Sections FEC Column (a) Cross Section (b) Elevation

FINITE ELEMENT MODEL

A complete 3D finite element model is developed in this study to investigate the behaviour and strength of FEC columns encompassing a variety of geometry and material properties. Both material and geometric nonlinearities are incorporated in the FE model. ABAQUS finite element code is used to develop the nonlinear FE model for FEC columns in this study [7].

The developed FE model is validated against FEC column tests performed in laboratory and experimental studies carried out by other researchers. Failure mode of FEC columns, axial load carrying capacity and axial shorting of individual elements obtained from the nonlinear FE analysis is compared with that obtained from the experiments.

Geometric Properties of FE Model

The FEC columns investigated in this study comprises of four components, core steel section, longitudinal reinforcing bars, transverse reinforcing bars and concrete. The steel section in FEC column is modelled with S4R shell element. Each node of the S4R shell element has six degrees of freedom- three translations and three rotations. This element uses one integration point on its mid-surface to form the element internal force vector. The longitudinal and transverse bars are modelled using T3D2 three dimensional truss elements. The concrete of FEC column is simulated using solid C3D8R element. It is a continuum three dimensional eight node reduced integration brick element with three translational degrees of freedom at each node. To ensure bonding between the concrete and the reinforcing bars, the rebars are defined as "embedded" reinforcement in the concrete blocks, which effectively couples the longitudinal behaviour of the rebar with that of the adjacent concrete.

Material Properties for FE Model

Steel and concrete are the main materials used in FE model for numerical investigation. Plastic properties and reinforcement of steel were determined for numerical analysis.

The material properties for concrete and steel used in the FE model are listed in Table 2. The subscripts y, sh and u in the table 2 and 3 signify the yield, onset of strain hardening and ultimate strain respectively. The stress strain data obtained from uniaxial tension test were converted to true stress and logarithmic plastic strain based on modulus of elasticity and engineering stress and strain. It was calculated based on coupon test of steel. The value of the Poisson's ratio for steel used in the numerical analysis is 0.3.

The damage plasticity model in ABAQUS was used to simulate the concrete material behaviour in the composite columns. Carreira and Chu [8,9] equations were used to generate the compression and tension stress-strain curve for concrete material in FEC columns.

The unlimited strength for concrete was obtained from standard cylinder tests performed at the test day on concrete for each test specimen.

Table 2 Material Properties of Reference Test Specimens

Specimen Designation	Properties of Test Concrete				Properties of Steel Plate					
	f_{cu} (MPa)	E_c (MPa)	ε_c (με)	γ	F_y (MPa)	F_{sh} (MPa)	F_u (MPa)	ε_y (mm/mm)	ε_{sh} (mm/mm)	ε_u (mm/mm)
SCN4A	27	24683	1900	0.18	350	355	626	0.003860	0.022320	0.129830
SCN4B	27	24683	1900	0.18	350	355	626	0.003860	0.022320	0.129830

Table 3 Material Properties of Reference Test Specimens

Specimen	Properties of Test Concrete				Properties of Reinforcement					
	f_{cu} (MPa)	E_c (MPa)	ε_c (με)	γ	F_y (MPa)	F_{sh} (MPa)	F_u (MPa)	ε_y (mm/mm)	ε_{sh} (mm/mm)	ε_u (mm/mm)
SCN4A	27	24683	1900	0.18	470	471	634	0.003220	0.019170	0.13550
SCN4B	27	24683	1900	0.18	470	471	634	0.003220	0.019170	0.13550

Loading Condition and Solution Strategy

Load is applied using displacement control technique on the top surface of the column. The base of the column is fixed in all directions. Riks solution strategy has been implemented to trace the stable post peak behaviour of the composite column up to failure. This method is generally used to predict the unstable geometrically nonlinear collapse of a structure.

VERIFICATION OF FE MODEL

Numerical simulation was conducted using the developed 3D model for FEC column on the current test specimens as well as on test specimens of other researchers. The specimens varied in their size, steel ratio and material properties. All the current specimens were tested experimentally under concentric axial load. But the reference specimens were test experimentally under eccentric load. Both types of specimens are verified numerically. Comparisons have been made between the experimental and numerical load-deflection behaviour and ultimate capacities of specimens mentioned in this study.

Numerical Investigation of Current Test Specimens

Current experimental test investigation consists of six FEC columns that are 100 mm x 100 mm in size. This experimental study has been carried out before and already reported in Rahman et.al [10]. Numerical simulation has been carried out with these specimens to determine their strength and behaviour. The experimental ultimate capacity and behaviour of these FEC columns are compared with the numerical models. The main variables considered in the test program were concrete compressive strength, cross sectional dimensions and percentage of structural steel. The geometric properties for experimental test are given in table 1. The failure mode and experimental load-deflection behaviour of the specimens were examined in the tests. Theses six columns have been divided in two groups SCN4A (SCN4A-1, SCN4A-2, SCN4A-3) and SCN4B (SCN4B-1, SCN4B-2, SCN4B-3). The values of mean experimental and numerical peak loads for six columns with normal strength concrete are shown in Table 4

The mean value of experimental-to-numerical peak load ratio, Pexp/Pnum and standard deviation is 1.05 and 0.009 respectively. This indicates the excellent performance of the finite element model in predicting the ultimate capacity of FEC columns with two different strengths of concrete for concentrically loaded conditions. As shown in Table 4, the ratio of the experimental-to-numerical average axial strain at peak load, εexp/εnum ranged from 1.026 to 1.26 and the mean value of 1.16 with a standard deviation of 0.14.

The strain ratios for specimens SCN4B group is slightly greater than 1.0, indicating an accurate representation of strength up to the peak load. It is found that the experimental result is about 6% more than the numerical for both the cases. The load-deformation behaviour has been

Table 4 Comparison of Numerical and Experimental Results for Normal Strength Concrete

Ser No	**Specimen Designation**	**Pick axial load**		**P_{exp}/ P_{num}**	**Avg. axial strain at peak load**		**ε_{exp}/ ε_{num}**
		P_{num} (KN)	**P_{exp} mean (KN)**		**Num. ε_{num} (με)**	**Exp. ε_{exp} ((με)**	
1	SCN4A	471	491	1.042	2550	2708	1.062
2	SCN4B	490	516	1.053	2541	3202	1.260
Mean				1.050			1.161
SD				0.009			0.140

determined for these two groups of FEC columns. It is observed that the load-deformation behaviour of these two FEC columns is quite similar. So, the load-deformation behaviour of columns' groups SCN4B is shown in Figure 2. The ascending branch of experimental and numerical load-deformation curves are very close but little different at descending branch.

same sizes (160x160mm) of FEC column for eccentric load. The length of these columns is 900 mm and concrete strength 21.1MPa. Chen & Yea [12] carried out study on same dimensions (280x280mm) of FEC columns for concentric load with variation of stirrups speacing.

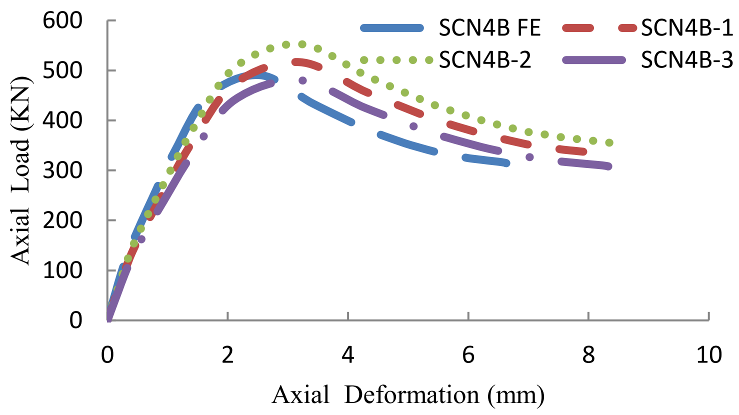


Figure 2. Comparison Between Experimental and Numerical Load

Numerical Investigation of Reference Test Specimens

Extensive numerical study has been reported in this paper on experimental test done by different researchers for FEC columns.

There are eleven columns (11) of different researchers are numerically simulated to determine the ultimate capacity. Morino et al. [11] carried out experimental study on

Similarly, Dundar et al. [4] and Ellobody & Young [6] carried out experimental studies for different strengths of concrete for eccentric load. Geometric properties of reference specimens are given in Table 5.

The matter properties of these reference columns are also given in Tables 6 and 7.

Table 5 Geometric Properties of Reference Specimen

References	Specimen Designation	Size B (mm)	Size D (mm)	Length of Column (mm)	Steel Plate Size b_f x d x t_w xt_f (mm)	Reinforcement Longitudinal	Reinforcement Tie Spacing (mm)
Morino et al.(1984)	A4-00	160	160	960	H100x100x6x8	4-φ 6mm	φ-4mm @150
	A4-45	160	160	960	H100x100x6x8	4-φ 6mm	φ-4mm @150
	A4-90	160	160	960	H100x100x6x8	4-φ 6mm	φ-4mm @150
Chen & Yeh (2005)	SRC1	280	280	1200	H150x150x7x10	12-φ16mm	φ-8mm @140
	SRC2	280	280	1200	H150x150x7x10	12-φ16mm	φ-8mm @ 75
	SRC3	280	280	1200	H150x150x7x10	12-φ16mm	φ-8mm @ 35
Dundar et al.(2008)	CC3	150	150	1300	T 50x50x5	4-φ 8mm	φ-6mm @100
	CC4	150	150	1300	T 50x50x5	4-φ 8mm	φ-6mm @100
Ellobody & Young (2010)	1	160	160	924	H100x100x6x8	4-φ 6mm	φ-4mm @75
	2	160	160	2309	H100x100x6x8	4-φ 6mm	φ-4mm @75
	3	160	160	3464	H100x100x6x8	4-φ 6mm	φ-4mm @75

Table 6 Material Properties of Reference Specimen

Specimen Designation	Properties of concrete in test region f_{cu} (MPa)	E_c (MPa)	ε_{cu} (με)	γ	Properties of Steel Plate F_y (MPa)	F_{sh} (MPa)	F_u (MPa)	ε_y (%)	ε_{sh} (%)	ε_u (%)
A4-00	21.1	22150	1728	0.18	344.8	344.8	431	0.19	1.95	17
A4-45	21.1	22150	1728	0.18	344.8	344.8	431	0.19	1.95	17
A4-90	21.1	22150	1728	0.18	344.8	344.8	431	0.19	1.95	17
SRC1	29.5	24932	1896	0.18	296	296	373	0.17	1.67	14
SRC2	28.1	24499	1868	0.18	296	296	373	0.17	1.67	14
SRC3	29.8	25024	1902	0.18	296	296	373	0.17	1.67	14
CC3	22.69	22714	1760	0.18	235	235	294	0.16	1.76	15
CC4	45.4	29270	2214	0.18	235	235	294	0.16	1.76	15
1	23	22828	1766	0.18	306	306	386	0.18	1.87	15
2	26	23829	1826	0.18	298	298	378	0.18	1.87	15
3	28	24468	1866	0.18	304	304	383	0.18	1.87	15

Axial load carrying capacity of these reference FEC columns had been determined experimentally in different conditions. These FEC columns are numerically simulated with determined load capacity for validation of FE models. Comparative study has been carried out between the experimental and numerical results of the aforesaid FEC columns as shown in Table 8. The mean value of experimental-to-numerical peak load ratio, P_{exp}/P_{num} and standard deviation are determined for all these specimen groups.

The average mean values and standard deviation of these four group specimens are 1.028 and 0.025 respectively. This indicates the excellent performance of the finite element model in predicting the ultimate capacity of these FEC columns.

Table 7 Material Properties of Test Specimen

Specimen Designation	Properties of Concrete in Test Region				Properties of Rebar's					
	f_{cu} (MPa)	E_c (MPa)	ε_{cu} (με)	γ	F_y (MPa)	F_{sh} (MPa)	F_u (MPa)	ε_y (%)	ε_{sh} (%)	ε_u (%)
A4-00	21.1	22150	1728	0.18	380	380	495	0.26	2.24	16
A4-45	21.1	22150	1728	0.18	380	380	495	0.26	2.24	16
A4-90	21.1	22150	1728	0.18	380	380	495	0.26	2.24	16
SRC1	29.5	24932	1896	0.18	350	350	441	0.24	1.95	15
SRC2	28.1	24499	1868	0.18	350	350	441	0.24	1.95	15
SRC3	29.8	25024	1902	0.18	350	350	441	0.24	1.95	15
CC3	22.69	22714	1760	0.18	500	500	625	0.2	1.97	18
CC4	45.4	29270	2214	0.18	500	500	625	0.2	1.97	18
1	23	22828	1766	0.18	376	376	489	0.25	2.21	14
2	26	23829	1826	0.18	376	376	489	0.25	2.21	14
3	28	24468	1866	0.18	376	376	489	0.25	2.21	14

Table 8 Comparison of Experimental and Numerical Results

References	Specimens Designation	Location of Load		Load Capacity (KN)		P_{num}/P_{expt}
		e_x (mm)	e_y (mm)	Experimental (P_{expt})	Numerical (P_{num})	
Morino et al.(1984)	A4-00	40	0	499.91	526	1.05
	A4-45	28.28	28.28	518.83	532	1.03
	A4-90	0	40	740.44	702	0.948
	Mean					**1.01**
	SD					**0.054**
Chen & Yeh (2005)	SRC 1	0	0	4220	4013	0.951
	SRC 2	0	0	4228	4033	0.954
	SRC 3	0	0	4399	4225	0.961
	Mean					**0.955**
	SD					**0.005**
Dunder et.al (2008)	CC 3	50	48.5	176	197	1.119
	CC 4	40	39.2	319	345	1.081
	Mean					**1.1**
	SD					**0.026**
Ellobody & Young (2010)	1	2.07	2.07	996	1068	1.07
	2	3.28	3.28	974	1026	1.05
	3	4.24	4.24	874	910	1.04
	Mean					**1.05**
	SD					**0.013**

COMPARISON OF FAILURE MODES

The failure modes for FEC columns were identified from the finite element analysis and compared with the failure modes in the test. The failure modes are concrete crushing (CC), structural steel yielding (SY), and flexural buckling of rebars (F).

The three modes can be easily identified by examining the stress of the concrete and structural steel elements against the measured material strength. The first two modes, CC and CY, occur simultaneously, with yielding of the steel flange occurring first followed by concrete crushing. The experimental failure mode of a FEC column is shown in Figure 3. The principle stress in concrete and steel of FEC column along 3-3 axis is shown in Figure 4.

Figure 3 Deformed shape and experimental failure

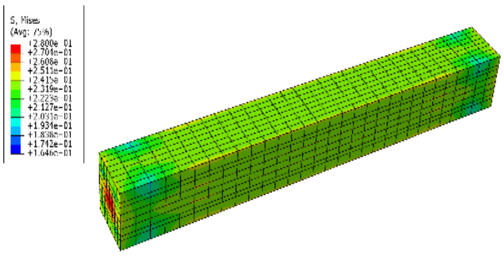


Figure 4 Deformed Shape and Numerical Failure

Load Carrying Capacity of Individual Elements in FEC Column

The finite element model is able to isolate the contribution of the core steel, reinforcement and concrete in the total load carrying capacity of the FEC columns. The axial load versus strain behavior of the individual materials in the composite columns is determined numerically for these two types of specimens with varying percentage of core steel.

It has been observed that the contribution of reinforcement, steel I-section and concrete in the total load carrying capacity of the FEC columns are 15%, 31% and 54% respectively. Axial load versus strain of individual elements of FEC column SCN4B is shown Figure 5. Moreover, increasing the core steel ratio from 1% to 2%, the total axial load capacity was increased by 5% between column SCN4A and SCN4B.

EFFECT OF CONCRETE STRENGTH ON CAPACITY OF FEC COLUMN

Axial load versus strain behaviour of the FEC columns have been determined numerically for concrete strength 41 MPa (6 ksi). It is observed that axial load carrying capacity of column SCN4B is increased by 28% when concrete strength is increased from 27 MPa (4 ksi) to 41

MPa (6 ksi). It is also found that higher concrete strength enhances the overall capacity of FEC columns significantly, as shown in Figure 6.

behaviour of FEC columns under concentric and eccentric loads with good accuracy.

The failure of the FEC column under

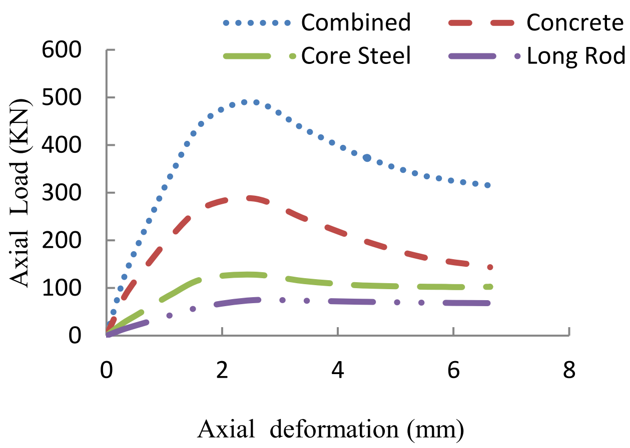
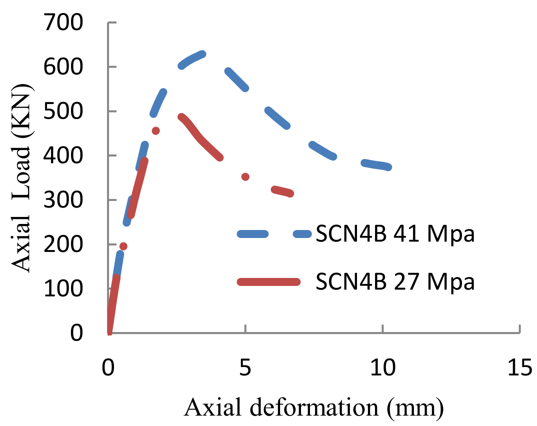


Figure 5 Axial Load-Deformation of Columns



CONCLUSIONS

Extensive numerical simulation has been carried out on ultimate load carrying capacity and the behaviour of square FEC columns subjected to short term axial and eccentric load. These studies have been done for different strengths of concrete, length of columns and varying steel ratio. A nonlinear 3-D FE model has been developed to analyse the behaviour of FEC columns. Nonlinear material behaviour for concrete has been incorporated in FE analysis. Geometric nonlinearities were also included in the model. It is found that the finite element model is able to predict the experimental

gravity loads occurred through crushing of concrete followed by yielding of steel. The FE model was able to simulate the experimental failure bahviour accurately. It observed from numerical study that concrete and core steel carries about 54% and 31% axial load of the total capacity of FEC columns. Concrete plays an important role on overall capacity of a FEC column. Load carrying capacity of FEC column increased by 28% when concrete strength is increased to 41 MPa (6ksi) from 27 MPa (4ksi).

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