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Flexural Rigidity Evaluation of Seismic Performance of Hollow-Core Composite Bridge Columns

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Abstract:
This paper investigates experimentally the seismic behavior of two hollow-core fiber-reinforced polymer-concrete-steel HC-FCS columns under cyclic loading as a cantilever. The typical precast HC-FCS member consists of a concrete wall sandwiched between an outer fiber-reinforced polymer (FRP) tube and an inner steel tube. The FRP tube provides continuous confinement for the concrete wall along the height of the column. Five large-scale HC-FCS columns were investigated during this study to estimate the effective flexural (which is an important factor to define the buckling capacity and deflection of such columns) and the effective structural stiffness of the composite columns. These columns have the same geometric properties; the only difference was in the thickness of the inner circular steel tubes and the steel tube embedded length into the footing. A three-dimensional numerical model has been developed using LS_DYNA software for modeling this large scale HC-FCS columns. The nonlinear FE models were designed and validated against experimental results gathered from HC-FCS columns tested under cyclic lateral loading and used to evaluate the effective stiffness's results. The estimated effective stiffness results that obtained from the experimental work were compared with the FE results. This study revealed that the effective flexural and the effective structural stiffness for the HC-FCS columns need more investigation to be addressed in the standard codes. Since the embedded hollow core steel tube socket connections cannot reach the fully fixed end condition to act as a cantilever member subjected to a lateral load with a fully fixed end condition. Moreover, the effective stiffness results were found to be highly sensitive to the steel tube embedded length and slightly to the unconfined concrete strength.

Introduction

It's estimated that Americans spend 14.5 million hours per day on traffic. 10 to 15% of that congestion is caused by work zones even when it occurs in the off-peak, they increase traffic congestion [1]. The need for a new solution is highly requested to reduce the amount of time it takes to build our roads and bridges from months to several hours. Accelerated bridge construction ABC includes such elements as prefabricated modular units that are built off-site in a controlled environment and then transferred to the construction area for rapid installation. ABC reduces traffic disruptions and life-cycle costs and improves construction quality and safety, resulting in more sustainable development [2]. One technique to accelerate bridge construction is to use precast bridge columns with excellent seismic performance.

An excellent candidate for precast columns is the concrete-filled tube, which consists of a hollow tube made out of steel or fiber reinforced polymer filled with concrete. Another candidate for precast columns is the hollow-core steel-concrete-steel (HC-SCS) columns consisting of two generally concentric tubes with concrete shell between them [3-8]. The concrete infill is confined by both tubes, resulting in high concrete confinement and column ductility [9]. All these mentioned research showed the superior seismic and axial capacity of HC-SCS columns.

Several regions around the world are susceptible to earth quake where large ductility demands are imposed on bridge columns. The design and the construction of a column-footing connection are crucial for precast columns to provide the ductility demands. Thus, under an earthquake event, all damage in the precast assembly is confined to the column and the adjacent element experiences no damage, emulating cast-in-place construction performance. However, there is neither CFST neither HC-FCS columns have a satisfactory codes in standards such as AASHTO LRFD Bridge Design Specifications [10], the American Institute for Steel Construction Steel Design manual (AISC), and the ACI American concrete Institute [11]. Based on the available literature, limited studies have been focused on the design for CFST column-to-cap beam connections using different types that were proposed in the literature for CFST columns includes welded and bolted steel plate, embedded base, and rebars, and embedded structural steel connections were used for precast column-footing connections [12-14]. Likewise, limited have been conducted on the understanding the performance of HC-FCS column [15], or on the column-footing connections [16]. These previous studies have concentrated mainly on determining the critical embedded length of the steel tube into the cap beam or the footing, but effect of the connection rigidity had not addressed comprehensively.

In this paper, a finite element model using LS_DYNA software were simulated to estimate the effective stiffness and flexural stiffness of HC-FCS column footing connections in addition to the standard codes and compared to the experimental test results.

Experimental work

Five 0.4-scale HC-FCS columns with different steel tube thicknesses and an embedment length were investigated in this study (Table 1). These columns were tested under constant axial load and lateral cyclic load. The tested HC-FCS columns have a circular cross-section with an outer diameter of 610 mm and a clear height of 2,032 mm. The lateral load was applied at a height of 2,413 mm with shear span-to-depth ratio of approximately 4.0. The column consisted of an outer filament-wound GFRP tube having a constant thickness of 9.5 mm along the height of the column. The inner steel tube had an outer diameter of 406 mm. A concrete wall having a thickness of 102 mm was sandwiched between the steel and FRP tubes (**Fig. 1**).

The columns' label used in the current experimental work consisted of four segments. The first segment is a letter F referring to flexural testing followed by the column's height-to-outer diameter ratio (H/D_o). The second segment refers to the column's outer diameter (D_o) in inch. The third segment refers to the GFRP matrix using E for epoxy and P for Iso-Polyster base matrices; this is followed by the GFRP thickness in 1/8 inch (3.2 mm), steel thickness in 1/8 inch (3.2 mm), and concrete wall thickness in inch (25.4 mm).

The HC-FCS column construction sequences and details have been illustrated in the literature [15, 17]. The mechanical properties of the Steel tube, FRP tube, concrete mix design, and rebar are mentioned in details in the literature [15, 17].

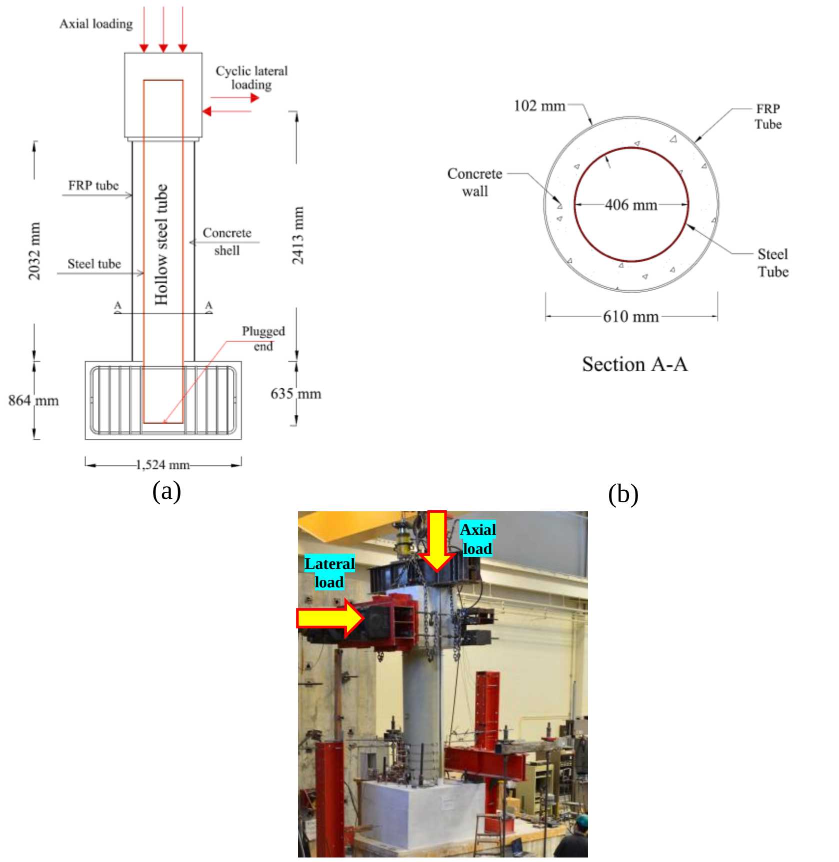

Fig. 1: Construction layout of the column (a) Elevation; (b) column cross-section; (c) HC-FCS column perior to the test

Table 1: Summary of the investigated HC-FCS columns

#	Column ID	Steel tube thickness t_s (mm)	Steel tube embedded length l_e (mm)
1 [15]	F4-24-E344	12.7	635
2 [18]	F4-24-E324	6.35	635
3	F4-24-E3(1.5)4	4.8	635
4	F4-24-P1(0.8)4	2.8	508
5	F4-24-E3(0.5)4	1.6	508

Parametric study

FE models verification

Five-dimensional numerical models have been developed using LS_DYNA software for modeling of large scale HC-FCS columns. The footing, concrete wall, and loading stub were modeled using solid elements with an average length of 25.4 mm and constant-stress onepoint quadrature integration to reduce the computational time and increase the model stability. The outer FRP and inner steel tubes were simulated using shell elements with an average height of 25.4 mm. The hourglass type and coefficient used during this study were 5 and 0.03, respectively. The models have been described in details By Abdulazeez et. al. (2017) [19].

Loading protocol and test setup

Constant axial load, P, of 489.3 kN (110 kips) corresponding to 5% of the calculated P_o (of an equivalent RC-column with the same diameter 610 mm (24 inches) and 1% longitudinal reinforcement ratio was calculated as in [32]) was applied to the column using three external prestressing strands on each of the west and east sides of the column. After applying the axial load, static cyclic lateral load was applied in a displacement control using two hydraulic actuators connected to the column loading stub. The loading regime is based on the recommendations of FEMA 2007. Two cycles were performed for each displacement amplitude [17].

Results and discussion

Rigidity evaluation

The combined load mechanism applied on the HC-FCS columns is described in **Fig. 2**. It simply represented by a cantilever member subjected to a lateral load at the top with a fully fixed end condition at the bottom. The structural stiffness (K) for this assembly can be estimated using as **Eq. 1**. However, the embedded hollow core steel tube socket connections cannot reach the fully fixed end condition; the flexural stiffness of the embedded steel tube socket connection should be reduced from the fully fixed cantilever. Thus, more comprehensive study should be conducted using parametric analysis.

$$F = K\,\delta\,;\;\; K = \frac{3EI_{eff}}{H^3} \tag{1}$$

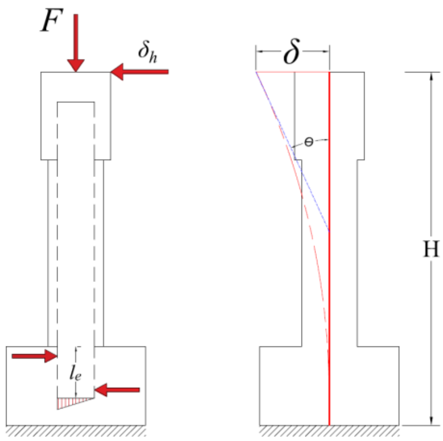

Fig. 2: HC-FCS column load resisting mechanism

The maximum elastic deflection is affected by the main parameters, including the embedded length, footing and column concrete strength, thickness of the steel tube. The effective flexural stiffness of has been estimated using the fololiwing standarad provesions expressions:

$$EI_{eff} = E_s\, I_s + \frac{0.2E_c I_g}{1+\beta_d} \qquad \text{ACI codes (2)}$$

$$EI_{eff} = E_s\, I_s + C_3 E_c I_c\ ;\ C_3 = 0.6 + 2\left(\frac{A_s}{A_s+A_c}\right) \le 0.9 \qquad \text{AISC codes (3)}$$

$$EI_{eff} = E_s\, I_s + 0.4\, I_c\left(\frac{E_c\, A_c}{A_s}\right) \qquad \text{AASHTO codes (4)}$$

Figure 3 and table 2 illustrate the evaluated results of EI_{eff} and K values for the investigated columns from the FE and the available codes compared to the experimental test results. As shown in **Fig. 3**, the AASHTO and AISC codes predicted larger EI_{eff} than the ACI code and FE results compared to the obtained experimental test results especially for F4-24-E344 HC-FCS column. While, the predicted flexural and structural stiffness results from the FE models as well as the ACI code were close to the experimental test results.

Table 2: Summary of the predicted flexural stiffness results

	EXP.		FE		ACI Eq. (2)	AASHTO Eq. (4)	AISC Eq. (3)
Column ID	K	EI_{eff} (N.m^2)	K	EI_{eff} (N.m^2)	EI_{eff} (N.m^2)	EI_{eff} (N.m^2)	EI_{eff} (N.m^2)
F4-24-E3(0.5)4	2.2	34.5	2	32.8	23	21	18.4
F4-24-P1(0.8)4	1.5	24.2	1.4	19.6	29	33	26
F4-24-E3(1.5)4	3.3	54.3	2.8	47.2	43	59	43.3
F4-24-E324	3.8	61.3	3.7	60	47.6	68	47
F4-24-E344	2.7	42.3	2.5	41	78	89	74.5

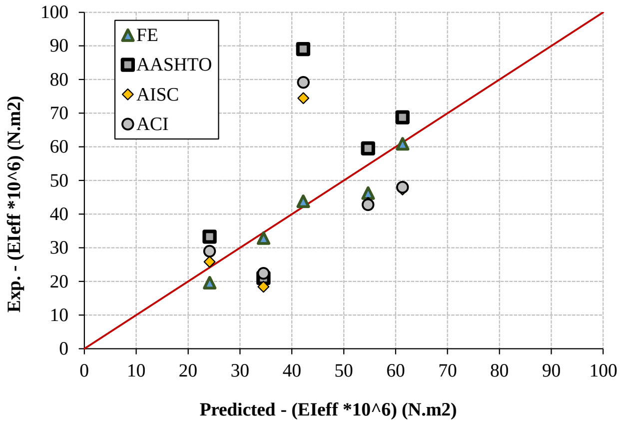

Fig. 3: Effective flexural stiffness
(Experimental vs. predicted from FE and standard codes)

The reason may due to the insufficient steel tube embedded length that led to severe footing damage and steel tube pullout, and thereby not achieving full flexural behavior for the column [16]. The lowest values of EI_{eff} were notice for columns F4-24-P1(0.8)4 and F4-24-E3(0.5)4 due to the short steel embedded length (508 mm). Consequently, the embedded length of the steel tube into the footing is a significant parameter that affects the stiffness of the entire socket connection and need to be addressed in the available codes for better HC-FCS column design.

Conclusion

The embedded length of the steel tube into the footing as well as the unconfined compressive strength for the column and the footing are significant parameters that affects the stiffness of the entire socket connection and need to be addressed in the available codes for better HC-FCS column socket connection design.

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