

Axial Load-Carrying Capacity of Thin-Walled HSS Stub Columns Filled with Waste Glass Concrete

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Abstract: The main objective of the present study is to investigate the axial load-bearing capacity of stiffened thin-walled Concrete-Filled Tube (CFT) stub columns with using Waste Glass (WG) as partial replacement of fine and coarse aggregate in the filling concrete. Ten hollow and thirteen CFT stub column specimens were manufactured and prepared for testing. Three of the thirteen CFT stub column specimens were filled with WG concrete. Among the total 23 specimens, 15 were provided with longitudinal stiffeners. Different values were used for the rigidity of longitudinal stiffeners, shape factor, aspect ratio, and constraining factor. All the specimens were tested under axial compression up to their maximum strength. The experimental investigations of this study indicated that the longitudinal stiffeners increased the load-carrying capacity of stub columns and showed better performance of stub columns at higher values of D/B and D/t. The confinement of concrete core was improved by using longitudinal stiffeners. The load-carrying capacities of tested CFT stub columns agreed well with the values predicted by using ACI 318-11, BS5400 and EC4 codes and the best agreement was with EC4.

Keywords: Stub columns, Concrete-Filled Tubes (CFT), Waste Glass (WG) concrete.

1. Introduction

Concrete filled tubes (CFTs) offer several constructional and structural advantages over, alone used, reinforced concrete or steel columns. Some of these advantages are enhancement in strength and stiffness, reduction in material, fast construction, excellent earthquake-resistance, good ductility, high fire resistance, and reduced cost of form works [1]. Several researches showed that the axial load-bearing capacity of square and rectangular CFTs is lower than that of circular ones due to the weaker effect of concrete confinement [2, 3, 4]. Studies showed that adequate stiffening measures are highly desirable for square and rectangular CFTs. One of the suggested means to enhance their performance is adding longitudinal stiffeners [5]. Lin-Hai Han, et al. (2005) [6] investigated experimentally the behavior of self-consolidating concrete-filled HSS stub columns subjected to axial load. They concluded that the strength index decreases with the increase of D/t or B/t due to the decrease of constraining factor (ξ). M. Mouli and H. Khelafi (2007) [7] studied the bond behavior and axial capacity of short composite columns of two rectangular hollow steel sections RHSS and found that the bond strength was not influenced by the section of steel tube specimens, but was significantly affected by the type of concrete. Walter Luiz Andrade de Oliveira, et al. (2009) [8] conducted experimental analysis of the confinement effects in steel-concrete composite columns. They varied concrete compressive strength and length/diameter ratio. Results showed that the load capacity increased with increasing concrete strength and decreasing length/diameter ratio. The increasing quantities of WG and the absence of suitable recycling process for collecting these quantities of WG cause serious problems in both environmental and health aspects. Many researchers studied the problem of using waste glass in normal concrete [9, 10, 11,12]. In Iraq however, only few researches were conducted on the use of waste glass in the production of concrete blocks, mortar and concrete [13, 14, 15].

2. Objective of the Study

The objective of this study is to investigate the effect of some parameters on the axial load-bearing capacity of stiffened thin-walled CFT stub columns including the effect of using WG as a partial replacement of fine and coarse aggregate in the filling concrete.

3. Materials and Mix Proportions

3.1 Materials

Cement: The cement used was ordinary Portland cement (Type I) conforming to IQS/5/1984 [16] and manufactured

and tested at Badosh cement plant in Mosul/Iraq with the chemical and physical properties shown in Tables 1 and 2, respectively.

Aggregates: The fine aggregate used was natural sand from Kanhash region with a bulk density of 1743 kg/m³ and fineness modulus of 3.1. Round-shaped river gravel of a maximum aggregate size of 10 mm was used as coarse aggregate. The grading of both fine and coarse aggregates was according to (IQS 45:1984) [17] as shown in Table 3, whereas their properties were determined according to ASTM C128 [18] and ASTM C127 [19], respectively as shown in Table 4.

Glass aggregates: The source of glass aggregates used was the waste of Turkey-made windows glass, collected from local windows glass venders, and then cleaned, crushed and sieved in order to obtain a grading similar to that of natural sand and gravel as given in

Table 3. The particles shape of crushed WG was angular. The properties of WG were determined according to ASTM C128 [18] and are shown in Table 4.

Water: The water used in current study was tap water.

Steel: The properties of the steel used are shown in Table 5. They were obtained from cutting three coupons from the manufactures steel tubes according to ASTM A370-03a [20] and tested up to rupture using a tensile machine of 800 kN capacity.

Table1: Chemical composition of cement

Component	Value	Limits of IQS:5/1984
SiO ₂ (%)	21.38	-----
Insoluble residue (%)	0.27	Max 1.5 %
Al ₂ O ₃ (%)	5.9	-----
Fe ₂ O ₃ (%)	2.4	-----
CaO(%)	62.31	-----
MgO(%)	3.77	Max 5 %
SO ₃ (%)	2.3	Max 2.8 %
Loss of ignition (%)	1.22	Max 4 %
Total	99.28	

Table 2: Physical properties of cement

Property	Value	Limits of IQS:5/1984
Fineness by Blain	2738	Min 2300 (cm ² /g)
Initial setting time	160	Min 45 (minute)
Final setting time	3.67	Max 10 (hr)
Stability	0.14	0.8 (%)
Compressive strength (mortars) 3 days	24.68	Min 15 (MPa)
Compressive strength (mortars) 7 days	33.32	Min 23 (MPa)

Table 3: Sieve analysis of fine and coarse aggregates

Sieve size (mm)	Percentage of passing (%)				Limits of IQS 45:1984 (%) [16]	
	fine	WG fine	coarse	WG coarse	fine	coarse
20	-	100	100	100	-	100
14	-		100	97	-	100
10	-	100	90	93	-	85-100
5	100	99	24	70	100	0-25
2.36	67	83	1	33	65-100	0-5
1.18	58	51	-	14	45-100	-
0.6	49	31	-	4	25-80	-
0.3	12	14	-	2	5-48	-
0.15	1	5	-	1	0-15	-
0.075		1				

Table 4: Physical properties of the aggregates

Type of aggregate	Specific Gravity	Bulk Density (kg/m ³)	Absorption (%)
Fine aggregate	2.6	1743	2.07
Round coarse aggregate	2.67	1687	1
WG	2.5	1564	0.17

Table 5: Mechanical properties of steel

Thickness (mm)	Yield Strength (MPa)	Ultimate Strength (MPa)	Elastic Modulus (GPa)	Elongation Percentage (%)
1	197.7	241.5	215	16

3.2 Mix proportions

The proportions of normal control mix based on the ACI method [21] were (1:1.96:2.73:0.5) (cement: sand: gravel: W/C). The same proportions were used for preparing WG concrete but with replacing 20 and 25% percentages of the natural fine and coarse aggregate by WG aggregates as shown in Table 6. These percentages of replacement were proved optimal (from the strength viewpoint) based on another separate experimental study (being published in the moment of writing this paper).

Table 6: Mix proportions of normal (control) and WG concrete

Mix No.	Description	Cement	Fine Aggregate	Fine WG	Coarse Aggregate	Coarse WG	w/c
1	CM	1	1.96	-----	2.73	-----	0.5
2	20% SR+ 25% GR	1	1.57	0.39	2.05	0.68	0.5

Where: CM= Control Mix; SR= Sand Replacement; GR= Gravel Replacement

4. Experimental Works

4.1 Testing of concrete mixes

Fresh and hardened normal and WG concrete properties were tested. The slump and fresh density tests were conducted according to BS EN 12350-2:2000 [22] and BS EN 12350-6:2000 [23], respectively. The compressive strength tests were conducted at 7 and 28 days according to BS EN 12390-3:2002 [24] and BS EN 12390-1:2000 [25]. The (150×150×150) mm concrete cubes were maintained in water at temperature 23±2°C. The tensile splitting strength tests were conducted at 7 and 28 days ages according to BS EN 12390-6:2000 [26] and BS EN 12390-1:2000 [25]. The (200×100) mm concrete cylinders were cured by placing them in water at temperature 23±2°C.

The flexural strength tests were conducted at 7 and 28 days according to BS EN 12390-5:2000 [27] and BS EN 12390-1:2000 [25]. The (100×100×400) mm concrete prisms were cured the same way as for the compressive and tensile splitting tests.

4.2 Fabricating and preparing of CFT stub column specimens

A total of 23 tubes were manufactured from structural steel sheets with a measured thickness of 1mm. Table 7 shows the geometrical and other properties of the manufactured stub columns. Each steel tube of a square or rectangular section was firstly brake-pressed from the sheets to form three edges open sections. Intermediate stiffeners were welded to each side of the stiffened square hollow section (SHS) column, while only two stiffeners were welded to the longer sides of the stiffened rectangular hollow section (RHS) column.

Finally, a fourth-edge plate was welded to each section.

Table 7: Geometrical properties and experimental ultimate loads of stub columns

No.	Specimen label *	B mm	D mm	L mm	D/t	b _s x t _s mmxmm	I _s mm ⁴	N _{ue} kN
1	SQH100-1	100	100	300	100	15×1	281	65.8
2	UQH100-2	100	100	300	100	-	-	31
3	SQH75-3	75	75	225	75	10×1	83	61.6
4	UQH75-4	75	75	225	75	-	-	38.2
5	SQH50-5	50	50	150	50	7.5×1	35	47.4
6	UQH50-6	50	50	150	50	-	-	36
7	STH100-7	50	100	300	100	15×1	281	52
8	UTH100-8	50	100	300	100	-	-	20.2
9	STH75-9	50	75	225	75	10×1	83	49.6
10	UTH75-10	50	75	225	75	-	-	36.8
11	SQN100-11	100	100	300	100	15×1	281	444.4
12	SQN100-12	100	100	300	100	10×1	83	408
13	UQN100-13	100	100	300	100	-	-	362
14	SQN75-14	75	75	225	75	10×1	83	265.2
15	SQN75-15	75	75	225	75	5×1	10	233.8
16	SQN50-16	50	50	150	50	7.5×1	35	151.7
17	SQN50-17	50	50	150	50	5×1	10	147
18	UQN50-18	50	50	150	50	-	-	113.6
19	STN100-19	50	100	300	100	15×1	281	283
20	STN75-20	50	75	225	75	10×1	83	193.8
21	SQG75-21	75	75	225	75	10×1	83	289.1
22	SQG75-22	75	75	225	75	5×1	10	264
23	UQG75-23	75	75	225	75	-	-	208.4

* S: Stiffened; U: Unstiffened; Q: sQuare; T: recTangular; H: Hollow (unfilled); N: filled with Normal concrete; G: filled with WG concrete.

Prior to concrete casting, the inner surface of steel tubes has been cleaned by brush to remove undesirable materials resulted from cutting the tubes and to remove any loose debris and rust. A total of 10 of the steel tubes (nos. 11-20 in Table 7) were constructed by using normal concrete with control mix proportion (1: 1.96: 2.73, w/c = 0.5). Three tubes (nos. 21-23) have been filled with waste glass concrete according to mix 2 in Table 6. The other 10 stub columns (nos. 1-10) were left empty. The concrete was poured in three layers and vibrated by vibrating table. Normal concrete was provided in two batches and the WG concrete provided in a single batch. From each of the three batches three (150×150×150mm) cubes and three (150×300mm) cylinders were prepared and tested to determine the cube and cylinder compressive strengths of each batch according to BS EN 12390-3:2002 [28] and ASTM C 39/C 39M [29], respectively. All the specimens of CFTs, cubes and cylinders were cured by placing them upright to air-dry until testing (at 28-day age). During curing a very small amount of shrinkage occurred at the top of the columns. This small amount of shrinkage was treated by filling the gap with cement powder.

4.3 Testing of CFT specimens

A 450 KN capacity testing machine was used for the compression tests of all specimens as shown in Figure 1. A cap plate (40 mm thick) was put on the top end of the column specimen to ensure a uniform distribution of the load on steel and concrete. The load was applied in small increments less than 10% of the total expected capacity. Three dial gauges were used in testing each specimen. Two of them were attached at mid height of the column specimen to measure the transverse displacement of column specimen in both orthogonal directions. This was done in order to ensure the load concentricity by keeping the difference of transverse displacements no more than 5%. The other dial gauge was placed vertically to measure longitudinal shortening of the column specimen. Each load increment was maintained for 2-3 minutes approximately and then the reading of axial deformation was recorded. This process continued until the column specimen reached its ultimate capacity.



Figure 1: Test setup of stub columns

5. Results and discussion

Modes of failure of the stub column specimens: The failure in all specimens was buckling of steel plate panels according to the modes shown in Figure 2. In the unstiffened hollow square stub columns, the steel tube exhibited alternately inward and outward buckling with the nodes at the corners of the columns (Figure 2a). In the stiffened square hollow stub columns the local buckling occurred in the steel tube as shown in Figure 2b for all D/t values (100, 75 and 50). In all CFT stub columns the concrete inside the tube prevents inward buckling and the steel tube provides lateral confinement to the concrete inside the tube. The CFT specimens exhibited bulging of steel tubes near the top end of the specimens initially and then the steel plates buckled at different locations including the center part of the specimens. Moreover, the unstiffened CFT stub columns buckled earlier than the stiffened CFT columns.

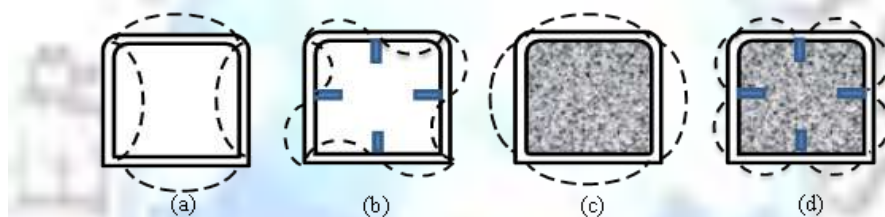
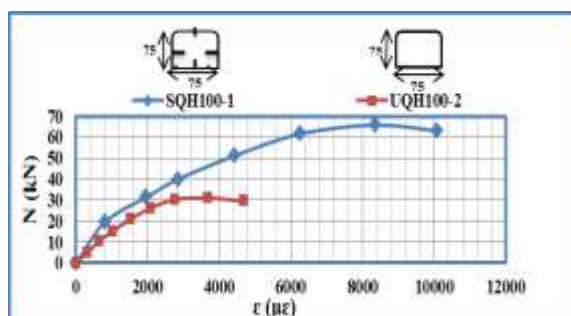
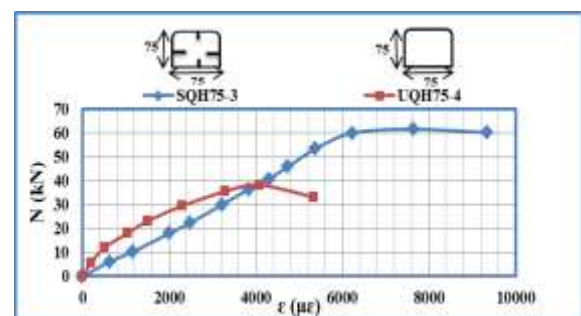


Figure 2: Modes of local buckling in tested specimens

Effect of longitudinal stiffeners on load carrying capacity: The load-strain relationships of stiffened and unstiffened specimens are shown in Figures 3 to 5. From these Figures and from Table 7 it can be noticed that the longitudinal stiffeners increased the load-carrying capacity of both hollowed and filled stub columns. The increase in the ultimate strength of stiffened hollowed specimens SQH100-1, SQH75-3, SQH50-5, STH100-7 and STH75-9 compared with their unstiffened counterparts UQH100-2, UQH75-4, UQH50-6, UTH100-8 and UTH75-10 were 112%, 61%, 32%, 157% and 35%, respectively. The effect of longitudinal stiffeners is higher when used in columns with higher values of D/B and D/t. In the case of stub columns filled with normal or WG concrete a better performance of longitudinal stiffeners is noticed. The increase in the ultimate strength of stiffened specimens filled with normal concrete and WG concrete SQN100-11, SQN50-16, SQG75-21 were 82.4, 38.1 and 80.7 kN, respectively. This is explained by the fact that the longitudinal stiffeners in filled stub columns improve the lateral confinement of concrete core as well as they delay the local buckling of the steel plate of the filled specimens, thus enhance their strength. Also, it can be noted from the experimental results that the higher the rigidity of longitudinal stiffeners (I_s), the higher the increase in the ultimate strength of stub columns. For example, the increase of load in CFT stub columns SQH100-11 and SQH100-12 were 82.4 and 46 kN, respectively.



(a)



(b)

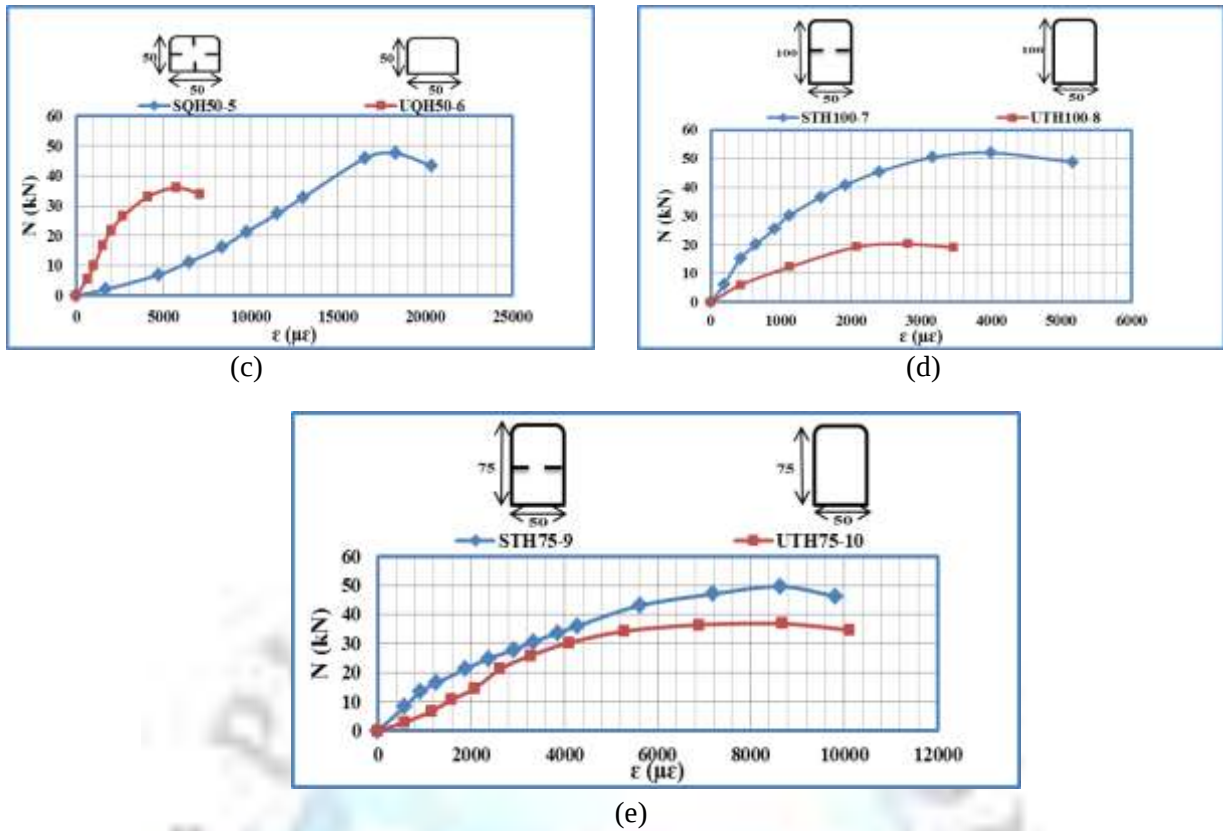


Figure 3: Load -strain relationship of hollowed test specimens.

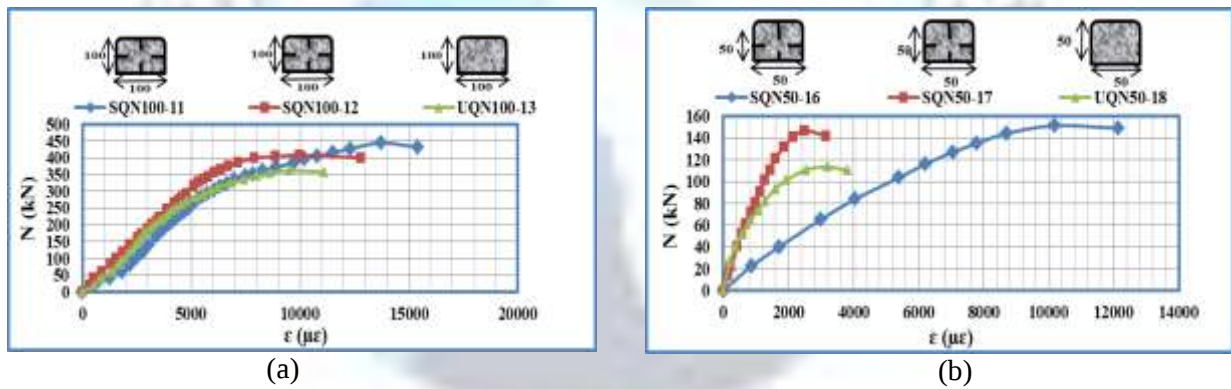


Figure 4: Load-strain relationship of test specimens filled with normal concrete.

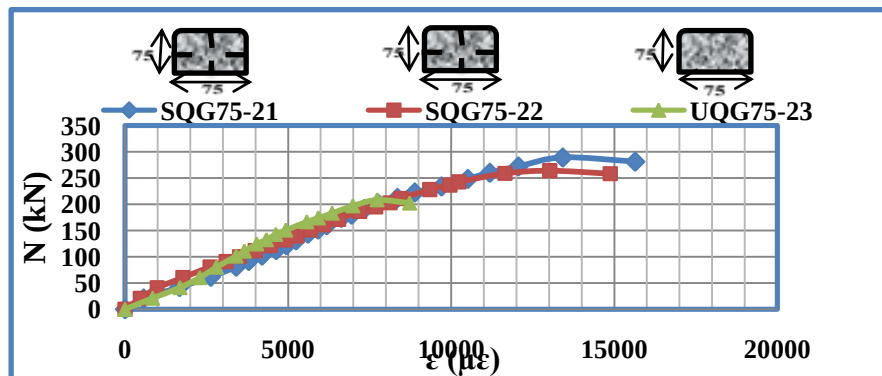


Figure 5: Load-strain relationship of test specimens filled with WG concrete.

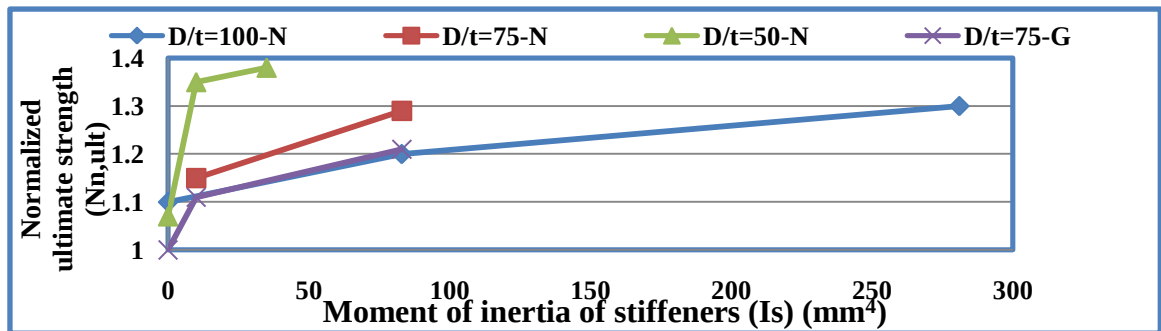


Figure 6: Effect of stiffeners rigidity on normalized ultimate strength

Effect of stiffeners rigidity on normalized ultimate strength: The equation below represents the ratio between the ultimate experimental strength of a given stub column, after subtracting the contribution of longitudinal stiffeners, to the contributions of concrete core and steel tube [30].

$$N_{n,ult} = \frac{N_{ue} - f_{y,s} A_{s,s}}{f_c A_c + f_{y,t} A_{s,t}} \quad \dots\dots\dots (1.1)$$

Where:

- N_{ue} : Experimental ultimate strength,
- $f_{y,s}$: Yield strength of the stiffener,
- $A_{s,s}$: Cross-sectional area of steel stiffeners,
- f_c : Characteristic compressive concrete strength ($=0.4 f_{cu}^{7/6}$),
- A_c : Cross-sectional area of concrete,
- $f_{y,t}$: Yield strength of the steel tube, and
- $A_{s,t}$: Cross-sectional area of steel tube.

Figure 6 illustrates the effect of stiffener rigidity on the ultimate normalized strength. It can be observed from the Figure that the normalized ultimate strength ($N_{n,ult}$) of specimens with $D/t=100$ and $D/t=75$ increased with increasing the rigidity I_s . Furthermore, the normalized strength ($N_{n,ult}$) didn't increase significantly with increasing the stiffener rigidity for the case $D/t=50$.

Effect of confinement on ultimate strength capacity: The effect of confinement on the ultimate strength capacity can be illustrated by computing the expected ultimate strength (N_{exp}) and comparing it with the ultimate strength from experimental results (N_{ue}). The expected strength is defined:

$$N_{exp} = N_{ue(Hollow)} + A_c f_{cu} \quad \dots\dots\dots (1.2)$$

The above equation represents the sum of experimental ultimate strength of the hollowed specimen and that of the concrete (area of concrete multiplied by its cubic compressive strength (f_{cu})). The experimental results of ultimate bearing capacity of stiffened composite stub columns, Which are shown in Table 8 and Figure 7, showed better behavior under axial compression compared to the expected results (except for specimen SQG75-21). This is due to the effect of stiffeners in confining the concrete core and the contribution of the last in delaying the local buckling of steel tube and hence enhancing the total ultimate strength capacity. The percentages of increase of stiffened specimens SQN100-11, SQN75-14, SQN50-16, STN100-19 and STN75-20 were 7.6, 4.4, 10.3, 19.4 and 3.3%, respectively. Whereas the experimental results of unstiffened composite stub columns showed values lower than the expected results due to the absence of confinement in these specimens.

Table 8: Expected and experimental ultimate strength

No.	Specimen label	$b_s \times t_s$ mm	f_{cu} N/mm ²	A_c mm ²	N_{ue} (Hollow) kN	N_{exp} (expected) kN	N_{ue} (experimental) kN
1	SQN100-11	15×1	36.4	9544	65.8	413.2	444.4
2	UQN100-13	-	36.4	9604	31	380.6	362
3	SQN75-14	10×1	36.4	5289	61.6	254.1	265.2
4	SQN50-16	7.5×1	39.6	2274	47.4	137.5	151.7
5	UQN50-18	-	39.6	2304	36	127.2	113.6
6	STN100-19	15×1	39.6	4674	52	237.1	283
7	STN75-20	10×1	39.6	3484	49.6	187.6	193.8

8	SQG75-21	10×1	43.8	5289	61.6	293.3	289.1
9	UQG75-23	-	43.8	5329	38.2	271.6	208.4

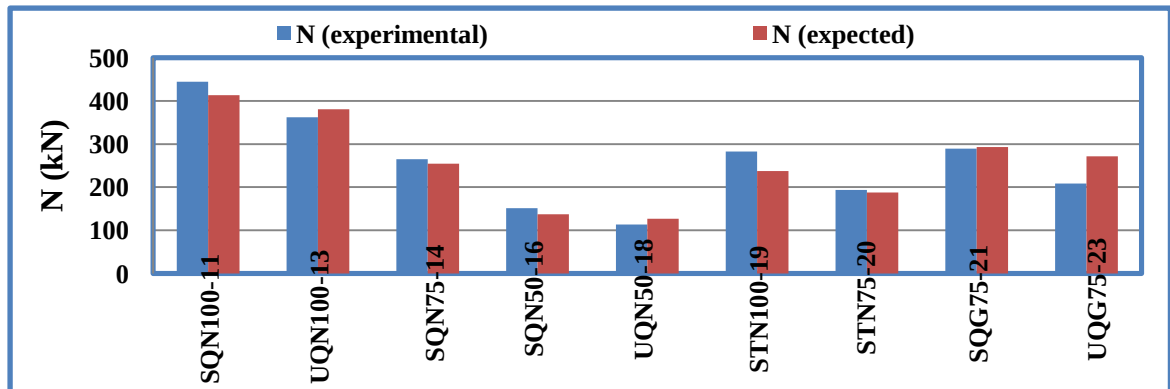


Figure 7: Comparison between expected and experimental ultimate strength

Effect of constraining factor on strength index: Equations (1.3) and (1.4) [31] were used to calculate the constraining factor (ξ) and the strength index (SI) of the composite stub columns used.

$$\xi = \frac{A_s f_y}{A_c f_{ck}} \quad \text{..... (1.3)}$$

$$SI = \frac{N_{ue}}{A_s f_y + A_c f_{ck}} \quad \text{..... (1.4)}$$

Where $A_s f_y$ for stiffened composite stub column is replaced by $(A_{s,t} f_{y,t} + A_{s,s} f_{y,s})$; f_{ck} : the average compression strength of cubic specimens of concrete (f_{cu}) multiplied by 0.67.

Table 9: Constraining factor and strength index of composite stub columns

No.	Specimen label	$b_s \times t_s$ mm×mm	$A_{s,t}$ mm ²	$A_{s,s}$ mm ²	A_c mm ²	f_y N/mm ²	f_{cu} N/mm ²	ξ	SI
1	SQN100-11	15×1	396	60	9544	197.7	36.4	0.39	1.37
2	SQN100-12	10×1	396	40	9564	197.7	36.4	0.37	1.28
3	UQN100-13	-	396	-	9604	197.7	36.4	0.33	1.16
4	SQN75-14	10×1	296	40	5289	197.7	36.4	0.51	1.36
5	SQN75-15	5×1	296	20	5309	197.7	36.4	0.48	1.22
6	SQN50-16	7.5×1	196	30	2274	197.7	39.6	0.74	1.44
7	SQN50-17	5×1	196	20	2284	197.7	39.6	0.7	1.42
8	UQN50-18	-	196	-	2304	197.7	39.6	0.63	1.14
9	STN100-19	15×1	296	30	4674	197.7	39.6	0.52	1.5
10	STN75-20	10×1	246	20	3484	197.7	39.6	0.57	1.34
11	SQG75-21	10×1	296	40	5289	197.7	43.8	0.43	1.3
12	SQG75-22	5×1	296	20	5309	197.7	43.8	0.4	1.21
13	UQG75-23	-	296	-	5329	197.7	43.8	0.37	0.97

From Table 9 and Figure 8 it is clear that the strength index increased with increasing the constraining factor which describes the interaction between steel tube and concrete core. Also it can be seen from Table 9 that decreasing the D/t of square stub columns filled with normal concrete increased the constraining factor due to the better effect of steel tubes of smaller D/t in resisting the pressure resultant from concrete core, and hence increasing strength index. For example, compare between specimens SQN100-12 & SQN75-14, SQN75-15 & SQN50-17, UQN100-13 & UQN50-18, for which the strength indexes were 1.28 & 1.36, 1.22 & 1.42, 1.16 & 1.14, respectively. The constraining factor for specimens filled with WG is less than that for those of the same properties but filled with normal concrete. This is because the compression strength of WG concrete core is higher than the compressive strength of normal concrete and thus larger pressure is applied on steel tubes confining WG concrete and this promotes the local buckling and causes decrease in the constraining factor and strength index. The constraining factor of specimens SQN75-14 and SQN75-15 were 1.36 and 1.22, respectively, and for the specimens SQG75-21 and SQG75-22 are 1.3 and 1.21.

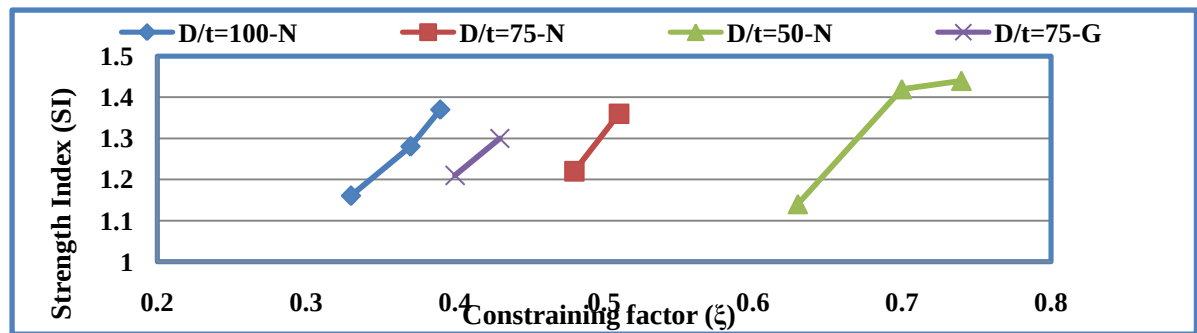


Figure 8: Relationship between strength index and constraining factor

Load carrying capacity of CFT according to different codes: The predictions of axial strengths of square and rectangular CFTs as per various codes and design equations are presented in this section and compared with the experimental values obtained in this study. The ultimate axial strengths of square and rectangular CFTs stub columns as per ACI 318-2011 [32], BS5400 [33], and EC4 [34] were calculated using equations 1.5, 1.6, and 1.7, respectively. These values were compared with experimental values as shown in Table 11.

$$N_{ACI} = A_s f_y + 0.85 A_c f_c \quad (1.5)$$

$$N_{BS5400} = A_s f_y + 0.675 A_c f_{cu} \quad (1.6)$$

$$N_{EC4} = A_s f_y + A_c f_c \quad (1.7)$$

Where:

f_c : Characteristic compressive cylinder strength of concrete, and
 f_{cu} : Characteristic cube strength of concrete.

From Table 11 it is found that the predictions of ultimate load of CFT stub columns by using ACI equation (1.5), BS5400 equation (1.6) and EC4 equation (1.7) agrees well with experimental results. In general the predicted values are conservative compared to the experimental strengths. It can be seen that the predicted results from EC4 are less conservative than ACI 318-2011 and BS5400 and agree better with the experimental results of stiffened stub columns filled with normal and WG concrete.

Table 11: Experimental and predicted ultimate strength

No.	Specimen label	A_{st} mm ²	A_{ss} mm ²	A_c mm ²	f_y N/mm ²	f_c N/mm ²	f_{cu} N/mm ²	N_{ue} kN	N_{ACI} / N_{ue}	N_{BS5400} / N_{ue}	N_{EC4} / N_{ue}
1	SQN100-11	396	60	9544	197.7	30	36.4	444.4	0.75	0.73	0.85
2	SQN100-12	396	40	9564	197.7	30	36.4	408	0.81	0.79	0.91
3	SQN75-14	296	40	5289	197.7	30	36.4	265.2	0.76	0.74	0.85
4	SQN75-15	296	20	5309	197.7	30	36.4	233.8	0.85	0.83	0.95
5	SQN50-16	196	30	2274	197.7	31.5	39.6	151.7	0.71	0.7	0.77
6	SQN50-17	196	20	2284	197.7	31.5	39.6	147	0.71	0.71	0.78
7	STN100-19	296	30	4674	197.7	31.5	39.6	283	0.67	0.67	0.75
8	STN75-20	246	20	3484	197.7	31.5	39.6	193.8	0.75	0.75	0.84
9	SQG75-21	296	40	5289	197.7	34.7	43.8	289.1	0.77	0.77	0.86
10	SQG75-22	296	20	5309	197.7	34.7	43.8	264	0.83	0.83	0.93

5. CONCLUSIONS

From the experimental results of this investigation, the following conclusions can be drawn:

- The longitudinal stiffeners increased the load-carrying capacity of stub columns in both cases (hollowed and filled). A better performance of the longitudinal stiffeners was noticed in stub columns with higher values of shape factor D/B and aspect ratio D/t.

- The stub columns with larger aspect ratio D/t , required higher rigidity of stiffeners.
- In general, the experimental results of stiffened stub columns showed higher ultimate bearing capacity compared to the expected results. This is due to the effect of stiffeners in improving the confinement of the concrete core and delaying the local buckling of steel tube and hence enhancing the load-carrying capacity.
- Decreasing the D/t of square stub columns filled with normal concrete increased the constraining factor (ξ) due to the better effect of steel tubes of smaller D/t in resisting the pressure resultant from concrete core, and hence increasing strength index (SI).
- The constraining factor of CFT stub columns filled with WG was less than the constraining factor of CFT stub columns of the same properties but filled with normal concrete. This is because the compression strength of WG concrete core is higher than the compression strength of normal concrete and thus larger pressure is applied on steel tubes confining WG concrete and this promotes the local buckling and causes decrease in the constraining factor and strength index.
- The load-carrying capacities of the CFT stub columns obtained from experimental tests agreed well with the values predicted by using ACI 318-11, BS5400 and EC4 codes and the best agreement was with EC4.

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