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Comparison between Experimental and Numerical Studies of Fully Encased Composite Columns

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Abstract—Composite column is a structural member that uses a combination of structural steel shapes, pipes or tubes with or without reinforcing steel bars and reinforced concrete to provide adequate load carrying capacity to sustain either axial compressive loads alone or a combination of axial loads and bending moments. Composite construction takes the advantages of the speed of construction, light weight and strength of steel, and the higher mass, stiffness, damping properties and economy of reinforced concrete. The most usual types of composite columns are the concrete filled steel tubes and the 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 reports experimental and numerical investigations of the behaviour of concrete encased steel composite columns subjected to short-term axial load. In this study, eleven short FEC columns with square shaped cross section were constructed and tested to examine the load-deflection behavior. The main variables in the test were considered as concrete compressive strength, cross sectional size and percentage of structural steel. 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 of the FEC columns. FE models have been validated against the current experimental study conduct in the laboratory and published experimental results under concentric load. It has been observed that FE model is able to predict the experimental behaviour of FEC columns under concentric gravity loads with good accuracy. Good agreement has been achieved between the complete experimental and the numerical load-deflection behaviour in this study. The capacities of each constituent of FEC columns such as structural steel, concrete and rebar's were also determined from the numerical study. Concrete is observed to provide around 57% of the total axial capacity of the column whereas the steel I-sections contributes 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 and percentage of structural steel 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, strength.

I. Introduction

COMPOSITE construction system first appeared in the United States in 1894 but the 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. There are three types of composite columns commonly used in high rise building. Typical cross-section of these columns are shown in Fig. 1. FEC columns the structural steel section is fully encased by surrounding concrete shown in Fig. 1 (a), whereas in partially encased composite columns, the steel section is partially encased by concrete as shown in Fig. 1 (b). On the other hand, in concrete filled tubular composite columns, as shown in Fig. 1 (c), the concrete is fully confined by the surrounding steel section. Extensive experimental investigations on FEC columns have been conducted by [1]-[4]. Shanmugam and Lakshmi [5] carried out extensive review on previous researches of composite columns. 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 [6] and [7]. However, very few studies have been found in the in the published literature on full scale 3D simulation of FEC columns. Recently, [8] developed a nonlinear 3-D FE model investigating the behaviour of axially concentric loaded FEC columns. Attempts have been made in this study to develop a full scale 3D FE model for FEC columns to explore the behavior 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 as well as with the experiments conducted by [3]. The validated FE model will eventually be used to explore the failure behavior and contribution of the individual elements such as concrete and steel of FEC columns.

(a) (b) (c)

Fig. 1 Typical X-sections of composite columns, (a) FEC (b) PEC (c) CFT

II. Experimental Investigations

An experimental investigation to determine the failure behaviour and load carrying capacity of FEC columns is presented in this study. The main variables considered in the test program were the concrete compressive strength, cross sectional dimensions and percentage of structural steel. The

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failure mode and experimental load-deflection behaviour of the specimens were examined in the tests.

A. Description of Test Specimens

The test program consisted of eleven (11) numbers of FEC columns of two different sizes with varying percentages of structural steel. Six composite column specimens with square cross section (100mmx100mm, as shown in Table I) were constructed with normal strength concrete. Another five specimens also with square section (150mmx150mm, as shown in Table II) were constructed with high strength concrete. The concrete compressive strength (f'c) for normal and high strength concrete were 27 and 41 MPa respectively. The yield strength of reinforcement and core steel were 415 MPa. The length (L) of all FEC test columns were 900 mm respectively. The typical cross section of these columns are illustrated in Fig. 2.

B. Column Test

The FEC columns were tested in pure compression by an UTM (Universal testing machine). The ultimate compressive load capacity of this machine is 2000 KN. The columns test set-up is illustrated in Fig. 3, which shows the general characteristics of the testing platens and the instrumentation used in the testing. Axial load was applied to the composite columns specimens at the rate of 5 KN/s. The digital reading of axial load and lateral displacement were collected by using an electronic data acquisition system during testing of each specimen.

Fig. 2 Typical X- section of FEC columns

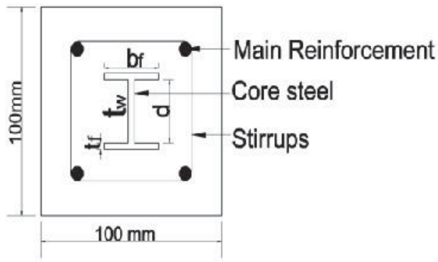

Fig. 3 Experimental column test setup in laboratory

TABLE I
GEOMETRIC PROPERTIES OF TEST SPECIMENS WITH NORMAL STRENGTH CONCRETE

Sl. No.	Specimen	Steel Plate Size	Reinforcement		Steel Ratio	
	Designation	$b_f x d x t_f x t_w$ (mm)	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

TABLE II
GEOMETRIC PROPERTIES OF TEST SPECIMENS WITH HIGH STRENGTH CONCRETE

Sl. No.	Specimen	Steel Plate Size	Steel Rebar		Steel Ratio	
	Designation	$b_f x d x t_f x t_w$ (mm)	Longitudinal rebar	Tie rebar (mm)	Plate (%As)	Rebar (%Asr)
1	SCH6A-1	30x30x5x5	4-φ8mm	φ6mm@75mm	2	1
2	SCH6A-2	30x30x5x5	4-φ8mm	φ6mm@75mm	2	1
3	SCH6A-3	30x30x5x5	4-φ8mm	φ6mm@75mm	2	1
4	SCH6B-1	45x45x5x5	4-φ8mm	φ6mm@75mm	3	1
5	SCH6B-2	45x45x5x5	4-φ8mm	φ6mm@75mm	3	1

III. FE MODEL

A nonlinear 3D FE model was developed in this study to investigate the behavior and strength of FEC columns encompassing a variety of geometry and material properties. Both material and geometric nonlinearities were incorporated in the FE model. ABAQUS FE code [9] was used to develop the nonlinear FE model for FEC columns in this study. The steel section in FEC column is modeled with S4R shell element. Each node of the S4R shell element has six degrees of freedom- three translations and three rotations. The longitudinal and transverse bars were modeled using T3D2 three dimensional truss elements. The concrete of FEC column was simulated using solid C3D8R element. The load was applied using displacement control technique on the top

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surface of the column. The base of the column was fixed in all directions. Rik's solution strategy has been implemented to trace a stable post peak behavior of the composite column up to failure.

IV. Material Properties for FE Model

Steel and concrete are the main materials used in FE model for numerical investigation. Plastic properties for these materials (as shown in Tables III and IV) were incorporated in the FE model. The subscripts $_y$, $_{sh}$ and $_u$ in the table signify the yield, onset of strain hardening and ultimate strain of the steel plates respectively. The stress strain data obtained from uniaxial tension test were converted to true stress and logarithmic plastic strain. It was calculated based on coupon test of steel plates. The value of the Poisson's ratio for steel used in the numerical analysis is 0.3. Elasto-plastic material model is used to simulate the behaviour of steel I section in FEC columns. The damage plasticity model in ABAQUS was used to simulate the concrete material behaviour in the composite columns. Equations of [10], [11] were used to generate the compression and tension stress-strain curve for concrete material in FEC columns. The ultimate strength (f'_{cu}) for concrete was obtained from standard cylinder tests performed on concrete at the test day for each test specimen.

V. Comparison between Experimental and Numerical Results

Numerical simulation has been conducted using the developed 3D model for FEC column on the current test specimens as well as on test specimens of [3]. The specimens varied in their size, steel ratio and material properties. All the specimens were tested under concentric axial load. Comparison between the experimental and numerical load-deflection behaviour and ultimate capacities are present in the following sections.

A. Test Specimens of Current Study

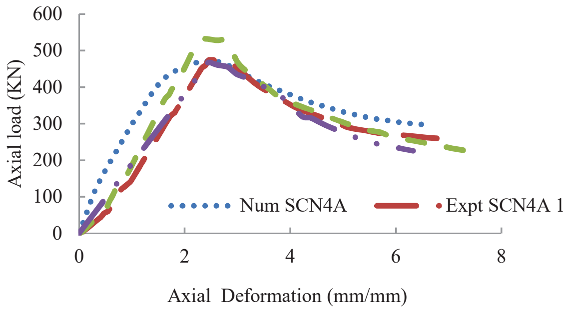
Axial compressive strength, axial shortening and failure behavior were observed and recorded for each FEC column specimens experimentally and numerically. The experimental and numerical load-deflection behaviour of the column group SCN4A, SCN4B, SCN6A and SCN6B are shown in Figs. 4 and 5. It was observed that FE model can predict the experimental behaviour of FEC columns with good accuracy in columns groups SCN4B and SCH6B. The local failure was observed in specimens SCN4A and SCH4B which occurred at the top and bottom of the column under short term axial load. The difference in the initial stiffness between the experimental and numerical load deflection behavior may be attributed due to the localized failure of concrete near the loaded platens of these test specimens. However, the axial capacity and peak-strain of these columns obtained from the numerical analysis matched very well with the corresponding experimental results. The values of mean experimental and numerical peak loads, for six columns with normal strength and five columns with high strength concrete are shown in Tables V and VI respectively. The mean value of experimental-to-numerical peak load ratio, P_{exp}/P_{num} and experimental-to-numerical average axial strain at peak load, $\varepsilon_{exp}/\varepsilon_{num}$, were compared for all groups of columns. It is observed that the mean value and the standard deviation of the ultimate load ratio and corresponding strain ratio of numerical and experimental results for the two groups of test columns are reasonable. This indicates the excellent performance of the FE model in predicting the ultimate capacity of FEC columns with two different strength of concrete.

TABLE III
Material Properties of Test Specimens

Specimen Design.	Properties of 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 SCN4B	27	24680	1900	0.18	350	355	626	0.003860	0.022320	0.129830
SCH6A SCH6B	41	30000	2000	0.18	350	355	626	0.003860	0.022320	0.129830

TABLE IV
Material Properties of Test Specimens

Specimen Designation	Properties of 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 SCN4B	27	24680	1900	0.18	470	471	634	0.00322	0.019170	0.13555
SCH6A SCH6B	41	30000	2000	0.18	470	471	634	0.003220	0.019170	0.13555



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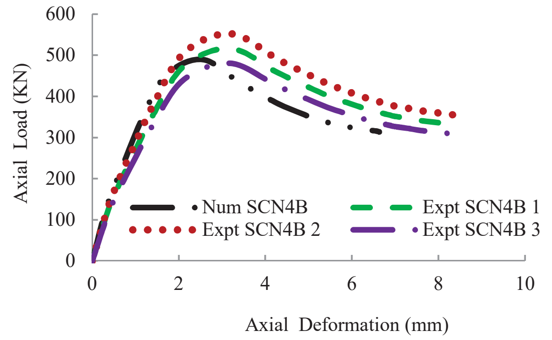
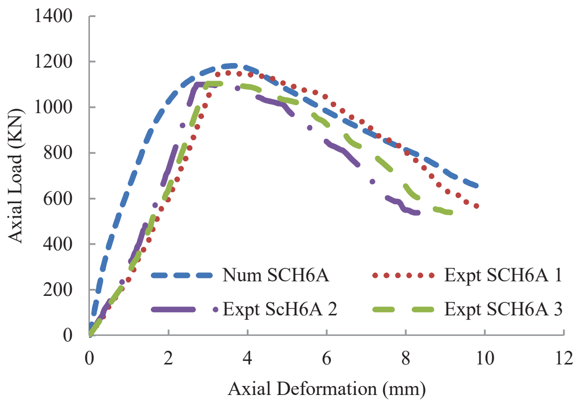


(b)

Fig. 4 Experimental and numerical behaviour of column groups (a) SCN4A (b) SCN4B



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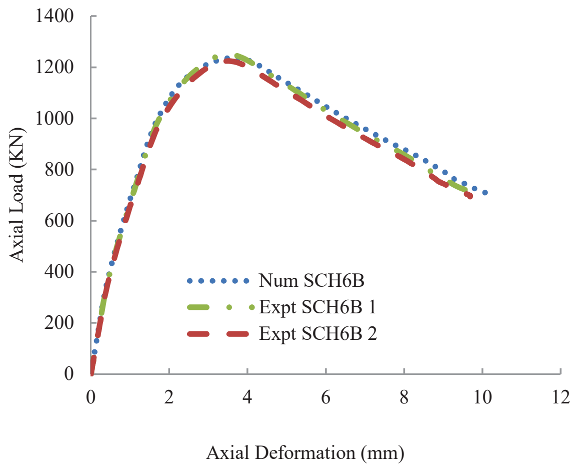


(b)

Fig. 5 Experimental and numerical behaviour of column groups (a) SCN6A (b) SCH6B

TABLE V
COMPARISON OF NUMERICAL AND EXPERIMENTAL RESULTS FOR NORMAL STRENGTH CONCRETE

Ser. No	Specimen	Pick axial load		P_{exp}/P_{num}	Avg. axial strain at peak load		$\varepsilon_{exp}/\varepsilon_{num}$
	Designation	P_{num} (KN)	$P_{exp\ mean}$ (KN)		Num. $\varepsilon_{num\ (\mu\varepsilon)}$	Exp. $\varepsilon_{exp\ ((\mu\varepsilon)}$	
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

TABLE VI
COMPARISON OF NUMERICAL AND EXPERIMENTAL RESULTS FOR HIGH STRENGTH CONCRETE

Ser No	Specimen	Pick axial load		P_{exp}/P_{num}	Avg. axial strain at peak load		$\varepsilon_{exp}/\ \varepsilon_{num}$
	Designation	P_{num} (KN)	$P_{exp(mean)}$ (KN)		N_{um}. ε_{num} ($\mu\varepsilon$)	Exp. ε_{exp} (($\mu\varepsilon$)	
1	SCH6A	1181	1117	0.946	3749	4686	1.250
2	SCH6B	1238	1240	1.002	3748	4314	1.151
Mean				0.974			1.201
SD				0.039			0.070

B. Test Specimens of [3]

Three different shapes of the structural steel section were used in the specimens i.e., H, cross and I shaped sections for the experimental test. The H-shaped steel section is more like the wide-flange section, while the I-shaped section has a narrow flange as illustraed in Fig. 6. Ten (10) specimens used in the verification were labeled from SRC1 to SRC 10. The specimens had square cross-sections of 280x280 mm and a constant niminal length of 1200 mm. The specimens had concrete cylinder strengths varying from 26.4-29.8 MPa and a steel yield stress of 296-345 MPa. The longitudinal and transverse reinforcement bars were 16mm and 8mm in diameter with detailed dimensions shown in Table VII. A nonlinear 3D FE model was developed to investigate the behavior and strength of these test specimens. Similarly, ABAQUS FE code [9] was used to develop the nonlinear FE model for FEC columns in this study. Plastic properties of steel and concrete were incorporated in the FE models as shown in Tables VII and IX.

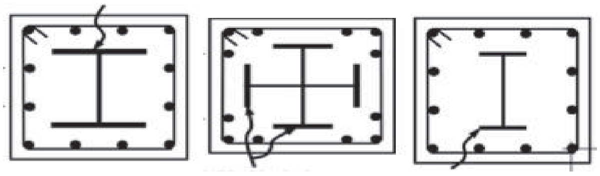


Fig. 6 Cross sections of FEC columns with (a) H, (b) Cross (c) I shaped steel sections

TABLE VII
GEOMETRIC PROPERTIES OF REFERENCE SPECIMENS

Specimen Designation	Size B (mm)	Size D (mm)	Length of Column (mm)	Structural Steel Shape	Structural Steel Size (b_f x d x t_w) (mm)	Tie Spacing (mm)
SRC1	280	280	1200	H	H150x150x7x10	φ-8mm @140
SRC2	280	280	1200	H	H150x150x7x10	φ-8mm @ 75
SRC3	280	280	1200	H	H150x150x7x10	φ-8mm @ 35
SRC4	280	280	1200	Cross	Two H175X90X5X8	φ-8mm @140
SRC5	280	280	1200	Cross	Two H175X90X5X8	φ-8mm @ 75
SRC6	280	280	1200	Cross	Two H175X90X5X8	φ-8mm @ 35
SRC7	280	280	1200	I	H150x75x5x7	φ-8mm @140
SRC8	280	280	1200	I	H150x75x5x7	φ-8mm @ 75
SRC9	280	280	1200	I	H150x75x5x7	φ-8mm @ 140
SRC10	280	280	1200	I	H150x75x5x7	φ-8mm @ 75

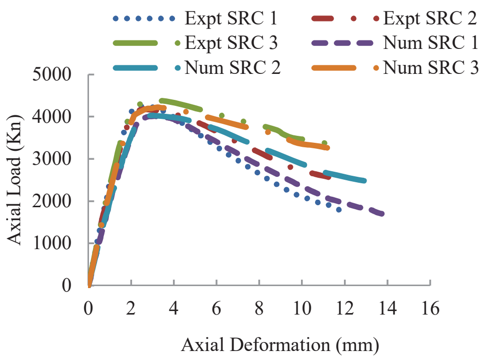
TABLE VIII
PROPERTIES OF CONCRETE AND STEEL PLATE

Specimen Designation	Properties of concrete				Properties of Steel Plate					
	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 (%)
SRC 1	29.5	24932	1896	0.18	296	296	373	0.17	1.67	14
SRC 2	28.1	24499	1868	0.18	296	296	373	0.17	1.67	14
SRC 3	29.8	25023	1902	0.18	296	296	373	0.17	1.67	14
SRC 4	29.8	25023	1902	0.18	345	345	431	0.18	1.87	15
SRC 5	29.8	25023	1902	0.18	345	345	431	0.18	1.87	15
SRC 6	29.5	24932	1896	0.18	345	345	431	0.18	1.87	15
SRC 7	28.1	24499	1868	0.18	303	303	379	0.19	1.95	17
SRC 8	26.4	24997	1834	0.18	303	303	379	0.19	1.95	17
SRC 9	28.1	24449	1868	0.18	303	303	379	0.19	1.95	17
SRC 10	29.8	25023	1902	0.18	303	303	379	0.19	1.95	17

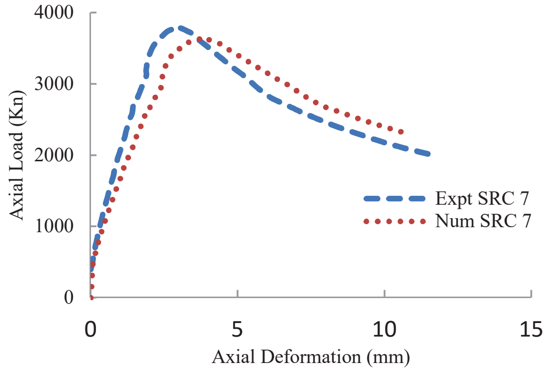
Numerical investigation of these columns were carried out using the developed model and compared with the experimental capacities determined by the author. Table X presents the maximum axial compressive load of the experimental tests and numerical predictions for all the specimens. The numerical models can accurately predict the experimental axial compressive load and peak strain. The average ratio of the numerical loads to experimental capacities, P_{num}/P_{expt} are 0.955,1.044 and 0.942 for the three steel shapes and the corresponding standard deviations are found to be 0 .005, 0.015 and 0.013. This indicates the excellent performance of FE model in predicting the ultimate capacity of these FEC columns with three different shapes of steel and strength concrete for concentrically loaded conditions. As shown in Table X, the ratio of the numerical - to- experimental average axial strain at peak load, $\varepsilon_{num}/\varepsilon_{exp}$ are 1.061, 1.134 and 1.228 and the corresponding standard deviations are 0.089, 0.051 and 0.048. There are three types of confinement areas inside the FEC columns ie; unconfined, partially confined and highly confined. Chen and Lin [7] shown analytically that the confinement factors influenced the overall capacity of columns. It mainly depends on spacing of stirrups, concrete strength and shapes of structural steel. Chen and Lin, reported in that study that confinement factors are comparatively more in H-shapes FEC columns. Similar, behaviour had been observed during numerical studies of these types of columns. The confinement area and concrete confinement factors are comparatively more in cross shaped steel section's FEC columns (SRC4-SRC6) than others. So, it is found from numerical study that the ultimate load carrying capacities are more in these columns than other shapes.

C. Load-Deformation Behaviour

Axial compressive strength, axial shortening and failure behavior has been determined numerically for these ten specimens and compared with the experimental result of [3]. The load-deflection behaviour of columns group (SRC1, SRC2 and SRC3) and column SRC7 are given in Fig. 7. It is found that the FE models are capable of predicting the ultimate capacity and peak strain of these columns with good accuracy.



(a)



(b)

Fig. 7 Experimental and numerical load of columns group (a) H (SRC1-SRC3) (b) SRC 7

TABLE IX
PROPERTIES OF CONCRETE AND REBARS

Specimen Designation	Properties of concrete			Properties of Rebar's					
	f_{cu}	E_c	ε_{cu}	F_y	F_{sh}	F_u	ε_y	ε_{sh}	ε_u
	MPa	MPa	με	(MPa)	(MPa)	(MPa)	(%)	(%)	(%)
SRC 1	29.5	24932	1896	350	350	438	0.24	1.95	15
SRC 2	28.1	24499	1868	350	350	438	0.24	1.95	15
SRC 3	29.8	25023	1902	350	350	438	0.24	1.95	15
SRC 4	29.8	25023	1902	350	350	438	0.25	2.21	14
SRC 5	29.8	25023	1902	350	350	438	0.25	2.21	14
SRC 6	29.5	24932	1896	350	350	438	0.25	2.21	14
SRC 7	28.1	24499	1868	350	350	438	0.26	2.24	16
SRC 8	26.4	24997	1834	350	350	438	0.26	2.24	16
SRC 9	28.1	24449	1868	350	350	438	0.26	2.24	16
SRC 10	29.8	25023	1902	350	350	438	0.26	2.24	16

TABLE X
COMPARISON OF NUMERICAL AND EXPERIMENTAL RESULTS OF REFERENCE SPECIMENS

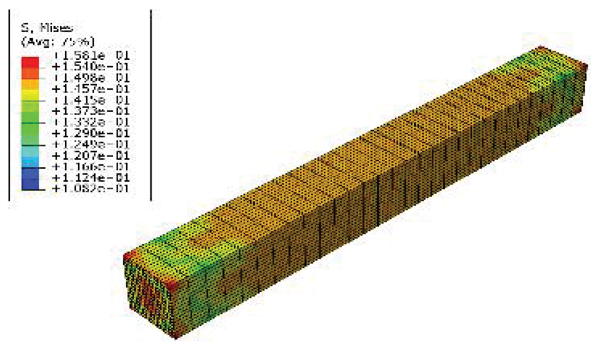
Specimens Designation	Steel Shapes	Load Capacity (KN)		P_{num} /P_{expt}	Axial Strain at Peak Load		$\varepsilon_{num}(\mu\varepsilon)$/ $\varepsilon_{expt}(\mu\varepsilon)$
		Experimental	Numerical		P_{expt}	P_{num}	
		(P_{expt})	(P_{num})		$\varepsilon_{expt}(\mu\varepsilon)$	ε_{num} (με)	
SRC1	H	4220	4013	0.951	2580	2740	1.062
SRC2	H	4228	4033	0.954	2200	2530	1.15
SRC3	H	4399	4225	0.961	2600	2524	0.971
Mean				**0.955**			**1.061**
SD				**0.005**			**0.089**
SRC4	Cross	4441	4642	1.045	2650	3099	1.169
SRC5	Cross	4519	4645	1.028	2850	3300	1.158
SRC6	Cross	4527	4741	1.058	3030	3256	1.075
Mean				**1.044**			**1.134**
SD				**0.015**			**0.051**
SRC7	I	3788	3636	0.959	2450	3145	1.283
SRC8	I	3683	3437	0.933	2550	3030	1.188
SRC9	I	3630	3636	0.944	2650	3145	1.186
SRC10	I	3893	3621	0.931	2500	3138	1.255
Mean				**0.942**			**1.228**
SD				**0.013**			**0.048**

D.Failure Modes

The failure modes for FEC columns were identified from FE analysis and compared with the failure modes observed in the current experiment. Failure modes were captured manually for all the specimens during the test. Failure pattern of column SCN4B is shown in Fig. 8 (b). The local failure was observed in few columns during experimental test. It was prevented using FRP (2mm thick) at the top and bottom of the columns. Experimentally, it is observed that concrete crushing occurred before yielding of the steel plate. Similar failure behavior was obtained in the nonlinear FE simulation of FEC columns under axial loads. The principle stress in concrete and steel of FEC column along 3-3 axis is shown in Fig. 8 (a).

VI. CONTRIBUTIONS OF STEEL AND CONCRETE

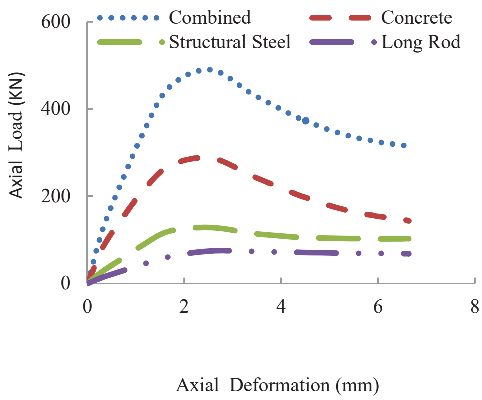
FE model is able to isolate the contribution of the core steel, reinforcement and concrete in the total load carrying capacity of the FEC columns. The axial load and axial deformation of individual element in the composite section were determined. Load carrying capacity of different elements of column group SCN4B and SRC1 are shown in Figs. 9 (a) and (b) respectively. It has been observed that in these test columns the contributions by structural steel sections are 30% and 26% and longitudinal reinforcements are 13% and 18% respectively on the ultimate capacity of the columns. On the other hand, concrete has been found to provide 57% and 56% of the total load carrying capacity of these composite columns respectively. The contribution of the individual elements of these FEC columns are very close in both the cases. More over increasing the core steel ratio from 1% to 2% the axial capacity is increased by 4% for normal strength concrete of columns groups SCN4A and SCN4B. Similarly, a comparative study has been carried out among these columns SRC1, SRC4 and SRC7 which were constructed only varying the percentage of structural steel. The load-deflection behaviour of columns SRC1 and SRC7 are shown in Fig. 7. The structural steel ratio in these columns are 5.16%, 5.9% and 2.35% respectively (shown in Fig. 10). From the nonlinear FE analysis the ultimate load carrying capacity of these columns (SRC1, SRC4 and SRC7) under concentric gravity loads are found to be 1114 kN,1245 KN and 670 KN respectively.



(a)

(b)

Fig. 8 Deformed shapes and stress contour at failure in column (a) Numerical (b) Experimental



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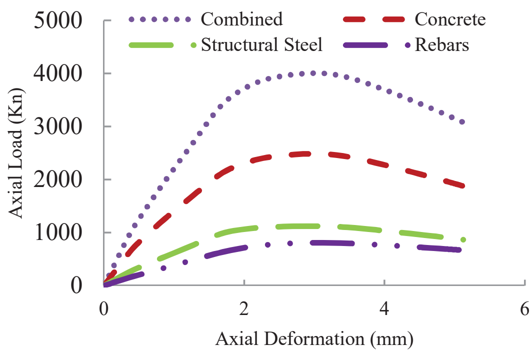

(b)

Fig. 9 Contribution of individual elements in FEC columns

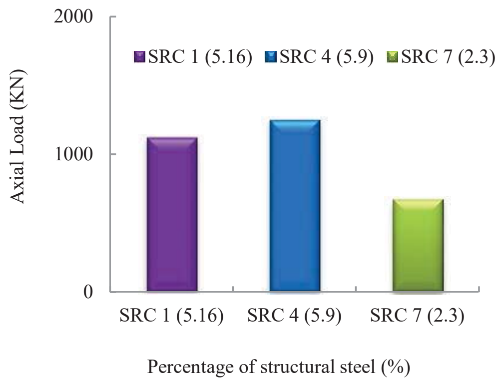

Fig. 10 Load verse percentage of structural steel in columns

The contributions of the structural steel shape of columns SRC1, SRC4 and SRC7 on the total axial strength are 28%, 27% and 19% respectively. Increasing the steel ratio from 2.35% (column SRC 7) to 5.9% (Column SRC 4) result in an increase in axial capacity by about 86% for structural steel. Therefore, structural steel ratio of the composite has significant effect on the axial strength and ductility.

VI. Conclusions

Experimental research on behaviour of two sizes square short FEC column subjected to short term axial load has been presented in this paper for two different concrete strengths and different percentage of structural steel. The complete experimental load-deflection behavior of the composite column specimens has been attained in the study. This study also conducted a nonlinear 3D FE analysis on the current experimental test specimens and experiments carried out by [3] for FEC columns under axial load. The inelastic material properties of structural steel, concrete, longitudinal and transverse reinforcement have been incorporated in the models. Nonlinear material behaviour for concrete has been simulated in FE analysis. Geometric nonlinearities are also included in the model. The composite column strengths, axial shorting at failure and failure modes of the columns were predicted using FE model. The comparison between the experimental and numerical results showed that the FE models predict the experimental behaviour of FEC columns under concentric gravity loads with good accuracy.

The developed model was used to isolate the contributions of concrete and steel section individually for columns SCN4B and SRC1. It was found that concrete carried about 57% and structural steel 28% axial load of the total capacity of FEC columns in both the case. The effect of structural steel ratio on the behaviour of FEC columns was also studied. The structural steel ratio was found to have significant effect on the strength and ductility of FEC columns.

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