International Journal of Engineering and Advanced Technology (IJEAT)
ISSN: 2249-8958 (Online), Volume-9 Issue-2, December, 2019

Load-Carrying Capacity of Short Concrete Columns Reinforced with Glass Fiber Reinforced Polymer Bars Under Concentric Axial Load

Nguyen Phan Duy, Vu Ngoc Anh, Nguyen Minh Tuan Anh, Polikutin Aleksei Eduardovich

Abstract: ***In this paper, 1 group of plain concrete square columns 150×150×600 mm and 11 groups of concrete columns reinforced with glass fiber reinforced polymer (GFRP) were cast and tested, each group contains of 3 specimens. These experiments investigated effect of the main reinforcement ratio, stirrup spacing and contribution of longitudinal GFRP bars on the load carrying capacity of GFRP reinforced concrete (RC) columns. Based on the experiment results, the relationship between load-capacity and reinforcement ratio and the plot of contribution of longitudinal GFRP bars to load-capacity versus the reinforcement ratio were built and analyzed. By increasing the reinforcement ratio from 0.36% to 3.24%, the average ultimate strain in columns at maximum load increases from 2.64% to 75.6% and the load-carrying capacity of GFRP RC columns increases from 3.4% to 25.7% in comparison with the average values of plain concrete columns. Within the investigated range of reinforcement ratio, the longitudinal GFRP bars contributed about 0.72%-6.71% of the ultimate load-carrying capacity of the GFRP RC columns. Meanwhile, with the same configuration of reinforcement, contribution of GFRP bars to load-carrying capacity of GFRP RC columns decreases when increasing the concrete strength. The influence of tie spacing on load-carrying capacity of reinforced columns was also taken into consideration. Additionally, experimental results allow us to propose some modifications on the existing formulas to determine the bearing capacity of the GFRP RC column according to the compressive strength of concrete and GFRP bars.***

Keywords: ***Reinforced concrete, Short column, GFRP, Concentric load.***

Revised Manuscript Received on December 30, 2019.

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I. INTRODUCTION

Corrosion of reinforced concrete (RC) structure, especially corrosion of steel is one of the urgent problems in construction. In corrosive environments, the fiber reinforced polymer (FRP) bar is considered a good substitute for steel reinforcement. Compared to steel bar, FRP is a material with many advantages such as corrosion resistance, high tensile strength, light weight, low electrical conductivity. Besides the advantages, it also has certain disadvantages, one of those is low compressive strength and compressive modulus of elasticity. So, almost previous studies have mainly concentrated on the flexural and shear behavior of FRP RC concrete members. Today, these members have been widely used for construction works, especially those in corrosive and special environments [6]. For design of FRP RC flexural structures, some countries have developed the design standards, such as ACI 440.1R-06.2006 [1], CAN/CSA-S806-02 [3], SP 295.1325800.2017 [18] etc. Therefore, the study of FRP RC has received great attention from scientists in recent years. Many authors have conducted researches on FRP RC under concentrically axial load. The direction of previous studies mainly focused on: evaluating the effect of longitudinal FRP reinforcement on bearing capacity of columns; failure modes; the influence of configuration of FRP transverse reinforcement on the behavior of columns and developing formulas to determine the load-carrying capacity etc. In the previous researches by Alsayed et al. [2], Luca et al. [4], Ehab M. Lotfy [11], Tobbi et al. [22], Mohammad Z. Afifi et al. [14], Mohamed et al [13], Richa Pateriya1 et al. [17], Muhammad N. S. et al. [15], Jianwei Tu et al. [23], it was showed that, when replacing the longitudinal steel bars with the FRP bars by the same amount, the axial load-carrying capacity of FRP concentrically RC columns decreases by 13%–16%. In FRP RC columns, FRP bars contribute about 3%–10% of the total load-carrying capacity. The increase of main FRP reinforcement ratio boosts the ductility of cross section which has a significant effect on ultimate strain (to 19%) and ultimate loads (to 22%) of columns. Study results of Ehab M. Lotfy [11] indicated that, with the main reinforcement ratio up to 1.7%, load-carrying capacity and reinforcement ratio follows a linear trend.

Failure modes of axially loaded FRP RC columns mainly depend on the configurations of longitudinal and transverse reinforcements. However, due to low compressive elastic modulus of FRP bar, column failure occurs mainly by buckling [2]. Research results by Luca et al. [4] showed that, with the same volumetric ratio FRP bars have lower confinement to concrete than steel bars. Reducing tie spacing or using special type of stirrups like spirals can prevent buckling failure, strengthen the confinement to the longitudinal bars and core concrete, thereby, increase ductility and load-carrying capacity of FRP RC columns [13], [14], [21].

Based on the formula for traditional steel RC column, many authors suggest formulas to predict load-carrying capacity of short concentric GFRP RC columns P_u :

$$P_u = P_c + P_F \tag{1}$$

where: P_c - bearing capacity of concrete section, this value is defined as for the traditional RC column, i.e. $P_c = 0.85 f_c' \left(A_g - A_s\right)$; P_F - bearing capacity of GFRP bars. Muhammad N. S. et al. [15] proposes to determine value P_F based on the concrete ultimate strain of 0.003, accordingly $P_F = 0.003 E_f A_f$. Mohammad Z. Afifi et al. [14] made a similar suggestion, but took the concrete ultimate strain value of 0.002. Tobbi et al. [22] suggested considering the contribution of GFRP bars in compression to be equal to 35% of GFRP tensile strength, i.e. $P_F = 0.35 f_y A_f$.

Ehab M. Lotfy [11] developed a general formula to predict axial load capacity N of the GFRP RC columns:

$$N = 0.4 f_{cu} A_c + 0.75 f_y A_{sc} \tag{2}$$

where: f_y - yield strength of GFRP; A_{sc} - cross section area of main reinforcement; f_{cu} - ultimate compressive strength of the concrete; A_c - cross section area of concrete.

Muhammad N.S. Hadi et al. [8] suggested following formula in which the confinement effect was taken into account:

$$P_{specimen} = f_{fb} A_{bar} + f_{c,core} A_{c,core} + f_{c,cover} A_{c,cover} \tag{3}$$

Despite the availability of a considerable amount of experimental data, there are still some gaps in assessing the behavior of GFRP RC columns. Firstly, the relationship between reinforcement ratio and load-carrying capacity has just been evaluated within a narrow range of reinforcement ratio (from 0% to 1.7%). Secondly, contribution of GFRP bars to load-carrying capacity of GFRP RC columns has not been sufficiently studied with limited concrete grades and reinforcement ratios. This article fills the above mentioned gaps by assessing the behavior of GFRP RC columns with a wide range of GFRP reinforcement ratio up to 3.24% and evaluating contribution of GFRP bars to load-carrying capacity of these columns within the same range of reinforcement ratios and varied concrete grades.

II. MATERIALS & EXPERIMENTAL PROCEDURES

A. Materials

Concrete

In this study, we use 2 concrete mixes to clarify the effect of longitudinal GFRP bars on the load-bearing capacity of GFRP RC columns corresponding to concrete grades (Table-I). The concrete mixtures consist of fine and medium sands, 10 mm and 20 mm coarse aggregates and water. All used aggregates meet current Vietnamese Standards [12], [19], [19]. Cubic strength of concretes R will be determined experimentally on cubes 150 mm×150 mm×150 mm, cylindrical and prismatic compressive strength (f_c' and R_b) are determined empirically through the cubic strength: $f_c' = 0{,}8R$ [23] and $R_b = \left(0{,}77 - 0{,}001R\right)R$ [20].

Table- I: Concrete mixture for testing specimens

Symbol	Cement PCB40, kg	Sand, m³	Gravel, m³	Water, lit
M1[a]	293	0.466	0.847	195
M2[b]	390	0.427	0.829	195

[a] Obtained according to concrete mix B25 (Vietnamese standard [20])
[b] Obtained according to concrete mix B35 (Vietnamese standard [20])

GFRP bars for longitudinal and transverse reinforcement

All GFRP bars used in experiment are provided by FRP VIETNAM., JSC [7]. In order to increase the bond strength between the GFRP bars and the concrete, the surface of the GFRP were made with transverse spiral grooves. Stirrups were made from GFRP-6 bar. For longitudinal reinforcement we use GFRP-6, GFRP-8, GFRP-10, GFRP-12 and GFRP-14. Technical specifications for GFRP bars are shown in Table- II. The tensile stress-strain diagram of the GFRP bar is illustrated in **Fig. 1** [10].

Table- II: Technical specifications for GFRP [7]

Bar	Outside diameter, mm	Cross section area, mm²	Tensile strength, MPa	Modulus of elasticity, *MPa*	Strain at peak stress, %	Adhesion stress, *MPa*
GFRP-6	6±0.5	19.62±8%	970	44300	1-3	12
GFRP-8	8±0.5	33.16±8%				
GFRP-10	10±0.5	56.71±8%				
GFRP-12	12±0.5	86.54±8%				
GFRP-14	14±0.5	122.65±8%				

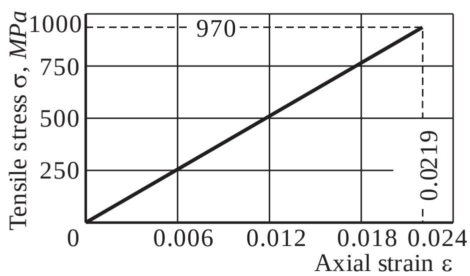


Fig. 1. Tensile stress-strain diagram of GFRP bars [10]

The compression test of GFRP bar is a complex issue and has also been of interest to many scientists. Wu Wei-Pin [24] reported that, the compressive strength of GFRP was about 45% of tensile strength. Similarly, Kobayashi and Fujisaki [9] concluded the compressive strength of the GFRP bars varied from 30% to 40% of their corresponding tensile strength. Deitz et al. [5] tested forty-five GFRP bars #15 in compression and pointed out that, GFRP bar failed by crushing, the ultimate compressive strength was approximately 50% of the ultimate tensile strength and Young's modulus in compression was the same as in tension. Qasim S. Khan et al. [16] tested GFRP (D15.9 mm, 80 mm long) in compression according to ASTM D695-10 with some modifications. The experimental outcomes revealed that, the ultimate strength and modulus of elasticity of GFRP bars in tension are 1.67 and 1.59 times greater than in compression respectively, and the compressive stress-strain relationship follows a linear trend until failure. Based on the findings of Qasim S. Khan and the results of tensile test of GFRP bar as indicated in **Fig. 1**, we determine the compressive strength and elasticity modulus in compression of current GFRP bars $R_{fc} = 581$ MPa and $E_{fc} = 27930$ MPa respectively and compressive stress-strain diagram has linear type.

B. Configuration of specimens

To achieve the research objectives, we manufactured one group of plain concrete columns and 10 groups of short concrete columns longitudinally and transversely reinforced with GFRP bars; each group consists of three identical columns in order to increase the accuracy of the test (). Groups differ in the reinforcement ratio the tie spacing and the concrete grade. The total area of longitudinal reinforcement A_f varies from 82.4 mm^2 to 728.0 mm^2, the reinforcement ratio μ_f correspondingly varies from 0.37% to 3.24%. Tie spacings are 50 mm; 100 mm and 200 mm. Groups of columns from #2 to #8 have the same concrete mix and configuration of stirrups but differ in reinforcement ratio; these columns together with the group #1 are used to study the load-carrying capacity and longitudinal reinforcement ratio relationship. Groups of columns # 5, #9 and #10 differ in tie spacing; these specimens are used to investigate the effect of tie spacing to the load-bearing capacity of the GFRP RC columns. Group of columns #7 and #11 differ only in concrete mix, these groups of columns are used to evaluate the contribution of the GFRP bars to the bearing capacity of the columns with different grades of concrete.

Table- III: Test matrix

Group of Columns	Specimen ID[a]	Longitudinal reinforcement			Transverse reinforcement	
		Bars	*A_f, mm^2*	*μ_f, %*	*Bar*	*Spacing*
#1	M1-*i*	-	-	-	-	-
#2	M1-4F6-F6S100-*i*	4F6	82.4	0.37	F6	100 mm
#3	M1-4F8-F6S100-*i*	4F8	139.6	0.62	F6	100 mm
#4	M1-4F10-F6S100-*i*	4F10	238.8	1.06	F6	100 mm
#5	M1-4F12-F6S100-*i*	4F12	364.0	1.62	F6	100 mm
#6	M1-8F10-F6S100-*i*	8F10	477.6	2.12	F6	100 mm
#7	M1-4F14-F6S100-*i*	4F14	516.0	2.29	F6	100 mm
#8	M1-8F12-F6S100-*i*	8F12	728.0	3.24	F6	100 mm
#9	M1-4F12-F6S50-*i*	4F12	364.0	1.62	F6	50 mm
#10	M1-4F12-F6S200-*i*	4F12	364.0	1.62	F6	200 mm
#11	M2-4F14-F6S100-*i*	4F14	516.0	2.29	F6	100 mm

[a] Each specimen is identified with four codes separated by hyphen (-): the first code (M1, M2) refers to concrete mix; the second code (4F6, 4F8 …) identifies the number and diameter of longitudinal GFRP bars; the third code (F6S100, F6S50 ….) denotes the diameter and tie spacing and the last code (*i*=1, 2, 3) shows the order of specimens in group; $\mu_f = A_f/A$; *A* – net cross-sectional area).

The dimensions of tested columns 150 mm×150 mm ×600 mm were chosen in line with the condition and capacity of the available testing facility in the laboratory and to eliminate the effect of friction on the ends of columns. The longitudinal rebars are cut into pieces with a length of 580 mm (protection concrete cover at both ends is 10 mm). The GFRP ties are flexed at factory. The longitudinal GFRP bars are located around the perimeter of the column cross section (4 bars or 8 bars) with the concrete cover of 20 mm. Longitudinal GFRP bars and stirrups are tied with soft steel wire. In order to prevent crushing at the ends of column, thereby ensure failure at the middle of the column, the two ends of the columns are reinforced with steel nets. The reinforcement configurations of the tested columns are shown in **Fig. 2** and **Fig. 3**.

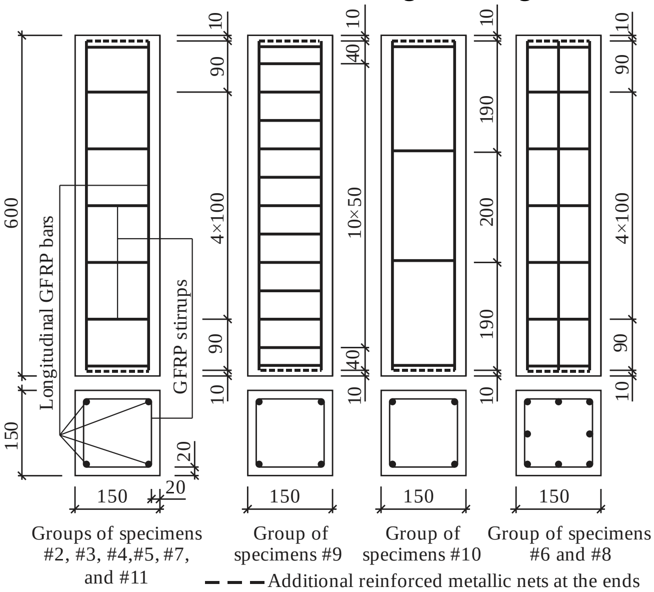


Fig. 2. Details and configuration of the testing columns

Fig. 3. Reinforcement cage for GFRP RC columns

Columns were cast horizontally in steel prism mound. Concrete was vibrated using an electric vibrator to compact and remove air bubbles. In addition to casting columns, we also cast cubes 150 mm×150 mm×150 mm to determine the average cubic compressive strength of concrete R_m. The specimens were covered with wet hessian to maintain the moisture conditions.

After 24 hours from the casting time, remove the formwork and continue curing. The curing process lasted 28 days, then the columns were cleaned, the actual section sizes were re-measured, the top and bottom ends were smoothened before testing.

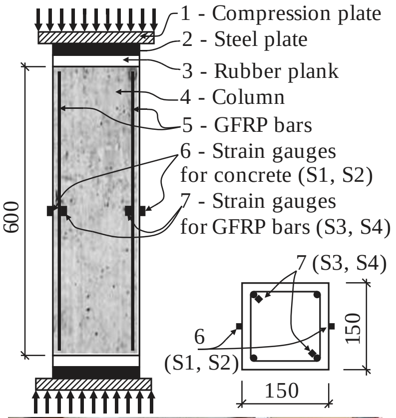
C. Test program and instrumentation

Fig. 4 shows the test setup and instrumentation employed to investigate the compression behavior of the GFRP RC columns. The columns were supported at both ends with two pairs of 8 mm thick steel plate. To fill the gaps between the steel plates and the surfaces of specimens, ensure uniform distribution of the applied load across the cross section and avoid eccentricity during loading we use two rubber planks.

The columns are tested using the universal testing machine with a maximum scale of 100 tons. The axial strains in concrete and longitudinal GFRP bars at the middle of the columns are measured by strain gauges (**Fig. 4**). Data from strain gauges are collected by STS-WIFI system.

Initially, we carry out preliminary loading on the column to the load about 10% of the expected failure load to check eccentricity. If necessary, rotate specimen by 90^0 to eliminate eccentricity. Then the load is continued evenly until the column is destroyed. The loading speed on specimens is controlled according to the average stress in concrete 0.6±0.1 MPa. In addition to the column tests, cubes of 150 mm×150 mm×150 mm are also tested.

a) Schematic diagram



b) Actual

Fig. 4. Test setup and instrumentation

III. TEST RESULTS AND DISCUSSION

A. Strain distribution in concrete and GFRP bars

From the experimental results, we develop graphs showing the relation between axial load and strains in concrete and GFRP bars curves. **Fig. 5** and **Fig. 6** illustrate the curves for two typical GFRP RC columns M1-4F12-F6S100-2 and M1-4F14-F6S100-3. From this figure it can be seen that, in the GFRP RC column, because the adhesion concrete and GFRP bars work together as a whole, so the axial strains in these two materials grows almost similarly.

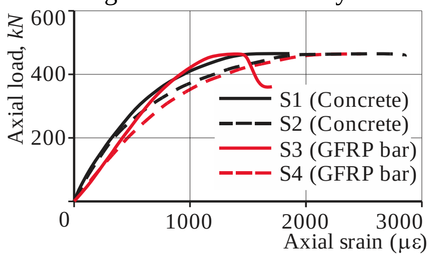


Fig. 5. Axial load versus strains curves for column M1-4F12-F6S100-2

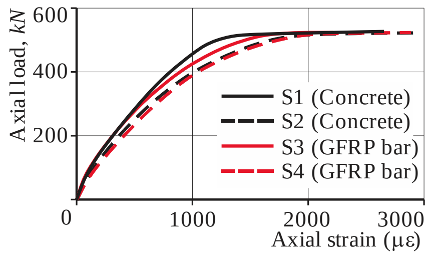


Fig. 6. Axial load versus strains curves for column M1-4F14-F6S100-3

The relative load and average strain curves for GFRP RC columns (groups #2 to #10) and plain concrete columns #1 are presented in **Fig. 7** and **Fig. 8** respectively. Due to the limitations when measuring with a strain gauge, we were only able to obtain strains to the maximum load ε_u (the beginning of the failure stage).

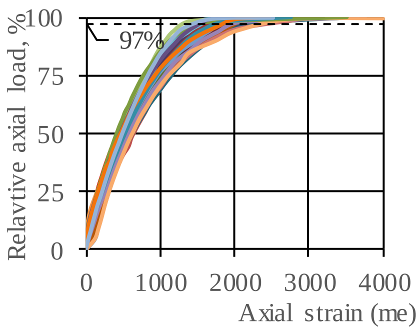
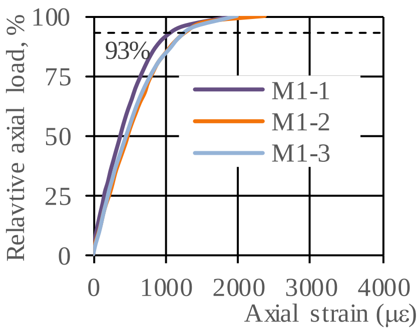


Fig. 7. Relative axial load versus axial strain in GFRP RC columns

Notes about chart legend on Fig. 7:

International Journal of Engineering and Advanced Technology (IJEAT)
ISSN: 2249-8958 (Online), Volume-9 Issue-2, December, 2019



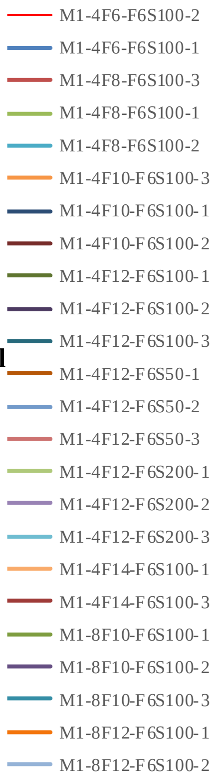


Fig. 8. Relative axial load versus axial strain in plain concrete columns

It can be seen in **Fig. 7** and **Fig. 8**, from the beginning of loading to the maximum load, the behavior of GFRP RC columns is divided into two stages (the boundary between these stages is shown by a dashed line). The first stage is considered a linear relationship between the load and the strain in which the strains nearly increase proportionally with the applied load. The second stage is close to the failure load, in this stage strains develop rapidly, especially plastic strains. For the group of plain concrete columns, the boundary of the two stages is about 93% of the failure load (**Fig. 8**). Meanwhile, in GFRP RC columns, due to the contribution of GFRP bars, the elastic stage is extended, so stage 2 starts a little later in comparison with plain concrete columns. Consequently, the boundary between two stages occurred at nearly 97% of failure load (**Fig. 7**).

To clarify the contribution of GFRP bars in increasing the extremes strains of the RC columns at ultimate load, a comparison of strains in the tested columns (Groups #1 to #10) at the maximum load (ε_u) are conducted (Table- IV). As can be seen from Table- IV, the strain in the group of plain concrete columns (Group #1) varies from 1364 με to 1592 με (average – 1489 με), while in the groups of GFRP RC columns, the value of strains varies from 1528 με to 2614 με, which is 2.64% to 75.6% higher than the average value in the plain concrete columns.

Table- IV: Test results

Groups of columns	Specimen ID	R_m, MPa	P_u, kN	Nonlinear regression analysis results		ε_u ($\mu\varepsilon$)	P_f, kN
				$P_{u,cal}$, kN	% error		
#1	M1-1	30.2	460	467.3	-1.6	1364	-
	M1-2	30.2	472	467.3	1.0	1592	-
	M1-3	30.2	440	467.3	-6.2	1510	-
#2	M1-4F6-F6S100-1	29.9	436	478.3	-9.7	1821	4.1
	M1-4F6-F6S100-2	30.8	512	478.3	6.6	1528	3.5
	M1-4F6-F6S100-3	29.6	470	478.3	-1.8	1530	3.5
#3	M1-4F8-F6S100-1	32.2	530	485.7	8.4	2249	8.6
	M1-4F8-F6S100-2	28.8	510	485.7	4.8	1566	6.0
	M1-4F8-F6S100-3	30.3	540	485.7	10.1	2078	8.0
#4	M1-4F10-F6S100-1	32.2	490	498.7	-1.8	2324	15.3
	M1-4F10-F6S100-2	28.8	475	498.7	-5.0	2115	13.9
	M1-4F10-F6S100-3	28.7	468	498.7	-6.6	2202	14.5
#5	M1-4F12-F6S100-1	32.2	578	515.3	10.8	1837	18.4
	M1-4F12-F6S100-2	28.8	465	515.3	-10.8	1728	17.3
	M1-4F12-F6S100-3	30.3	510	515.3	-1.0	2326	23.3
#6	M1-8F10-F6S100-1	31.2	520	530.2	-2.0	1635	21.5
	M1-8F10-F6S100-2	28.7	542	530.2	2.2	1655	21.8
	M1-8F10-F6S100-3	30.3	530	530.2	0.0	1910	25.1
#7	M1-4F14-F6S100-1	28.8	506	535.2	-5.8	2156	30.7
	M1-4F14-F6S100-2	31.2	516	535.2	-3.7	1820	25.9
	M1-4F14-F6S100-3	30.3	528	535.2	-1.4	1848	26.3
#8	M1-8F12-F6S100-1	33.2	626	563.4	10.0	1988	39.9
	M1-8F12-F6S100-2	28.7	522	563.4	-7.9	1638	32.9
	M1-8F12-F6S100-3	29.9	576	563.4	2.2	1570	31.5
#9	M1-4F12-F6S50-1	32.2	579	-	-	2133	21.4
	M1-4F12-F6S50-2	28.8	470	-	-	1963	19.7
	M1-4F12-F6S50-3	30.3	483	-	-	2614	26.2
#10	M1-4F12-F6S200-1	32.2	468	-	-	1622	16.3
	M1-4F12-F6S200-2	28.8	491	-	-	2138	21.4
	M1-4F12-F6S200-3	30.3	494	-	-	1697	17.0
#11	M2-4F14-F6S100-1	43.2	612	-	-	2158	30.7
	M2-4F14-F6S100-2	43.2	632	-	-	1937	27.5
	M2-4F14-F6S100-3	43.2	642	-	-	1687	24.0

B. Relationship between load-carrying capacity and reinforcement ratio

During the experimental process, it is found that the first visible cracks in GFRP RC columns occur at a load from 91% to 96% of the maximum load. Due to the contribution of GFRP bars, after crack appearing, GFRP RC columns continue to absorb the load. The failure process of GFRP-RC columns occurs in the following sequence: first, the protective layer of concrete in the middle part of the column outside the reinforcement separates while the concrete core remains inside the reinforcement cage in the form of an hourglass; further with the increasing applied load, the confining material ruptures and the specimen is completely destroyed. (**Fig. 9**). Failure modes of tested GFRP reinforced columns depend on the pitches of stirrups. In columns with sparse pitches of stirrups 100 mm and 200 mm, the column failure occurs by buckling of longitudinal bars. In columns with dense pitches of stirrups 50 mm, due to the small tie spacing, the longitudinal reinforcement becomes more stable, then the lateral expansion of the confinement core leads to an increase in deformation and stress in the stirrup. As a result, the core is broken first and then the column is completely destroyed.

Fig. 9. Failure of the GFRP RC column specimens

The loading on columns is carried out following the above process until complete failure, the maximum load from the test is the load-carrying capacity P_u. The load-carrying capacity and average cubic strength of tested columns are indicated in the Table- IV. When changing GFRP reinforcement ratio from 0.36% to 3.24%, the average load-carrying capacity of the groups GFRP RC columns (groups #2 to #8) is 3.4% to 25.7% higher than the average bearing capacity of plain concrete columns (group #1).

By conducting a nonlinear regression analysis of the experimental data for the groups of columns with the same concrete mix #1 and stirrup configuration (Groups from #1 to #8), we obtain a suitable relationship between the main reinforcement ratio and the bearing capacity of tested columns as shown in **Fig. 10**. It can be seen that, within the reinforcement ratio from 0% to 3.24% and current concrete mix #1, the relationship between the bearing capacity and the GFRP reinforcement ratio is almost linear and this relationship could be expressed as follows:

$$P_u = 29.66\mu_f + 467.3 \tag{4}$$

According to the test results, the average bearing capacity of plain concrete columns is $P_C = 457.33\ kN$, which is approximately equal to the constant on the right side of expression (4). Therefore, equation (4) can be rewritten as follows:

$$P_u = 29.66\mu_f + P_C \tag{5}$$

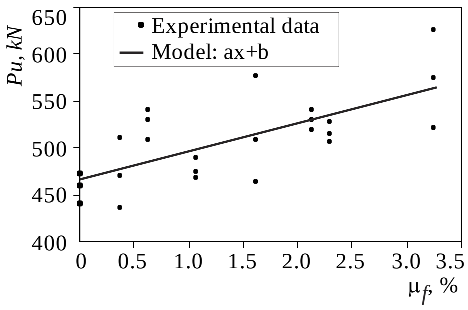


Fig. 10. Load-carrying capacity versus GFRP reinforcement ratio

C. *Influence of tie spacing on load-carrying capacity of GFRP RC columns*

As indicated above, the configuration of the transverse reinforcement remarkably affects the behavior of GFRP RC columns under pure axial load. Early studies in this field showed that, decreasing pitches of stirrups increases the confinement effect as well as limits transverse deformation. Consequently, the bearing capacity of the column rises. To clarify this, groups of columns #5, #9 and #10 were fabricated and tested (Table- III). The average load-carrying capacities of these groups are shown in **Fig. 11**. It can be seen from the graph, the average bearing capacity of group with tie spacing 200 mm is 484.3 *kN*, which is 5.2% and 6.5% lower than the average bearing capacity of the groups with tie spacing 50 mm and 100 mm respectively. Meanwhile, the average bearing capacity of the groups with tie spacing 50 mm and 100 mm is almost the same, which suggests that, when the tie spacing is reduced to a certain value, its influence on load-carrying capacity is not significant.

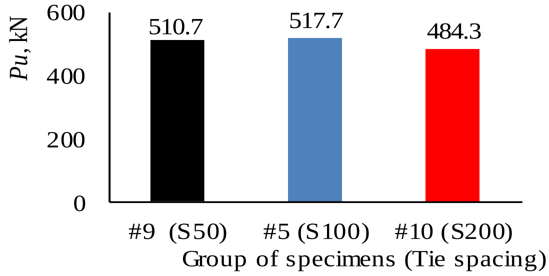


Fig. 11. Load-carrying capacities of groups of columns with different tie spacings

D. Contribution of GFRP bars to load-carrying capacity

The contribution of GFRP bars is defined as the percentage of total axial force in the GFRP bars to maximum load - P_f / P_u, % (Table- IV). In which, P_f depends on the average strains in the GFRP bars at the maximum load picked up from the tests and compressive modulus of elastic – $P_f = \varepsilon_f E_{fc} A_f$

Fig. 12 illustrates response of the ratio P_f / P_u to the change in reinforcement ratios. It can be seen that the contribution of GFRP reinforcement is positively related to the reinforcement ratios. In the range of reinforcement ratio from 0.36% to about 1.6%, this relationship is almost linear, futher incesing the reinforcement ratio the raising trend of ratio P_f / P_u decreases in comparison with reinforcement ratio. This can be explained as follows: increasing the longitudinal reinforcement ratio makes longitudinal bars more stable under axial load, which reduces the probability of buckling failure, resulting in higher strains at the maximum failure load (Table- IV).

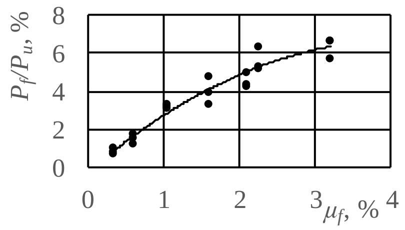


Fig. 12. Contribution of GFRP bars to load carrying capacity P_f/R_u versus reinforcement ratio curve

Retrieval Number: B2372129219/2019©BEIESP
DOI: 10.35940/ijeat.B2372.129219
Journal Website: www.ijeat.org

Published By:
Blue Eyes Intelligence Engineering
& Sciences Publication

International Journal of Engineering and Advanced Technology (IJEAT)
ISSN: 2249-8958 (Online), Volume-9 Issue-2, December, 2019

To identify the contribution of longitudinal reinforcement on the bearing capacity of the column when using different classes of concrete, groups of columns #7 and #11 with the same reinforcement but different classes of concrete were tested (Table- I). According to the test results, we compare the GFRP contribution of these groups of columns (**Fig. 13**). The comparison result shows that, when increasing concrete strength, the contribution of longitudinal GFRP bars on the bearing capacity of the column decreases in comparison with lower concrete strength. This can be explained by two reasons: first - the bearing capacity of the column is largely determined by the strength of the concrete; second - the ultimate strain of the concrete decreases with increasing strength.

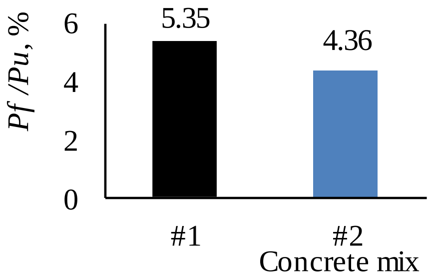


Fig. 13. Contribution of GFRP bars to load-carrying capacity of columns with different concrete grades

E. Prediction of load-carrying capacity

As indicated above, to calculate the bearing capacity of reinforced concrete columns, most authors use the tensile strength and elastic modulus of GFRP bars. Combining the formula for traditional reinforced concrete columns according to Vietnamese Standard 5574:2018 [20] and the compression test of GFRP bars conducted by Qasim [16], we propose the following formula to predict the bearing capacity of GFRP RC columns:

$$P_u = R_b\left(A - A_{st}\right) + \varepsilon_u A_{st} E_{fc} \quad (5)$$

where: A - the cross-sectional area of the column; A_{st} - total cross-sectional area of longitudinal GFRP bars; E_{fc} - the elastic modulus of the GFRP bar in compression, which is 1.59 times less than the modulus of elasticity in tension [16]; ε_u - ultimate strain in concrete at maximum load. Basing on Vietnamese Standard 5574:2018 [20] and experimental results (Table- IV) we recommend this value should be equal 0.002. **Fig. 14** shows the ratio of experimental maximum load to the theoretical ultimate capacity estimated by equations (1) and (6) ($P_{u(exp)}/P_{u(theor)}$). Results from equation (6) almost match the results from the formulas proposed by Mohammad Z. Afifi et al. [14] and Muhammad N. S. et al. [15]**,** which pointed out that the ratio of the experimental maximum load compared to the theoretical ultimate capacity ranged from 0.86 to 1.13. Meanwhile, according to the equation proposed by Tobbi et al. [22] this ratio varied from 0.94 to 1.34. Although formula (6) gives skewed results compared to the test results. However, it can be explained by the error of the experimental equipment and the influence of eccentricity during the experiment.

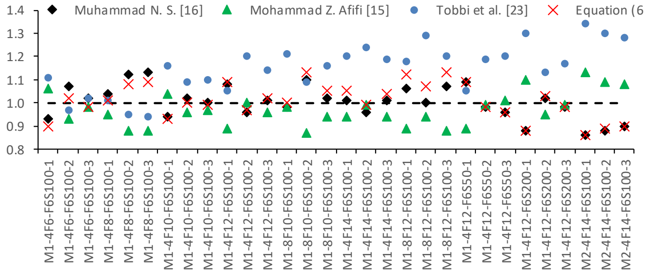


Fig. 14. Comparison of experimental maximum load with theoretical ultimate capacity

IV. CONCLUSION

The results of experimental studies of 11 groups of reinforced concrete and concrete columns allow us to distinguish two typical stages of works of the columns from beginning loading to maximum load, to identify the relation between the reinforcement ratio and the bearing capacity of the columns and to clarify the contribution of GFRP bars to the load-carrying capacity with different reinforcement ratios and concrete grades. In addition, the influence of the tie spacing on the bearing capacity and failure modes of columns is also considered and the formula for predicting the load carrying capacity of GFRP RC columns are modified. Based on the experimental results presented in this study the following conclusions can be drawn:

- From beginning of loading on column specimens to maximum load, behavior of GFRP RC columns and plain concrete columns under instant axial load can be divided into two stages: elastic stage and failure stage;
- Within the studied longitudinal reinforcement ratios from 0% to 3.24% (groups #1 to #8), the relationship of the load-carrying capacity of GFRP RC columns and the reinforcement ratio is practically linear. According to the obtained results, it seems that the bearing capacity of the GFRP RC column will continue to increase as the reinforcement ratio exceeds 3.24%. Further studies should be conducted to examine the limit reinforcement ratio for GFRP RC columns;
- Configuration of transverse reinforcement greatly affects the bearing capacity and failure modes of the column. However, if the tie spacing is further reduced, the bearing capacity of the column is almost unchanged and the failure type of the column changes from buckling of longitudinal bars to the rupture of concrete core;
- Increasing the reinforcement ratio boosts the ultimate strains at the maximum load, so the contribution of GFRP bars to load-carrying capacity of GFRP RC columns increases gradually with reinforcement ratio. However, the increase in ultimate strain is limited. When the reinforcement ratio exceeds 1.6%, the relationship between the contribution of GFRP bars and the bearing capacity of columns is likely characterized by a curved shape and the contribution of GFRP bars tends to reach a fixed value at the reinforcement ratio from 3.24% onwards;

- GFRP bars can be used for columns with high strength concrete, however in this case the contribution of GFRP bars to overall load-carrying capacity will be reduced.

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