Journal of Engineering and Applied Sciences 13 (23): 9813-9824, 2018
ISSN: 1816-949X

Structural Behavior of Hybrid Reinforce Concrete Columns

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Abstract: This study presents experimental investigation of the structural behavior of reinforced concrete columns with hybrid cross-section (HSC and SFRC) and hybrid longitudinal reinforcement (steel and CFRP) bars. The experimental program included testing seventeen column specimens. One of column specimens was selected as a pilot column and sixteen columns were subdivided into 5 groups to study the effects of: type of concrete in the outer shell (HSC and SFRC), type of interface between core and shell (monolithic, surface intentionally roughened and surface intentionally roughened with shear connectors), type of main reinforcement and percentage of hybridization(ordinary steel and CFRP bars) and eccentricity to depth ratio (e/h) on the overal behavior, ultimate strength, deflection, cracking pattern, cracking load, modes of failure and ductility. All the column specimens have the same dimensions and tested under eccentric axial load only. The experimental work results showed that a significant improvement in the behavior and carrying capacity of the tested columns.

Key words: Hybrid column, monolithic, construction joint, shear connectors, HSC, SFRC

INTRODUCTION

Column-member, usually vertical or predominantly vertical, used primarily to support axial compressive load but that can also resist moment, shear or torsion. Columns have been used as part of the lateral-force-resisting system to resist combined axial load, moment and shear. From geometric view, the column is a member with a ratio of height-to-least lateral dimension exceeded (3), used primarily to support axial compressive load (Al-Haddad, 2016).

The outer concrete shell in a reinforced concrete column has a significant contribution in carrying the design loads. Such contribution may reach up to 40% of the ultimate loads, especially in a column having a small cross sectional area (>900 cm^2). On the other hand, the outer shell acts to protect the steel reinforcement against corrosion and fire attack. Therefore, any damage or cracks in the outer shell will affect seriously both the strength and performance of the column (Al-Yassiri, 2017).

"Hybrid strength concrete refers to a new concept of casting two or more different types of concrete in the same section. Experimental and numerical studies for the behavior of reinforced hybrid concrete construction are summarized. Aziz, 2006 introduced experimental and theoretical investigations to study the shear behavior of hybrid reinforced concrete I-beams cast monolithically. A new manner by replacing (or strengthening) a certain part (s) or layer (s) of I-shaped reinforced concrete beams by Steel Fiber Reinforced Concrete (SFRC) or High Strength Concrete (HSC) has been introduced (Anonymous, 2014). Malik, 2015 presented an experimental and theoretical investigations studied the effects of hybridization of T-shaped beam by HSC and SFRC, the presence of construction joint, using epoxy resin layer and shear connectors for flexural and shear behavior of simply supported reinforced concrete T-shaped beams. The results obtained from his adopted technique showed significant effects of SFRC and HSC on overall shear and flexural behavior of such beams. Mahdi, 2015 studied an experimental and theoretical investigations of the behavior and ultimate strength of double-symmetrical concrete corbels with hybrid reinforcement (steel and CFRP) bars subjected to vertical distributed applied load. He concluded that a significant improvement could occur in the behavior and carrying capacity in corbels of hybrid reinforcement in main tension or in horizontal reinforcement (stirrups) (Aziz, 2006). Al-Haddad, 2016 investigated experimental and theoretical investigations for shear and flexural behavior of reinforced concrete corbel-column systems made of hybrid concrete at the corbel-column connection region. The results that obtained from the experimental study was found that changing corbel concrete type from NSC-SFRC or HSC in shear and flexural behavior were increased the strength capacity and the cracking loads compared with homogenous NSC systems having same (a/d) ratio (Mahdi, 2015). Al-Yassiri (2017) studied an experimental

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and numerical investigation of structural behavior of one-way and two-way hollow core slabs made from Normal Strength Concrete (NSC) or Hybrid Strength Concrete (HYSC) and reinforced with different types of rebars, steel and CFRP reinforcing bars. The test results showed that the increase of the voids in the cross section of the slab to a certain limit leads to the weakening of the shearing resistance, reduce the ultimate load capacity and ductility of the slab compared with the solid slab (Malik, 2015).

Information about the response of the structural behavior of a concrete column with a cross section of hybrid concrete and hybrid reinforcement of steel and CFRP bars was not available. Furthermore, the presence of vertical construction joint in RC. Corbels with homogenous or hybrid concrete was not considered in previous studies".

Experimental program

Details of specimen geometry and reinforcement: All columns are identical in shape and external dimensions. The model dimensions selected in the present study were a square section of 150×150 mm and a total length of 1300 mm. The length between brackets (middle portion) is 700 mm and the dimensions of the bracket are 150*250*300 mm. All columns are reinforced with eight (ϕ 8 mm) deformed steel bars as longitudinal reinforcement (p = 0.0178) [except (C_7) and (C_8)]. Columns (C_7 and C_8) are reinforced with four (ϕ 8 mm) deformed steel bars and eight (ϕ 6 mm) CFRP bars as longitudinal reinforcement and concrete cover (20 mm). All columns are reinforced with ties (ϕ 4.5 mm@125 mm spacing) deformed steel bars. All columns are designed according to Anonymous (2014), Al-Haddad (2016) specifications.The dimensions and reinforcement details of test specimens are shown in Fig. 1.

Description of test groups: The experimental program comprises of five groups of column specimens with gomogenous and hybrid cross-sections and hybrid longitudinal reinforcement. The first group consists of four columns (C_3-C_6). The main variables were types of concrete of the shell (H_2 or SFRC), the eccentricity of the axial load (E_1 or E_2) and monolithic casting between the concrete of the core and the concrete of the shell. The second group consists of two columns (C_7, C_8). The main variables were the type of the concrete of the shell (SFRC), the eccentricity of the axial load (E_1 or E_2), monolithic casting between the concrete of the core and the concrete of the shell and 50% hybridization of the longitudinal reinforcement (steel and CFRP bars). The third group consists of four columns (C_{10}-C_{13}). The main variables were types of concrete of the shell (H_2 or SFRC), the eccentricity of the axial load (E_1 or E_2) and presence of construction joint between the concrete of the core and the concrete of the shell with type of interface (surface intentionally roughened). The fourth group consists of four columns (C_{14}-C_{17}). The main variables were the same of the group three in additional to presence of shear connectors welded on the ties. Designations and details

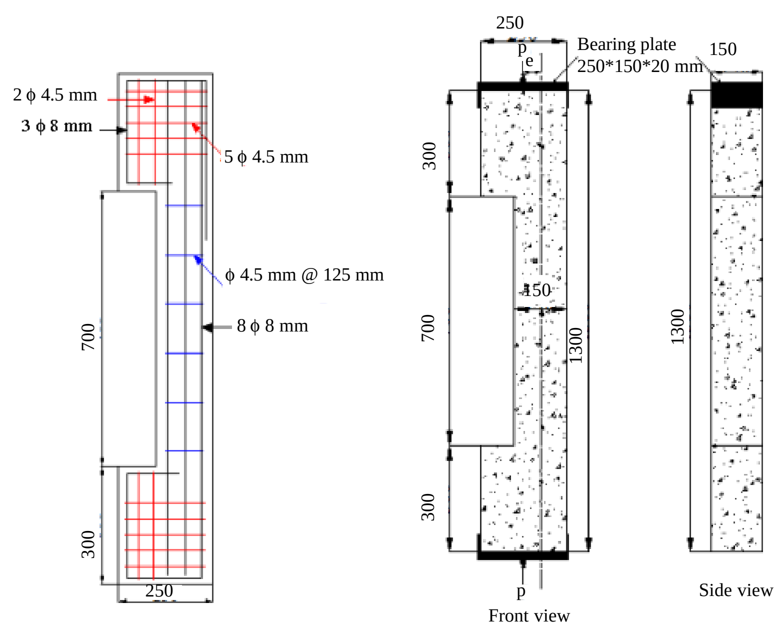

Fig. 1: Dimensions and reinforcement details of test specimens

of tested columns cross-sections are reported and presented in Fig. 2 as follows: symbols used in specimens designation refer to: C_1-C_{17} sequence of specimens in the test groups, H_1: High strength concrete type 1, H_2: High strength concrete type 2, S: Steel fiber reinforced concrete (J): construction joint between core and shell, (J_{c}): construction joint between core and shell with shear connectors, (E_{1}): the amount of the load eccentricity 50 mm (e/h = 1/3), (E_{2}): the amount of the load eccentricity 75 mm (e/h = 1/2) and (R_{50}): replacement 50% of reinforcing steel bars with CFRP bars.

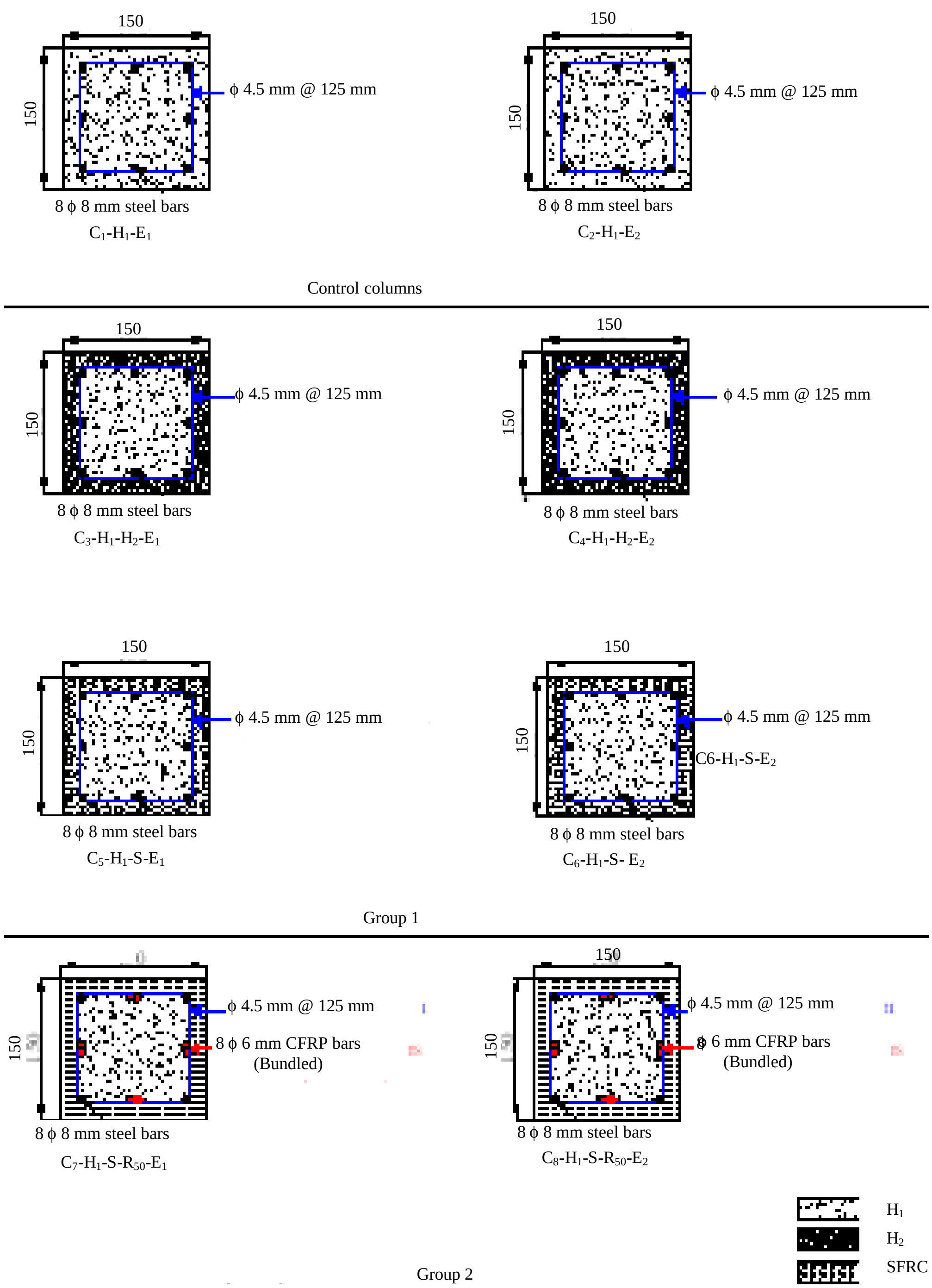

Fig. 2: Continue

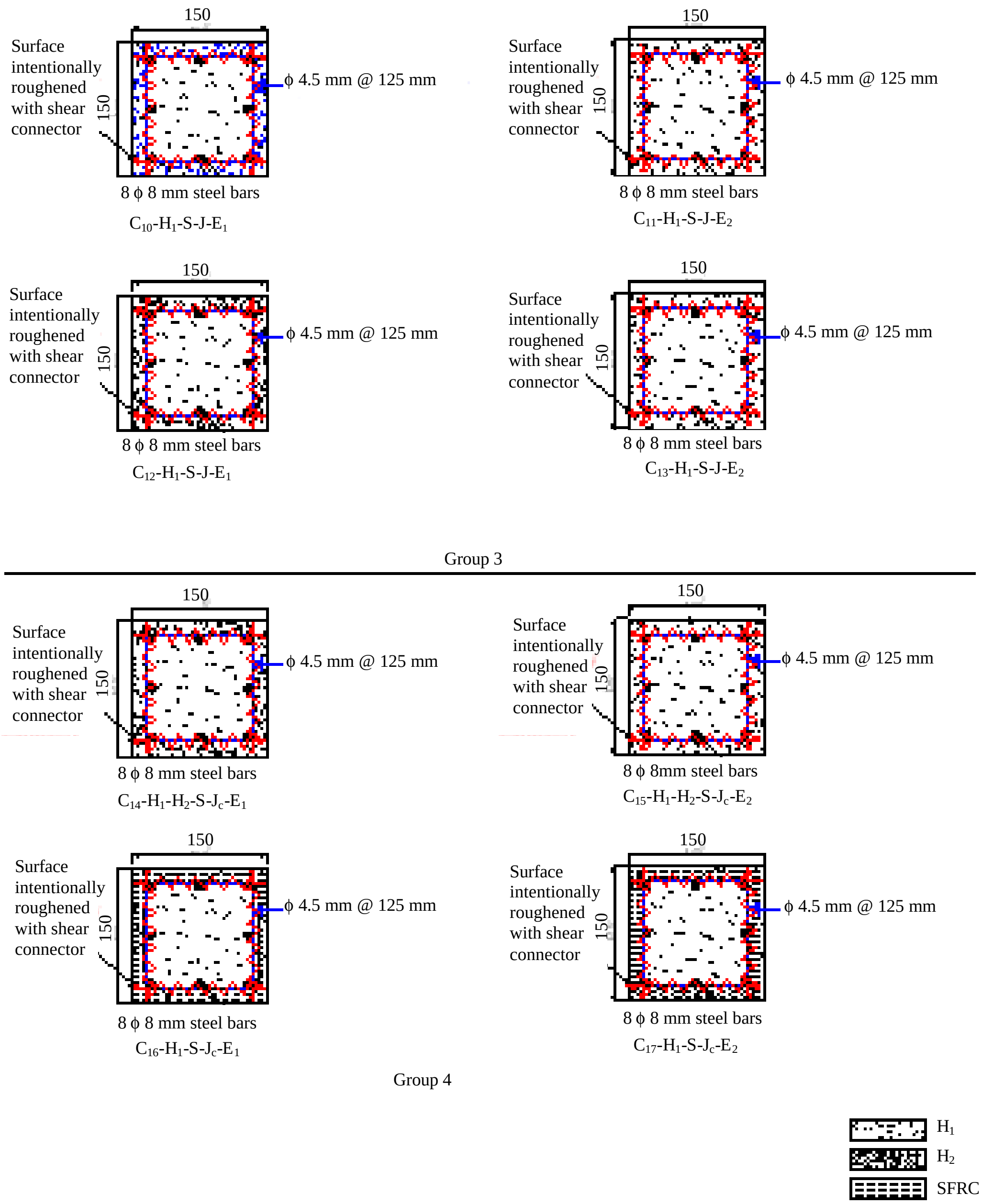

Fig. 2: Designation of test groups

MATERIALS AND METHODS

In the experimental program, tensile test of steel reinforcing bars was carried out on (ϕ 5 mm and ϕ 8 mm) deformed steel reinforcing bars with average yield strengths (fy) of 602 and 517 MPa and average ultimate strengths of 602 and 586 MPa, respectively which conform to the American specification (ASTM-A370) (Anonymous, 1983). Also, CFRP bars of Aslan 201 series were used as longitudinal reinforcement of diameter 6mm and tensile strength of 2241 MPa with modulus of elasticity 124 GPa.

Three types of self compact concrete mixes (H_1, H_2, and SFRC) were used after several trial mixes for making the specimens. Mix proportions of H_1, H_2 and SFRC are illustrated in Table 1. The concrete was prepared with Portland cement (type 1), rounded, well graded gravel of 14 mm maximum size, Natural locally available fine aggregate of nominal maximum size 4.75 mm, fresh drinking water, Hyperplast PC200 high performance super plasticizing admixture and limestone powder as filler.Also, micro stright steel fibers (0.2×13 mm) with Volume fraction (V_f = 1.0%) and aspect ratio (L_f/D_f) = 65 were used in steel fiber reinforced concrete.

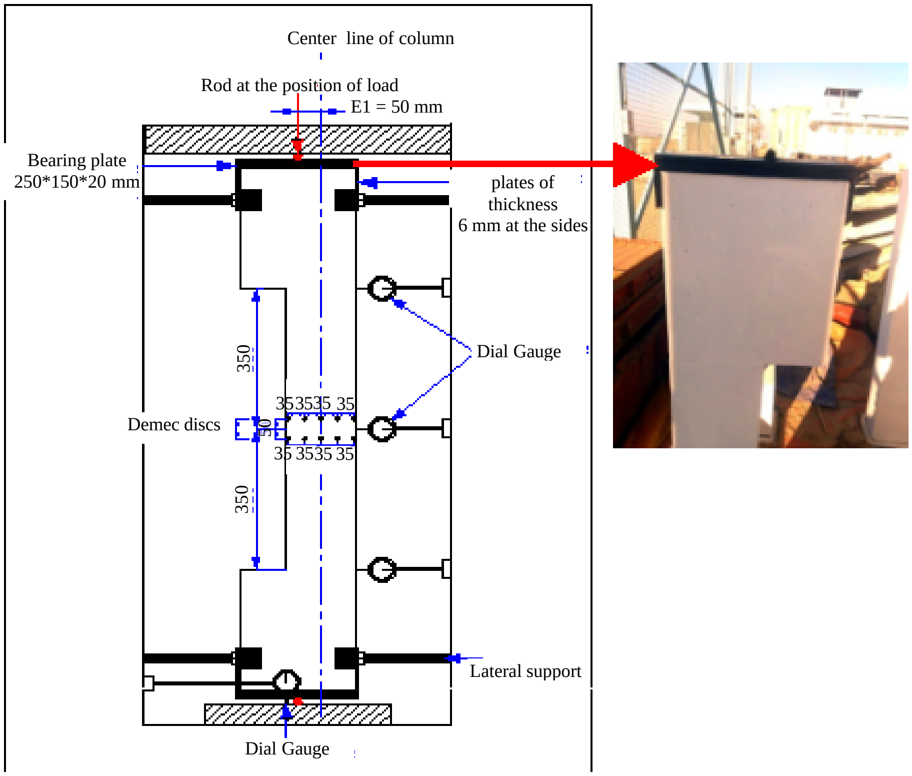

Fig. 3: Loading system

Table 1: Selected trial mixes for fresh and hardened SCC

	Quantity (km^3)		
Materials	H_1	H_2	SFRC
Cement	400	500	400
Gravel	800	840	800
Sand	850	871	850
Water	210	170	210
Hyperplast PC 200	7.20	12	7.20
Filler	120	100	120
SF	…..	…..	78

Testing: Hydraulic universal testing machine was used to test the column specimens as well as the control specimens. The testing machine has a capacity of (2000 kN). All the columns were supported axially and laterally unrestrained columns and tested up to failure. Two bearing plates of dimensions (250×150 ×20) mm with plates of the thickness 6 mm at the sides were used as caps for columns to prevent local failure. Also, lateral supports were provided to keep the column vertical before and after the loading. During each load step the corresponding lateral deflection and axial deformation were recorded as well as the first crack load. Dial gauges of accuracy 0.01mm were used to measure the lateral deflection and the axial deformation.The loading system is shown in Fig. 3. The compressive strength test of concrete cylinders (150×300) mm and cubes (150×150×150) mm were carried out on H_1, H_2 and SFRC in accordance with BS1881-116 (Richart and Brown, 1934) at test time of each specimen with average values (51.34, 72.76, 55) MPa and (62.35, 88.30, 66.62) MPa for each type of concrete mix, respectively.

RESULTS AND DISCUSSION

The main aim of the present research is to study the effect of concrete hybridization technique on the structural behavior and ultimate strength of hybrid reinforced concrete column cast monolithically or with construction joint between the concrete of the core and the shell. The overall structural behavior of column specimens with hybrid cross-sections and hybrid reinforcement were investigated and discussed.

Control columns

Specimen C_1-H_1-E_1: Figure 4a shows the load deflection response of the column specimen C_1. The maximum applied load was 507 kN .The first crack in column appeared at 136 kN. The crack pattern of the specimen is shown in Fig. 4b.

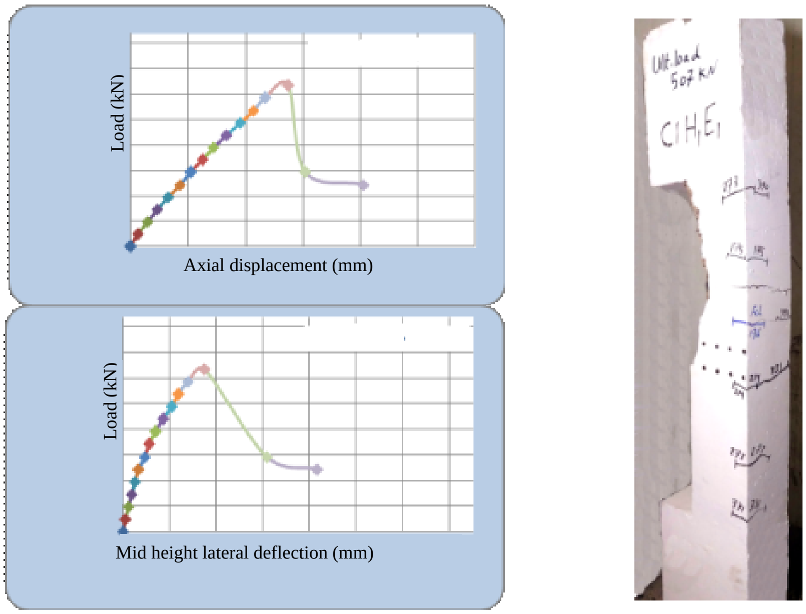

Fig. 4: a) Load-deflection of specimen C_1 and b) Crack pattern of C_1

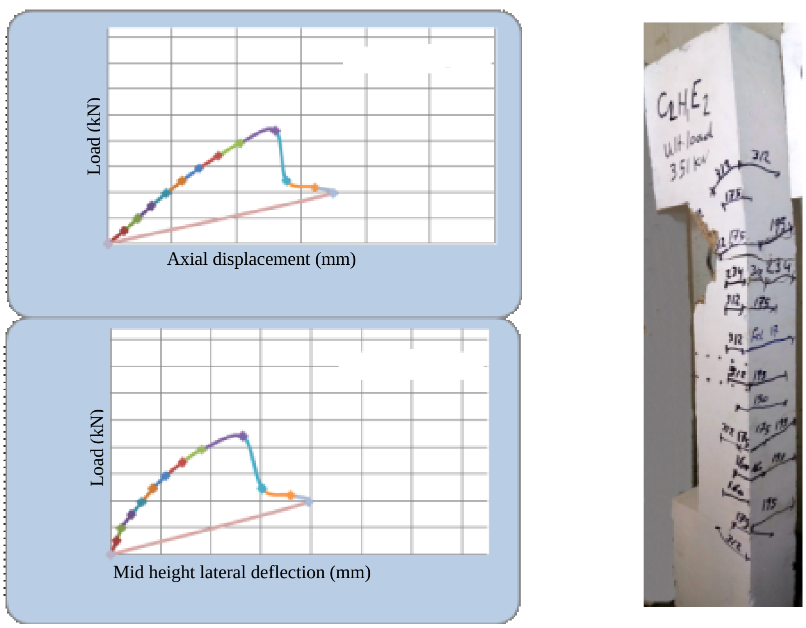

Fig. 5: a) Load-deflection of specimen C_2 and b) Crack pattern of C_2

Specimen C_2-H_1-E_2: Figure 5a shows the load deflection response of the column specimen C_2. The maximum applied load was 351 kN .The first crack in column appeared at 117 kN. The crack pattern of the specimen is shown in Fig. 5b.

Columns with monolothic casting

Columns of hybrid concrete cross section

Specimens C_3-H_1-H_2-E_1 and C_4-H_1-H_2-E_2: In these specimens, the outer shell of the columns were made with type H_1. The first crack in columns appeared at concrete load (195 and 136 kN), respectively. The failure happened on the compression side suddenly at load (526 and 425 kN), respectively. Compared with control specimens (C_1 and C_2) there is an increase in the cracking load about (43.38 and 16.32%), respectively, also, there is an increase in the ultimate load capacity about (3.74 and 21.08%) and the ductility was decreased (1.13 and 5.68%), respectively.

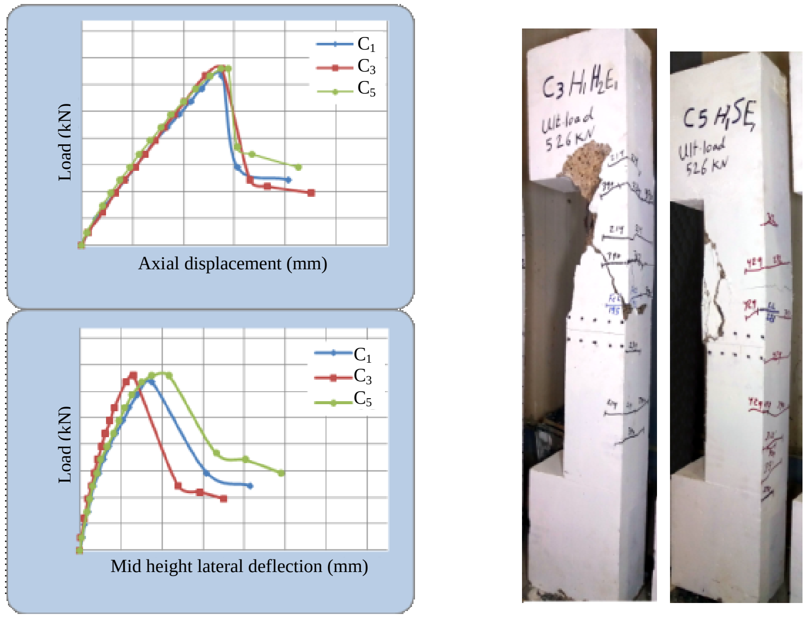

Fig. 6: a) Load-deflection of specimen C_3 and C_5 and b) Crack pattern of C_3 and C_5

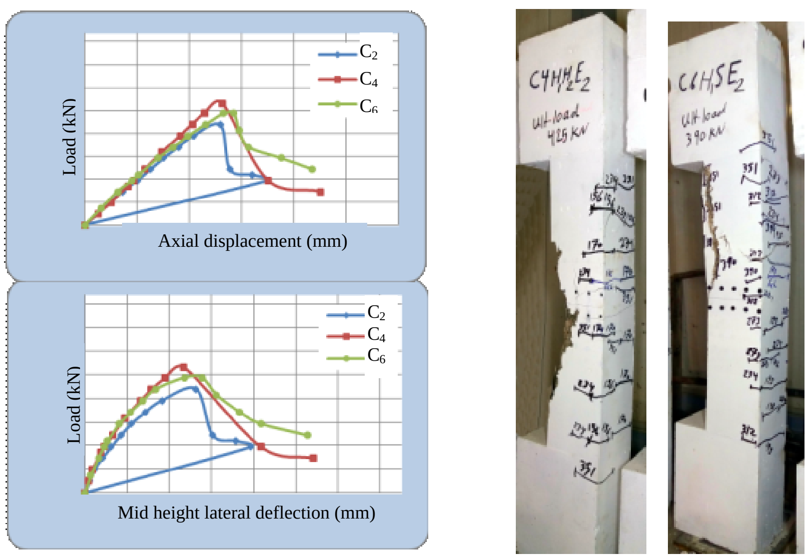

Fig. 7: a) Load-deflection of specimen C_4 and C_6 and b) Crack pattern of C_4 and C_6

Figure 6a and 7a show the load deflection response of the column specimens C_3 and C_4, respectively. Figure 6b and 7b show the crack pattern of the specimens C_3 and C_4, respectively.

Specimens C_5-H_1-S-E_1 and C_6-H_1-S-E_2: In these specimens, the outer shell of the columns were made with concrete type SFRC. The first crack in columns appeared load (273 and 175 kN), respectively. The failure at happened on the compression side gradually at load (526 and 390 kN), respectively. Compared with control specimens (C_1 and C_2) there is an increase in the cracking load about (100 and 50%), respectively also, there is an increase in the ultimate load capacity about (3.74 and 11.11%) and the ductility was increased (8.74 and 4.54%), respectively. Figure 6a and 7a shows the load deflection response of the column specimens C_5 and C_6, respectively. Figure 6b and 7b shows the crack pattern of the specimens C_5 and C_6, respectively.

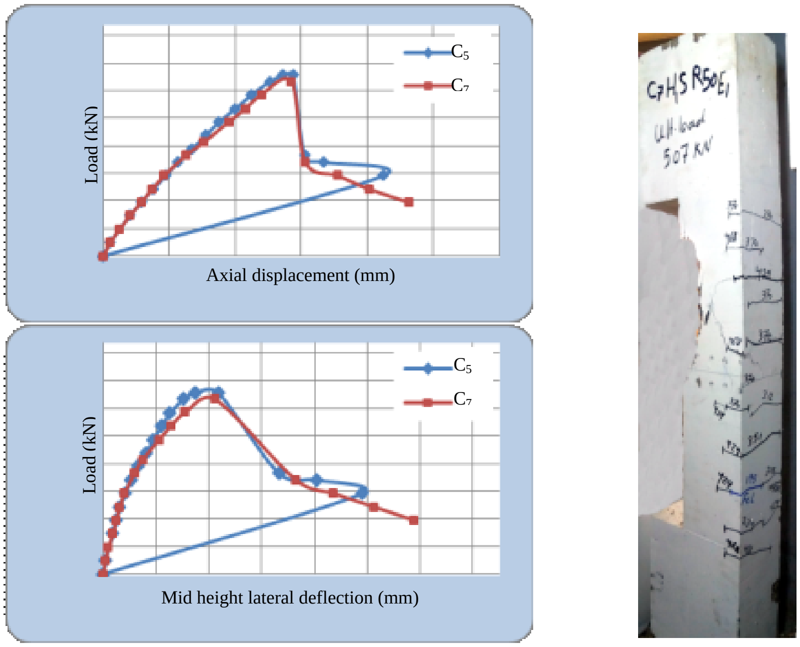

Fig. 8: a) Load-deflection of specimen C_7 and b) Crack pattern of C_7

Column specimens of hybrid reinforcement

Specimens C_7-H_1-S-R_{50}-E_1 and C_8-H_1-S-R_{50}-E_2: In these specimens, the outer shell of the columns were made with concrete type SFRC and the longitudinal reinforcement hybridized with 50% CFRP bars .The first crack in columns appeared at load (195 and 117 kN), respectively. The failure happened on the compression side gradually at load (507 and 390 kN), respectively. Compared with reference specimens (C_5 and C_6) there is s decrease in the cracking load about (40 and 49.57%), respectively, also, there is a decrease in the ultimate load capacity about (3.75 and 0%) and the ductility was increased (1.04 and 5.43%), respectively. Figure 8a and 9a shows the load deflection response of the column specimens C_7 and C_8, respectively. Figure 8b and 9b shows the crack pattern of the specimens C_7 and C_8, respectively.

Column specimens with construction joints

Specimens C_{10}-H_1-H_2-E_1 and C_{11}-H_1-H_2-E_2: In these specimens, the outer shell of the columns were made with concrete type H_1 and presence of construction joint between the core and the shell with interface type (surface intentionally roughened). The first crack in columns appeared at load (292 and 156 kN), respectively. The spalling happened on the compression side suddenly at load (604 and 487kN), respectively. Compared with reference specimens (C_3 and C_4) there is an increase in the cracking load about (49.74 and 14.70%), respectively also, there is an increase in the ultimate load capacity about (14.82 and 14.59%) and the ductility was decreased (1.14 1.80%), respectively. Figure 10a and 11a shows the load deflection response of the column specimens C_{10} and C_{11}, respectively. Figure 10b and 11b shows the crack pattern of the specimens C_{10} and C_{11}, respectively.

Specimens C_{12}-H_1-S-J-E_1 and C_{13}-H_1-S-J-E_2: In these specimens, the outer shell of the columns were made with concrete type SFRC and presence of construction joint between the core and the shell with interface type (surface intentionally roughened). The first crack in columns appeared at load (312 and 195 kN), respectively. The spalling happened on the compression side gradually at load (565 and 409 kN), respectively. Compared with reference specimens (C_5 and C_6) there is an increase in the cracking load about (129.41 and 66.66%), respectively also, there is an increase in the ultimate load capacity about (11.44 and 16.52%) and the ductility was decreased (6.25-2.17%), respectively. Figure 10a and 11a show the load deflection response of the column specimens C_{12} and C_{13}, respectively. Figure 10b and 11b show the crack pattern of the specimens C_{12} and C_{13}, respectively.

Specimens C_{14}-H_1-H_2-J_c-E_1 and C_{15}-H_1-H_2-J_c-E_2: These specimen are the same of the specimens C_{10} and C_{11} with presence of shear connectors across the construction joint. The first crack in columns appeared at load (156 and 136 kN), respectively. The spalling happened on the side suddenly at load compression (624 and

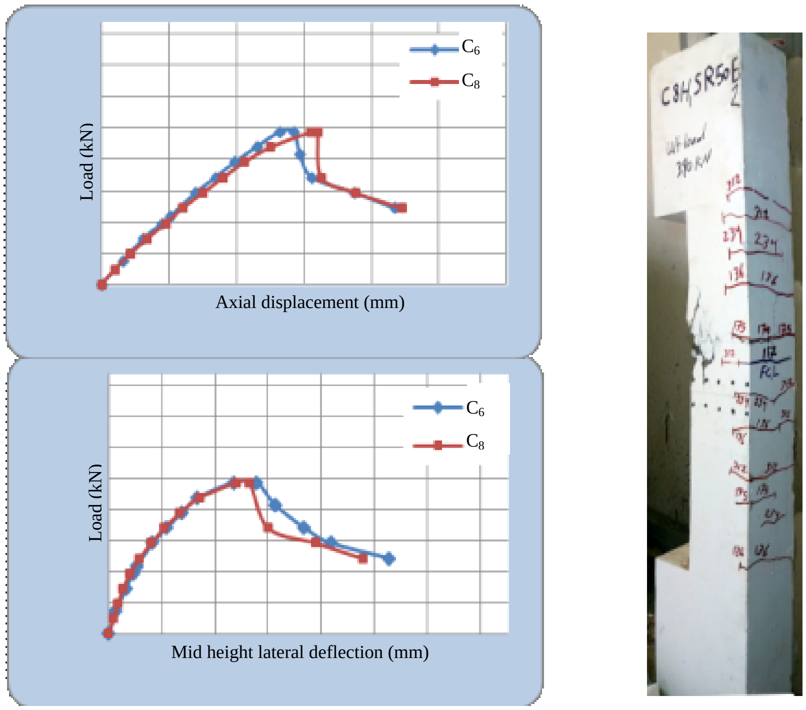

Fig. 9: a) Load-deflection of specimen C_8 and b) Crack pattern of C_8

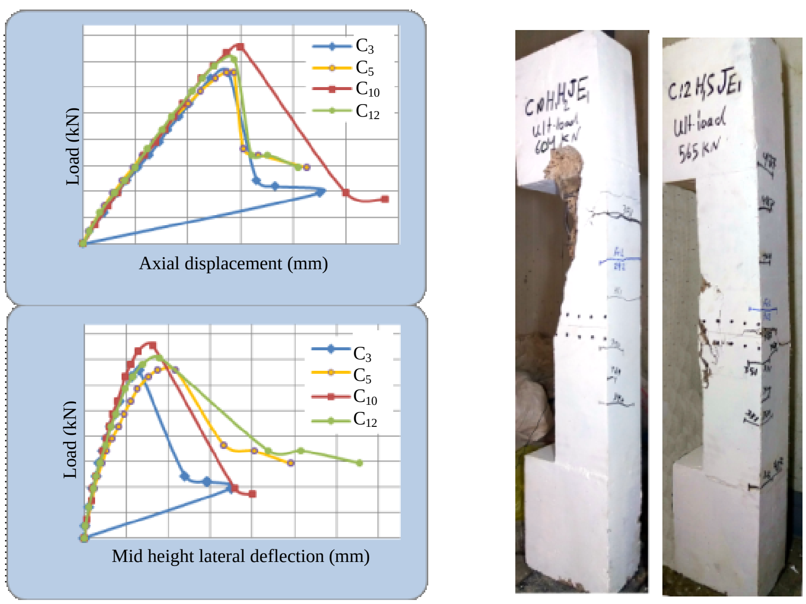

Fig. 10: a) Load-deflection of specimen C_{10} and C_{12} and b) Crack pattern of C_{10} and C_{12}

507 kN), respectively. Compared with reference specimens (C_{10} and C_{11}) there is a decrease in the cracking load about (87.18 and 14.70%), respectively, also, there is an increase in the ultimate load capacity about (23.10 and 44.44%) and the ductility was increased (5.78-11.65%), respectively. Figure 12a and 13a show the load deflection response of the column specimens C_{14} and C_{15}, respectively. Figure 12b and 13b shows the crack pattern of the specimens C_{14} and C_{15}, respectively.

Specimens C_{16}-H_1-S-J-E and C -H -S-J -E: These specimen are the same of the specimens C_{12} and C_{13} with

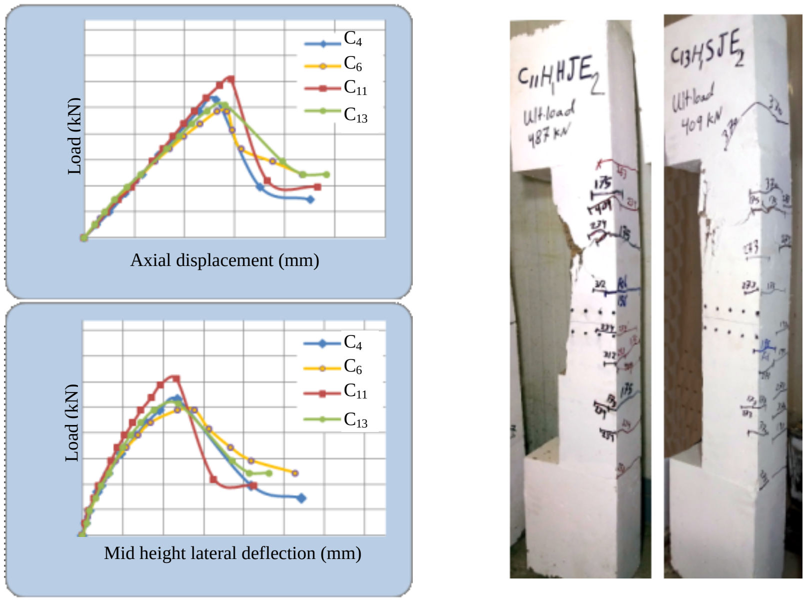

Fig. 11: a) Load-deflection of specimen C_{11} and C_{13} and b) Crack pattern of C_{11} and C_{13}

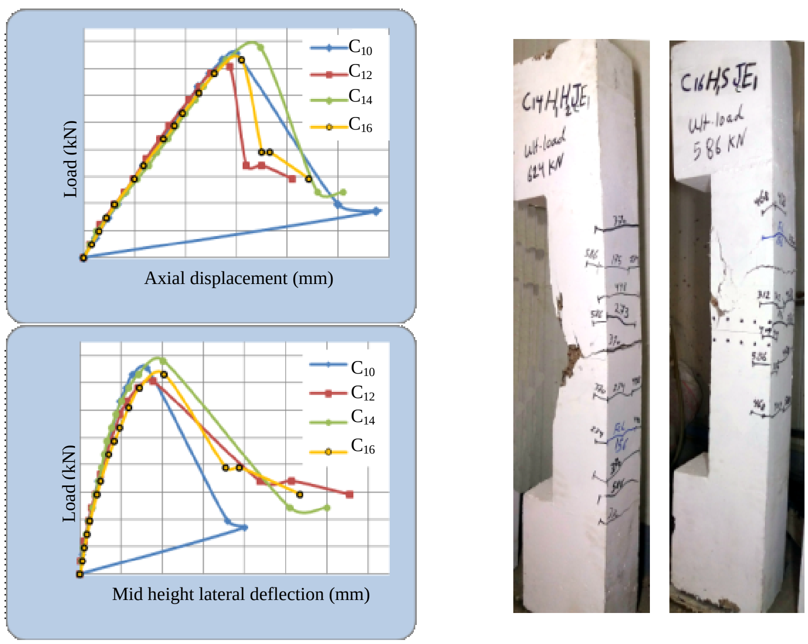

Fig. 12: a) Load-deflection of specimen C_{14} and C_{16} and b) Crack pattern of C_{14} and C_{16}

presence of shear connectors across the construction joint. The first crack in columns appeared at load (156 and 117 kN), respectively. The spalling happened on the compression side gradually at load (586 and 429 kN), respectively. Compared with reference specimens (C_{12} and C_{13}) there is a decrease in the cracking load about (100 and 66.66%), respectively, also, there is an increase in the ultimate load capacity about (3.71 and 4.89%) and the ductility was increased (1.66, 3.89%), respectively. Figure 12a and 13a show the load deflection response of the column specimens C_{16} and C_{17}, respectively. Figure 12b and 13b show the crack pattern of the specimens C_{16} and C_{17}, respectively.

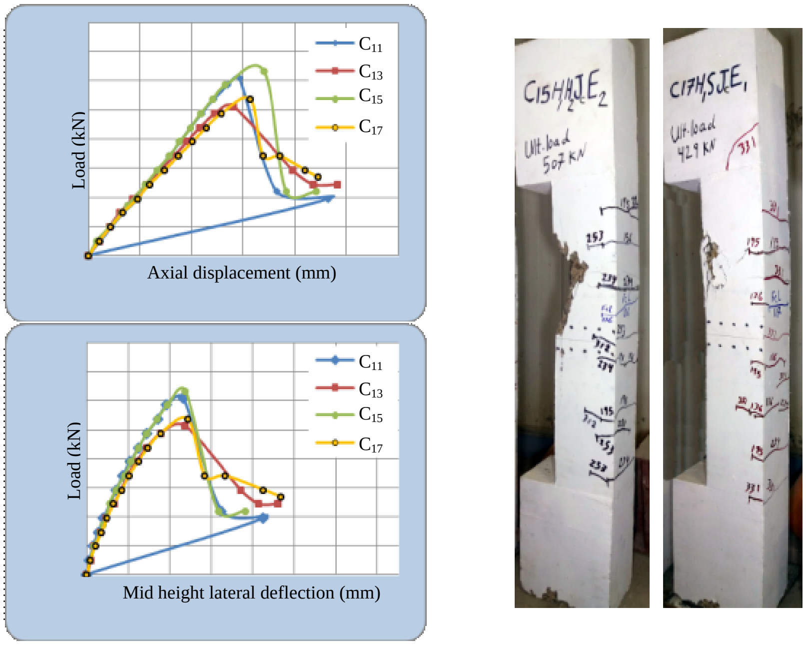

Fig. 13: a) Load-deflection of specimen C_{15} and C_{17} and b) Crack pattern of C_{15} and C_{17}

CONCLUSION

For the column specimens hybridized with concrete type H_2 in the outer shell when the (e/h) values [(1/3) and (1/2)], the ultimate strength was increased (3.74 and 21.08%). On the other hand, the cracking load was increased (43.38 and 16.32%), respectively.

When the (e/h) values [(1/3 and (1/2)], the presence of SFRC in the outer shell was improved in the load carrying capacity about (3.74 and 11.11%) and there was an enhancement the cracking loads about (100 and 50%), respectively.

For the column specimens hybridized with 50% of CFRP bars as longitudinal reinforcement when the (e/h) values [(1/3 and (1/2)], the ultimate strength was decreased about (3.75 and 0%). On the other hand, the cracking load was decreased (40 and 49.57%), respectively.

When the (e/h) values [(1/3 and (1/2)] and comparing with the column specimens with monolithic casting, the presence of construction joint between the concrete of the core and the shell led to increase in the ultimate load about (14.82 and 14.59%) and (7.41 and 4.87%) for column specimens hybridized with concrete type H_2 and SFRC in the outer shell, respectively on the other hand, the cracking load was increased about (49.74 and 14.70%) and (14.28 and 11.42%) for column specimens hybridized with concrete type H_2 and SFRC in the outer shell, respectively.

When the (e/h) values [(1/3 and (1/2)] and comparing with the column specimens without shear connectors, using of shear connectors in the construction joints led to increase in the ultimate load about (3.31 and 4.10%) and (3.71 and 4.89%) for column specimens hybridized with concrete type H_2 and SFRC in the outer shell, respectively on the other hand, the cracking load was decreased about (87.18 and 14.70%) and (100 and 66.66%) for column specimens hybridized with concrete type H_2 and SFRC in the outer shell, respectively.

For all the column specimens, the ductility ratio were calculted when the (e/h) values [(1/3 and (1/2)], the ductility ratio was decreased about (1.13 and 5.68%) for column specimens hybridized with concrete type H_2 in the outer shell but the ductility ratio was increased about (8.74 and 4.54%) for column specimens hybridized with concrete type SFRC in the outer shell, respectively using of CFRP bars (50% hybridization) was increased the ductility ratio about (1.04 and 5.43%), respectively the presence of the construction joint was decreased the ductility about (1.14, 1.80%) for column specimens hybridized with concrete type H_2 in the outer shell and about (6.25 and 2.17%) for column specimens hybridized with concrete type SFRC in the outer shell, respectivily and comparing with the column specimens with monolothic casting, Also, the presence of the shear connectors was increased the ductility about (5.78 and 11.65%) for column specimens hybridized with concrete type H_2 in the outer shell and (1.66 and 3.89%) for column specimens hybridized with concrete type SFRC in the outer shell, respectivily and comparing with the column specimens without shear connectors.

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