Finite Element Modeling of Hollow-Core Steel-Concrete-Steel Columns Subjected to Pure Torsion

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ABSTRACT: This paper presents a finite element (FE) modelling of concrete-filled steel-concrete-steel columns (HC-SCS) subjected to pure torsion. The HC-SCS column consists of a concrete wall sandwiched between two steel tubes. The steel tubes confined the outer and the inner surfaces the concrete tube. LS-DYNA software was used to develop FE analysis of three-dimensional HC-SCS models to simulate torsional loading. FE models were validated against the experimental results of six HC-SCS columns tested under pure torsion gathered from literature. The FE results were in a good agreement with the experimental backbone curves. The difference between the experimental results and FE results was lower than 10.6% in predicting the torsion capacity of the columns. Parametric study was conducted to investigate the effect of the diameter, the diameter-to-thickness ratio (D/t_o), and the yield stress (f_{yo}) of outer steel tube on the torsional capacity and torsional angle of the HC-SCS column. The HC-SCS column's torque capacity increased with the increase in outer steel tube's strength, diameter, and thickness. The torsional angle of the column increased with the increase in outer steel tube's strength, thickness, and decreased with increase in its diameter.

1 INTRODUCTION

A vast research was being focused on developing accelerated bridge columns that reduces the onsite construction time. This idea leads to innovative concrete-filled steel tubular (CFST) bridge columns. CFST's are widely being used as bridge columns and high rise building structures. These columns reduce site construction time and improve work zone safety. These columns were used as bridge columns, piles, and were extended to high rise structures especially in Japan, China and Europe. There are more advantages of CFST's over RC or steel structures, but due to lack of design provisions, these structures were not in common (Dawood et al. 2012). Spalling of concrete core was avoided by confinement of steel tube that acts as shear reinforcement while buckling of steel tube was avoided by the bracing action of concrete core.

The construction cost increases when the columns self-weight increases. To address this issue, the member was made with hollow inside as concrete-filled double-skin tubular (CFDT) column. The CFDT column consists of two concentric tubes and sandwiched concrete between them. The tubes of the CFDT columns could be either FRP or steel tubes (Abdelkarim and ElGawady 2014a). The CFDT columns were represented as hollow-core steel-concrete-steel (HC-SCS) columns. Recent investigations were focused on the behavior of HC-SCS columns under different loading conditions (Teng et al. 2007). A conclusion was drawn that the HC-SCS column with circular section has superior performance compared to other shapes.

Although, pure torsion had no real world existence but it combines as flexural-torsional or axial-torsional or axial-flexural-torsional loading. During earthquakes, torsion loading becomes more critical. The main idea to be focused on the study of pure torsion was only to make reasonable assessment. Investigation of torsional behavior of HC-SCS columns were studied by Han et al. (2007) and Huang et al. (2013). Better performance was observed in the HC-SCS columns over the conventional reinforced concrete (RC) columns due to confinement for the concrete core that delays failure and improves ductility and strength.

The current study was focused on modelling and analyzing of six HC-SCS columns subjected to pure torsional loading using LS-DYNA software. These models were verified against the experiment results gathered from Huang et al. (2013). An extended parametric study was conducted to investigate the effect of important parameters on the behavior of HC-SCS under pure torsion.

2 FINITE ELEMENT MODELING

2.1 Geometry and loading

A total of six columns named CO111, CO112, CO211, CO212, CO311, and CO312 were replicated from the experimental work. All the columns were symmetric about X and Y-axis while rotation was about Z-axis. The height of each column was 550 mm. The diameter of outer steel tube (D) was 165 mm with thickness (t_o) varied from 3 mm - 4.6 mm, and the diameters of inner steel tube (d) were either 42 mm or 75 mm with thickness (t_i) varied from 3 mm-5 mm. The concrete shell thickness was ranged from 40.4 mm-58.5 mm. The steel plates located at top and bottom of the column were 235 mm x 235 mm x 25 mm. The loading plate was 94 mm x 324 mm x 25 mm (See Fig. 1).

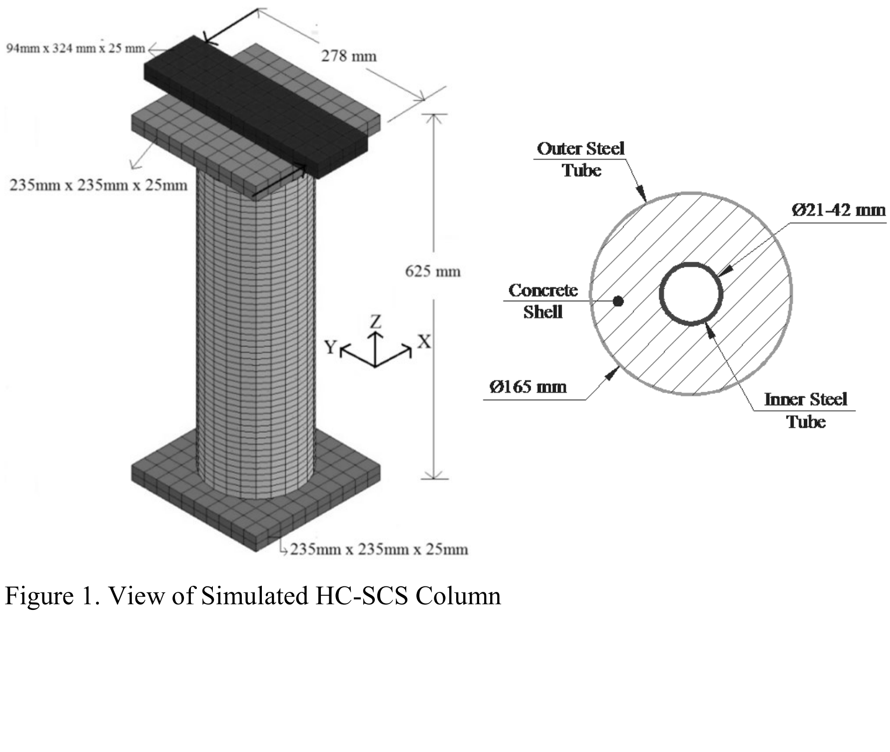


Figure 1. View of Simulated HC-SCS Column

The elements dimensions were identified by doing sensitive analysis. The solid part of the columns (concrete core and steel plates) were modelled using 8-node brick elements. Steel tubes were modelled using 4-node shell elements. Each FE model had 11,072 elements and 13,047 nodes. To avoid spurious singular modes of the elements, hourglass control was used. A hourglass value of 0.1 was set for all the columns.

2.2 Material models

The Karagozian and Case Concrete Damage Model Release 3 (K&C model), developed from theory of plasticity was used in this study because it has good correlation with the experimental results from previous studies (e.g. Abdelkarim and ElGawady 2014b). All of the columns had a concrete cylindrical compressive strength (f_c') of 42 MPa. The fractional dilation parameter (ω) was set to a default value of 0.5. The equation of state (EOS), that controls the compressive behavior of the concrete under tri-axial stresses were automatically generated.

The material model "003-plastic_kinematic" was used to define the steel tube's elastic-plastic stress-strain curve. The main parameters needed yield stress (f_y), elastic modulus (E), which was taken as 200 GPa, and Poisson's ratio 0.3 were used for all the columns. The yield strength of outer and inner steel tubes (f_{yo} and f_{yi}, respectively) were varied between 260 MPa and 365.4 MPa. "Add Erosion" parameter that capture the failure was added to the material model of the steel tube. The strain value of steel tubes at failure was considered as 0.04 (Abdelkarim and ElGawady 2014a). Rigid material was defined for all of the steel plates.

2.3 Contact elements

Surface-to-surface was used to simulate the contact interface between the concrete shell and the steel tubes that considers the slip and separation that occurs between master and slave contact pairs. Hence, slip/debonding was displayed if occurred between the concrete wall's surface and the steel tube's surface. The coefficient of friction between the steel tubes and the concrete shell was 0.6 (Rabbat et al. 1985). Tied node-to-surface contact element was used to connect the steel plates to the column's top and the bottom to simulate the full fixation as in the experimental work. Tied surface-to-surface contact element was used between loading plate and the top steel plate for full fixation.

2.4 Loading and boundary conditions

The bottom steel plate was directionally and rotationally restrained as in experimental work. The loading plate was restrained directionally in the Z-direction because it was restrained from the hydraulic jacks used in the experiment. Torque was applied in equal and opposite directions at the end of the loading plate with an arm length of 278 mm (Fig.1). The torque load applied in the experimental work was till the jack reached its maximum travel stroke that indicates the loading was stopped before the failure of the column. Finite element analysis was conducted until the column failed in the form of either the steel tubes rupture or the concrete shell failure. The summary of variables is listed in Table 1.

Table 1. Summary of Columns Variables (reproduced after Huang et al. 2013)

Specimen label	Outer tube		Inner tube		f_{yo}, MPa	f_{yi}, MPa	f_{cu}, MPa
	D, mm	t_o, mm	d, mm	t_i, mm			
CO111	165	3.0	42	3.0	260.0	326.6	50
CO112		3.0	75	5.0	260.0	355.4	
CO211		4.0	42	3.0	286.4	326.6	
CO212		4.0	75	5.0	286.4	355.4	
CO311		4.6	42	3.0	365.6	326.6	
CO312		4.6	75	5.0	365.6	355.4	

3 RESULTS AND DISCUSSIONS

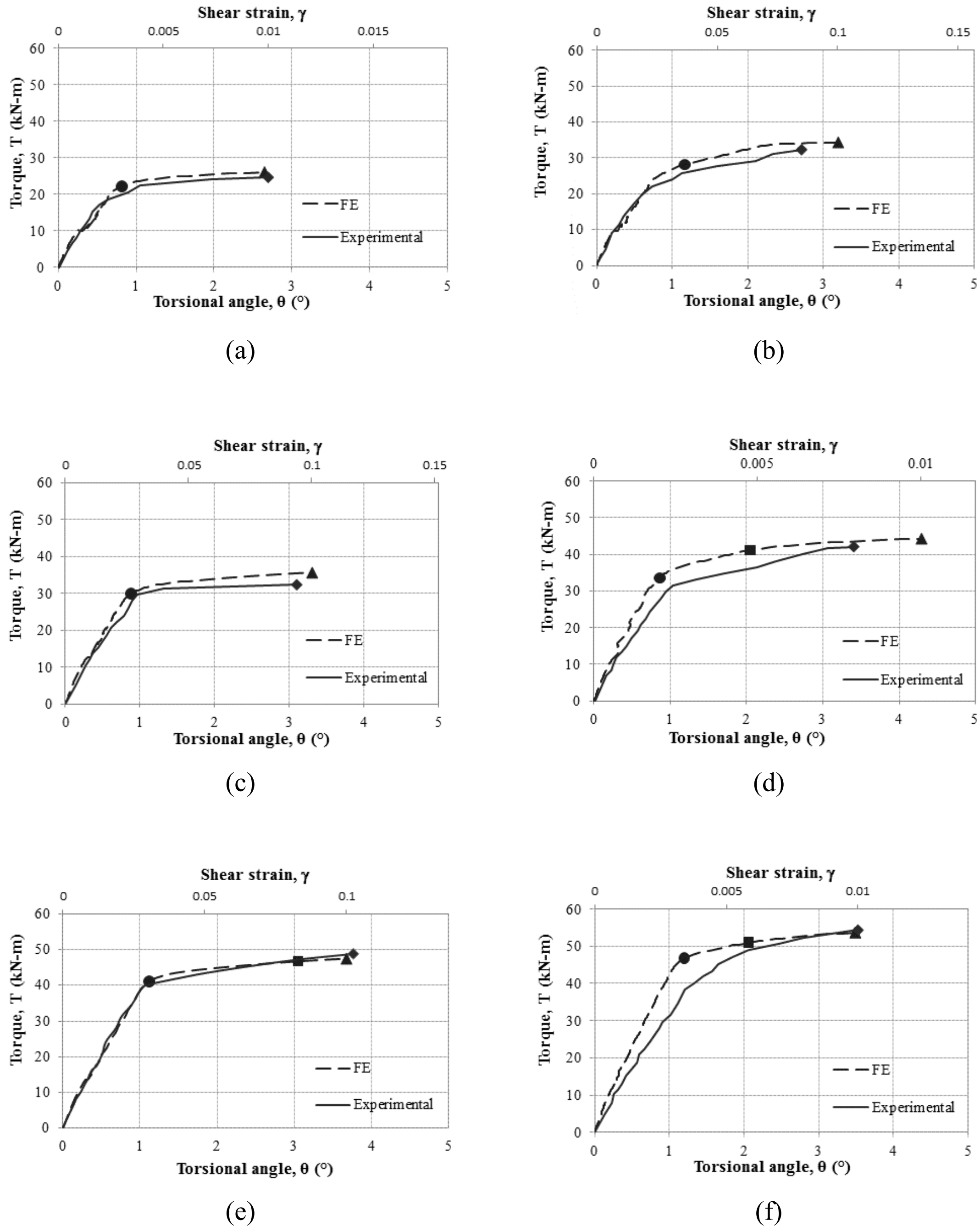
The torque capacity (T_{FE}) of the FE column was considered when the steel tubes of the column reached shear strain value of 0.01 (Huang et al. 2013) where the curve between the torque and torsional angle was stabilized. Torsional angle(θ) was calculated at the top of the column. A comparison was drawn between the FE analysis results and the experimental results (T_{ue}) from Huang et al. 2013 was shown in Table 2 and Fig. 2. A good agreement occurred between the two studies with an error between 1.3% and 10.2% (Table 2). The results in Table 2 and Fig. 2 were terminated when the steel tube‘s shear strain reached 0.01 since the experimental results were also terminated at 0.01 shear strain of steel tube. However, the manuscript will further discuss more in detail about the rupture of the each test specimen and its corresponding torsional angle. The failure of mode of columns doesn't defer much but was expalined in detail.

The outer steel tube yielded prior to the inner steel tube indicates majority of the toque load was carried by outer steel tube. There was a sudden drop in shear stress versus shear strain curve for some of the concrete elements due to local damage. The damage in those concrete elements due to the maximum principal stress of these elements reached the ultimate tensile stress. The damage in concrete shell becomes severe after 0.01 shear strain in outer steel tube was countered by the confinement of the steel tube. Due to majority of the torque load taken by the outer steel tube, it fails first and then load was carried by the inner steel tube. Due to lack of confinement for concrete in outer direction, expands its volume and gets failed. After failure of concrete shell, inner steel tube alone carries load for small interval and gets failed. The failure of inner steel tube was abrupt due to absence of concrete shell that provided lateral stability.

Table 2. Summaries of Experimental Results (Huang et al. 2013) vs. Finite Element results

No.	T_{ue}, kN.m	T_{FE}, kN.m	% error in torque capacity in FE *	θ (experimental) (°)	θ (FE) (°)	% error in Torsion angle in FE
CO111	24.6	26.0	5.4%	2.7	2.7	0
CO112	33.2	34.5	3.9%	2.7	3.2	18
CO211	32.3	35.6	10.2%	3.1	3.0	3
CO212	42.1	44.3	5.2%	3.4	4.3	26
CO311	48.8	47.5	2.6%	3.8	3.7	3
CO312	54.3	53.6	1.3%	3.5	3.5	0

* The percentage of the absolute value of the difference between the experimental and the FE/Analytical torsion capacities divided by the experimental torsion capacity

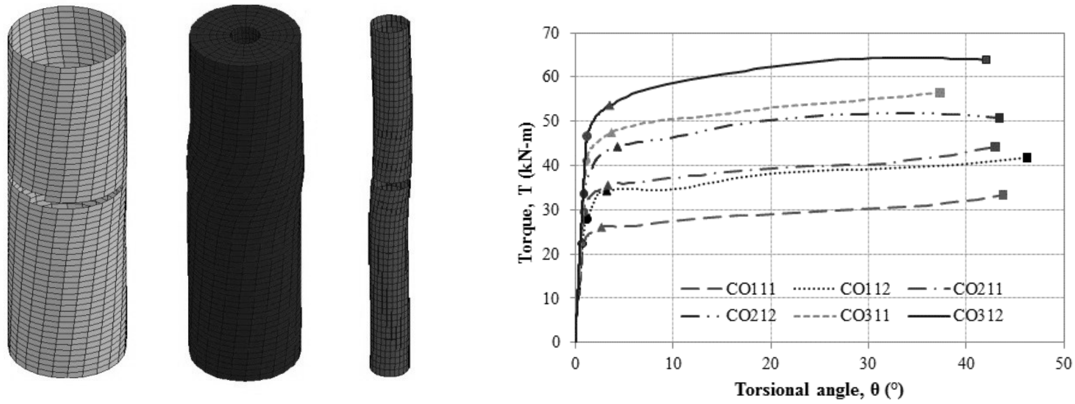


● yielding of outer steel tube ■ yielding of inner steel tube
▲ T_{FE} of FE column ◆ T_{ue} of experimental column

Figure 2. Experimental (Huang et al. 2013) vs. FE backbone curves for specimens: (a) CO111, (b) CO112, (c) CO211, (d) CO212 (e) CO311, (f) CO312

Higher torsional angle was observed at the end of elastic plastic state due to stiffness degradation. There was no vertical debonding/slip observed between the concrete shell and steel tubes. The shear stress carried by outer steel tube was almost double the one carried by the inner steel tube. But due to strain hardening in outer steel tube after elasto-plastic limit, increment in shear stress of the outer steel tube was decreased resulting in higher shear stress concentration in inner steel tube. This results in higher shear stress concentration on inner steel tube than outer steel tube at failure of the column. The resistance of the column was decereased by 70% after the rupture of outer steel tube. The concrete’s maximum shear stress was considered when the initial small portion of concrete elements failed prior to failure of column.

The typical failure of FE column was illustrated in Fig. 3. The increase in volume of outer steel tube during the experimental work was verified in the FE analysis. Fig. 3 also shows the torque-torsional angle for the columns until the rupture of the steel tube. The failure of all of the six columns was in similar manner with hoop rupture in outer steel tube followed by diagonal crack in concrete shell followed by abrupt failure of inner steel tube.



● Yielding of steel tube
■ Torsional moment at failure of FE column ▲ T_{FE} of FE column

Figure 3. Ultimate failure of FE column and corresponding torque vs torsion angle curve

4 PARAMETRIC STUDY

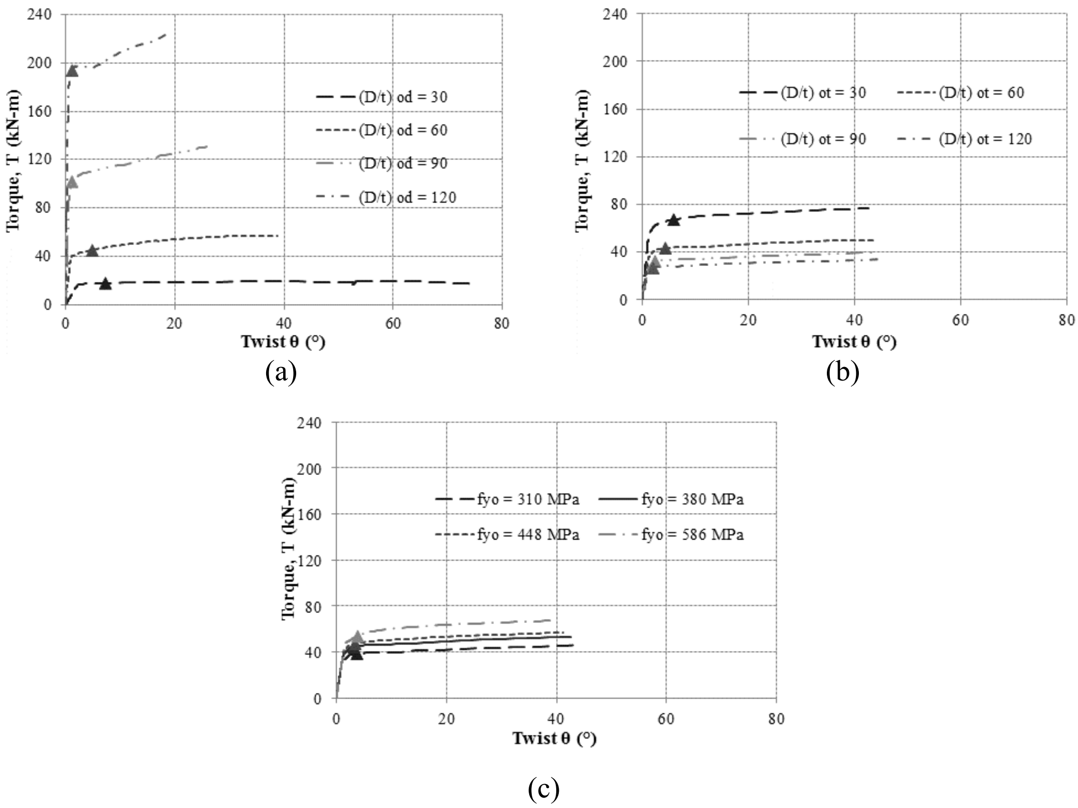
Parametric study analysis was conducted in an attempt to study the influence of parameters on the HC-SCS column‘s torque capacity and torsional angle. The main parameters D/t_{so} and strength of outer steel tube were focused in this manuscript. The back bone curves and the column’s torque capacity are illustrated (Fig. 4).

4.1. Diameter-to-thickness ratio of outer steel tube

The diameter-to-thickness ratio of the outer steel tube (D/t_o) had more influence over the torque capacity of the column. The diameter and thickness was varied to study the influence of concrete shell thickness and buckling of the outer steel tube. D/t_o of the steel tube was varied between 30 and 120. When the diameter-to-thickness increased, the torque capacity decreased while the torsional angle increased (Figs. 4(a) & 4(b)). When the diameter increased, the torque capacity increased while the torsional angle decreased.

The increase in diameter of the outer steel tube increased the concrete shell’s thickness and improved the lateral stability to the outer steel tube. The concrete shell became more stiffer and brittle with increase in diameter resulting in overall decrease in torsional angle at failure. The contribution of inner steel tube towards capacity becomes more significant with lower concrete shell thickness due to small difference in section modulus of outer and inner steel tube. The diagonal cracks were decreased with concrete shell's thickness. The confinement action of steel tubes was decreased with concrete shell's thickness due to more stiffness in concrete shell.

The stiffness of column was linearly increased with thickness till yielding. The behavior of steel tubes were not changed after yielding. AISC manual defines the critical d/t ratio at which steel tube local buckling in flexure occurs at $0.07 * \left(\frac{E}{f_y}\right)$ in the parametric study was 36.8. The stability provided by concrete shell avoided buckling of steel tube even at 120 of d/t ratio. The stress taken by concrete shell increased with thickness due to confinement. The torsional angle increased with the increase in thickness (decrease in D/t ratio) till the 0.01 shear strain and remained same at the failure.



▲ T_{FE} of the columns

Figure 4: Effects on HC-SCS behavior by: (a) D/t_o of outer steel tube by varying diameter; (b) D/t_o of outer steel tube by varying thickness; (c) strength of outer steel tube

4.2. Strength of outer steel tube (f_{yo})

The outer steel tube's strength was between 310 MPa and 586 MPa. Since the column's capacity mainly depends on the failure of the outer steel tube, the capacity of column increased with the strength of outer steel tube. There was 39% increase in torque capacity and 5% increase in twist with 86% increase in strength of outer steel tube (Fig. 4(c)). The initial shear crack occurred in concrete was not affected by the strength of outer steel tube. It confirmed that members behave individually before yielding however the confinement action for the concrete shell was improved after yielding. Stress carried by the inner steel tube before yielding of column would be decreased with strength. But finally at failure, stress carried by inner steel tube remains same. The stiffness of the column is increased with strength. Strain hardening was improved with the strength. The torsional angle was decreased with the strength due to increase in stiffness. The failure mode of outer steel tube changed from hoop to diagonal rupture with the increase in strength of the outer steel tube.

5 SUMMARY AND FINDINGS

The proposed model using LS-DYNA was able to predict the behavior of HC-SCS columns under pure torsion. The Karagozian and Case Concrete Damage Model Release 3 (K&C model), with automatically generated parameters, were in good agreement with the exerimental results. The average error in column‘s torsion capacity from the FE analysis was was 4.8%.The HC-SCS column’s torque capacity increased with the increase in outer steel tube's strength, diameter, and thickness. The torsional angle of the column increased with the increase in outer steel tube's strength, thickness, and decreased with increase in its diameter.

6 ACKNOWLEDGEMENT

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