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Maximum axial load carrying capacity of Fibre Reinforced-Polymer (FRP) bar reinforced concrete columns under axial compression

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Abstract

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Keywords

(frp), maximum, reinforced, axial, compression, under, columns, concrete, load, carrying, capacity, fibre, reinforced-polymer, bar

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Maximum Axial Load Carrying Capacity of Fibre Reinforced-Polymer (FRP) Bar Reinforced Concrete Columns under Axial Compression

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Abstract

In this study, a new equation is proposed to compute the maximum axial load carrying capacity of FRP bar reinforced concrete columns under axial compression. The equation proposed in this study was critically compared with the equations proposed in the previous research studies using a wide range of experimental data taken from the available literature. In general, it was found that computing the contribution of the FRP longitudinal bars in concrete columns based on the modulus of elasticity (stiffness) of the FRP bars provides more rational predictions than computing the contribution of the FRP longitudinal bars based on the ultimate tensile strength of the FRP bars. It was also found that using a concrete compressive strength-based empirical equation in estimating the axial strain in the FRP longitudinal bars in concrete columns provides more accurate predictions of the contribution of the longitudinal FRP bars in the axial load sustained by the FRP bar reinforced concrete columns.

Keywords:

Concrete columns; FRP bars; concentric axial load; maximum load carrying capacity

1 Introduction

The main function of a reinforced concrete column is to sustain axial loads with or without bending moments. The axial load carrying capacity of steel bar reinforced concrete columns decreases over the design (service) life of the concrete structures due to the corrosion of steel bars, especially in coastal regions or in harsh environments. The cost of rehabilitation and repair of deteriorated concrete structures is significantly high [1]. The National Association of Corrosion Engineers (NACE) International reported that the United States of America spends about two billion dollars annually to replace and repair the piers of the concrete bridges and about one billion dollars annually for maintaining marine piling systems [2].

The review of the literature found that Fibre Reinforced-Polymer (FRP) composites can be used in a wide range of civil/structural applications. The FRP composites have various structural forms that can be classified into two main classes: 1) External reinforcement (FRP jacketing) and 2) Internal reinforcement (FRP reinforcing bars) [3-4]. The FRP composites including FRP bars possess many advantageous characteristics such as the resistance to the harsh environmental conditions, light weight and high tensile strength [5-6]. Hence, FRP bars have the potential to replace steel bars and overcome the deterioration of concrete structures associated with the corrosion of steel reinforcement. However, the use of FRP bars as reinforcement in compression concrete members is still not recommended. This is because the ultimate compressive strength of the FRP bar is considerably lower than its ultimate tensile strength [7]. Chaallal and Benmokrane [8] tested GFRP bars of three different diameters (15.9, 19.1 and 25.4 mm) and observed that the average compressive strength of the GFRP bars was 77% of the tensile strength. Kobayashi and Fujisaki [9] reported that the strength of the Aramid-FRP (AFRP), Glass-FRP (GFRP) and Carbon-FRP (CFRP) bars under axial compression were about 10%, 30-40% and 30-50% of their tensile strength, respectively. Deitz et al. [10] tested GFRP bars with a diameter of 15 mm under axial compression and observed that the ultimate compressive strength of the GFRP bars was approximately equal to 50% of their tensile strength.

The acceptance of FRP bars as longitudinal reinforcement in concrete compression members such as concrete columns requires the development of design guidelines. In this regard, experimental and analytical research studies were conducted to investigate and to understand the behaviour of concrete columns reinforced longitudinally with FRP bars. Generally, the load carrying capacities of the FRP bar reinforced concrete columns under concentric and eccentric axial loads are lower than the load carrying capacities of steel bar reinforced concrete columns having same dimensions and reinforced with the same longitudinal and transverse reinforcements. The reason for this is mainly attributed to the lower ultimate compressive strength and the lower modulus of elasticity of FRP bars in compression compared to those of conventional steel bars in compression [2, 11-16]. Alsayed et al. [11] reported that the direct replacement of the longitudinal steel reinforcement with an equivalent amount of GFRP reinforcement reduced the axial load carrying capacity of the concrete columns by about 13%, irrespective of the type of the transverse reinforcement (steel or GFRP). Choo et al. [12] observed through an analytical study on FRP bar reinforced square concrete columns that ignoring the contribution of the longitudinal FRP bars in the compression region of the concrete columns may be overly conservative. Tobbi et al. [13] and Afifi et al. [14] reported that GFRP and CFRP longitudinal bars can contribute up to 10% and 13% of the axial load carrying capacity of the concrete columns, respectively. Hadhood et al. [15] reported that GFRP longitudinal bars contributed about 5% of the axial load carrying capacity of GFRP bar reinforced high strength concrete (HSC) columns. A similar contribution for GFRP bars in HSC columns was also reported in Hadi et al. [16].

Due to the variances in the reported ultimate compressive strength of the FRP bars and their contribution as longitudinal reinforcement in concrete columns, no theoretical equation was recommended in the CAN/CSA S806-12 [17] or in ACI 440.1R-15 [18] to predict the maximum axial load carrying capacity of FRP bar reinforced concrete columns. Nonetheless, several theoretical equations were proposed in the previous research studies to predict the maximum axial load carrying capacity of FRP bar reinforced concrete columns. However, these equations have not been adequately assessed using a wide range of experimental data.

In this study, a new equation is proposed to predict the maximum axial load carrying capacity of concrete columns reinforced longitudinally with FRP bars. The theoretical equations, proposed in this study and in the previous studies, were critically assessed using a wide range of experimental data taken from the available literature. The observations reported in this study can help in establishing guidelines for designing FRP bar reinforced concrete compression members

2 Conceptual assumptions

The analysis of conventional steel bar reinforced concrete members is based on several basic assumptions, which are essential to compute the load carrying capacity of these members under different loading conditions. It was reported that these assumptions might be applicable to be used for GFRP bar reinforced concrete members [12, 15, 19]. Therefore, the assumptions were presented first and were used to analytically investigate the behaviour of GFRP bar reinforced circular concrete columns under concentric axial loads. The basic assumptions are:

1. The maximum strain, ε_c , in concrete does not exceed an assumed ultimate concrete compressive strain, ε_{cu}.
2. A perfect bond exists at the interfaces between the GFRP bars and the surrounding concrete.
3. The axial strain in the concrete, ε_c , and the axial strain in GFRP reinforcing bars, ε_f , are equal at any concentric axial load.

3 Maximum axial load carrying capacity of reinforced concrete columns

This study mainly focuses on the development of a theoretical equation for predicating the maximum axial load carrying capacity of FRP bar reinforced concrete columns, which occurs when columns are subjected to pure compression loads (axial loads with zero eccentricities). The proposed equation for the maximum axial load carrying capacity of FRP bar reinforced concrete columns can be incorporated in the future design codes for composite structures. The effects of the combined axial and flexural loads on the behaviour of FRP bar reinforced concrete columns are considered beyond the scope of this study.

3.1 Steel bar reinforced concrete columns

The maximum axial load carrying capacity, P_o, of conventional steel bar reinforced concrete columns under concentric axial load can be predicted using Eq. 1 [20-21].

$$P_o = P_c + P_{bar,st} \tag{1}$$

$$P_c = \alpha f_c' \left(A_g - A_{st}\right) \tag{2}$$

$$P_{bar,st} = f_y A_{st} \tag{3}$$

Equation 1 represents the summation of the axial loads sustained by the concrete and steel longitudinal bars. The P_c represents the contribution of the concrete considering the gross area of the columns A_g as shown in Eq. 2. The $P_{bar,st}$ represents the contribution of the longitudinal steel bars. The f_y and A_{st} are the yield strength and the total cross-sectional area of the longitudinal steel bars. It is noted that in Eq. 1 - Eq. 3, the effect of shear reinforcement has not been taken into account for calculating the maximum axial load carrying capacity of reinforced concrete columns under concentric axial loads [20].

The compressive strength of plain concrete in full-scale concrete columns tested under concentric axial loads is generally lower than the compressive strength of standard concrete cylinders. The differences between the compressive strength of concrete in columns and the compressive strength of standard concrete cylinders are commonly attributed to the differences in the shape, size and concrete casting process between columns and cylinders. In Eq. 2, the parameter α is a reduction factor that represents the ratio between the in-place compressive strength of concrete in actual concrete columns to the compressive strength of standard concrete cylinders. Extensive experimental investigations were carried out on reinforced concrete columns and the parameter α was recommended to be taken equal to 0.85 [22]. The recommended value for the parameter α was considered in ACI 318-14 [20] to determine the contribution of the concrete in the maximum axial load carrying capacity of conventional steel bar reinforced concrete columns (Eq. 4).

$$P_o = 0.85 f_c' \left(A_g - A_s\right) + f_y A_s \quad (4)$$

3.2 FRP bar reinforced concrete columns

Different equations were proposed in the previous research studies to predict the maximum axial load carrying capacity of FRP bar reinforced concrete columns. It is important to note that the contribution of the concrete, in the analytically computed axial load carrying capacity of FRP bar reinforced concrete columns, remains similar in all of the proposed equations. In other words, the differences in the analytically computed values of P_o for FRP bar reinforced concrete columns are primarily due to the different concepts adopted in different proposed equations for calculating the contribution of the FRP longitudinal bars ($P_{bar,FRP}$).

The compressive strength of FRP bar is considerably lower than its tensile strength and the behaviour of FRP bar under compressive loads differs significantly, as mentioned above. Hence, ACI 440.1R-06 [23] recommends not to reinforce concrete columns longitudinally with FRP bars and ACI 440.1R-15 [18] provided no recommendations in this regard. The CAN/CSA S806-12 [17] permits reinforcing concrete columns longitudinally with FRP bars. However, CAN/CSA S806-12 [17] recommends neglecting the contribution of the

FRP longitudinal bars when predicting the maximum axial load carrying capacity of the FRP bar reinforced concrete columns. Based on the recommendations in the CAN/CSA S806-12 [17], the maximum axial load carrying capacity of FRP bar reinforced concrete columns can be predicted using Eq. 5.

$$P_o = 0.85 f_c' \left(A_g - A_f\right) \tag{5}$$

where A_f represents the total cross-sectional area of GFRP longitudinal bars.

However, a considerable number of research studies observed that disregarding the contribution of FRP longitudinal bars in compression, as in Eq. 5, might result in a large difference between the analytically computed and the experimentally obtained axial load carrying capacity of the FRP bar reinforced concrete columns [14, 24-25]. Therefore, two approaches were considered to compute the contribution of FRP longitudinal bars in the maximum axial load carrying capacity of FRP bar reinforced concrete columns. In the first approach, the axial load sustained by FRP longitudinal bars is calculated using the tensile strength of the FRP bars, $\alpha_f f_{fu} A_f$ (Eq. 6). In the second approach, the axial load sustained by FRP longitudinal bars is calculated using the axial strain in the FRP bars and the stiffness (modulus of elasticity) of the FRP bars, $\varepsilon_f E_f A_f$ (Eq. 7).

$$P_o = 0.85 f_c' \left(A_g - A_f\right) + \alpha_f f_{fu} A_f \tag{6}$$

$$P_o = 0.85 f_c' \left(A_g - A_f\right) + \varepsilon_f E_f A_f \tag{7}$$

In Eq. 6, the α_f is a reduction factor that represents the ratio between the strength of FRP bar under compression and the strength of the FRP bar under tension. Different values for α_f were recommended in the previous studies. Alsayed et al. [11] suggested taking α_f equal to 0.6. Later, Tobbi et al. [13] recommended taking α_f equal to 0.35 based on experimental observations reported in Kobayashi and Fujisaki [9]. Also, α_f was recommended to be taken equal to 0.35 in Afifi et al. [26] for GFRP bar reinforced circular concrete columns. However, for CFRP bar reinforced circular concrete columns, Afifi et al. [14] recommended taking α_f equal to 0.25.

In Eq. 7, different values were also suggested for the axial strain in the FRP longitudinal bars, ε_f , at the maximum axial load carrying capacity of the concrete columns. Mohamed et al. [2] suggested taking ε_f equal to 0.002, explaining that this value (ε_f = 0.002) represents the axial strain in the FRP longitudinal bars at the initiation of the micro-cracks in the plastic stage of the concrete. However, Hadi et al. [25] recommended taking ε_f equal to 0.003, which represents the ultimate strain of the concrete, ε_{cu}.

It is obvious that different research studies proposed different equations based on a limited number of experimental data. Therefore, there is no consensus in the previous research studies on a unified equation for predicting the maximum axial load carrying capacity of FRP bar reinforced concrete columns, which may also be attributed to the variances in the response of the FRP bars under axial compression.

In this study, the axial load sustained by FRP longitudinal bars, $P_{bar,FRP}$, was predicted based on the stiffness (modulus of elasticity) of the FRP bars because the modulus of elasticity of FRP bars in compression is approximately similar to the modulus of elasticity of FRP bars in tension [8, 10]. On the other hand, the compressive strength of the FRP longitudinal bars corresponding to their ultimate tensile strength tends to fluctuate significantly, depending on the manufactures and the type of the FRP bars (AFRP, GFRP and CFRP). The significant fluctuations in the compressive strength of the FRP bars can be considered one of the main reasons why different values for the reduction factor α_f were proposed in different research studies. Hence, simply changing the value of the reduction factor α_f in Eq. 6 might not provide reasonable predictions for the maximum axial load carrying capacity of FRP bar reinforced concrete columns. The axial strain in the FRP longitudinal bars ε_f at the maximum axial load carrying capacity of the concrete columns was considered to be equal to the concrete axial strain at peak stress ε_{co}. The concept adopted in this study is consistent with the third assumption in Section 2, which states that the axial strain in the concrete and the axial strain in longitudinal FRP reinforcing bars are equal at any concentric axial load. Accordingly, the maximum axial load carrying capacity of FRP bar reinforced concrete columns can be predicted using Eq. 8:

$$P_o = 0.85 f_c' \left(A_g - A_f\right) + \varepsilon_{co} E_f A_f \tag{8}$$

Based on a considerable number of theoretical and experimental research studies, several empirical formulas were proposed in the past few decades for computing the concrete axial strain at peak stress, ε_{co}. In this study, four of the available, applicable and widely accepted formulae (Eq. 9 - Eq. 12) were used to compute ε_{co} in Eq.8.

$$\varepsilon_{co} = 735\ (f_c')^{0.25} \times 10^{-6} \tag{9}$$

$$\varepsilon_{co} = 780\ (f_c')^{0.25} \times 10^{-6} \tag{10}$$

$$\varepsilon_{co} = 0.0005\,(f_c')^{0.4} \tag{11}$$

$$\varepsilon_{co} = 0.0016\,\exp(240\ f_c'/E_1) \tag{12}$$

Equation 9 was proposed in Popovics [27] for normal strength concrete with compressive strength of up to 50 MPa. whereas Eq. 10, proposed in Wee et al. [28], covered concrete with a compressive strength of up to 125 MPa. Legeron and Paultre [29] proposed Eq. 11 for concrete with compressive strength ranging between 20 and 125 MPa, while Eq. 12, proposed in Yang et al. [30], is applicable to concrete with compressive strengths ranging between 10 and 180 MPa. Although Eq. 9 is applicable for normal strength concrete, an average difference of only 6% was observed between the values of the concrete axial strain, ε_{co}, obtained from using Eq. 9 and Eq. 10. But, the values of ε_{co} obtained using Eq. 9 and Eq. 10 were consistently below the values of ε_{co} obtained using Eq. 11 and Eq. 12, especially for concrete having compressive strength greater than 100 MPa.

4 Critical assessment of the proposed equations

The equation proposed in this study was critically reviewed using a wide range of published experimental data (Table 1). The equation proposed in Tobbi et al. [9] was also examined. Hadi et al. [25] recommended assuming ε_f equal to ε_{cu}. The equation proposed in Hadi et al. [25] was also assessed, firstly by taking ε_{cu} equal to 0.003 as defined in the ACI 318-14 [20] and secondly by taking ε_{cu} equal to 0.0035 as defined in the CSA A23.3-14 [21].

Table 2 presents the ratios between the analytically predicted and the experimentally obtained axial load carrying capacity ($P_o/P_{exp.}$) for the experimentally tested specimens presented in Table 1. The analytically predicted axial load carrying capacity, P_o, was calculated using either Eq. 6 by taking α_f equal to 0.35, as recommended in Tobbi et al. [13] or using Eq. 8, in which the value of ε_{co} was either computed using the formulas presented in the above section (Eq. 9 - Eq. 12) or taken equal to ε_{cu} (0.003 or 0.0035 as defined in the ACI 318-14 [20] and CSA A23.3-14 [21], respectively).

In Table 2, the accuracy of the equations proposed in this study and in the previous research studies in predicting the maximum axial load carrying capacity of FRP bar reinforced concrete columns was examined using four different mathematical measurements: Mean value (μ); Standard Deviation (SD); Coefficient of Variation (COV) and the Mean Absolute Percentage Error ($MAPE$). The Mean value (μ) represents the central value of the discrete set of P_o values. The Standard Deviation (SD) was used to quantify the level of variation (dispersion) for the values of P_o. High standard deviation indicates that the predicted axial load carrying capacities of the FRP bar reinforced concrete specimens are spread out over a wider range of values (less reliable) and vice versa. The standard deviation (SD) indicates to an absolute term of how much the values of P_o are spread. The values of P_o were then assessed as a percentage of how far away they spread from their mean value using the Coefficient of Variation (COV). The lower the COV, the lower the dispersion of P_o is from the mean value. The Mean Absolute Percentage Error ($MAPE$) is also used as a measurement to examine the accuracy of the equation proposed in this study and the equations proposed in previous research studies for the maximum axial load carrying capacities of the FRP bar reinforced concrete columns. The lower the $MAPE$, the better predictions provided by the proposed equation.

Table 2 presents the comparison between the experimental and theoretical axial load carrying capacity of FRP bar reinforced concrete columns. It was found that Eq. 8, in which the contribution of the FRP bars is computed based on the stiffness of the FRP bars, provides more reliable and safer predictions for P_o compared to Eq. 6, in which the contribution of the FRP bars is computed using the tensile strength of the FRP bars. This might be mainly attributed to the fact that the modulus of elasticity of the FRP bars in tension is approximately equal to the modulus of elasticity of FRP bars in compression, while there is a large difference between the tensile and the compressive strength of the FRP bars. It was also found that, in Eq. 8, the use of the formula proposed by Legeron and Paultre [29] (Eq. 11) in computing $P_{bar,FRP}$ provides lower discrepant values of P_o, as shown in Fig. 1, giving a standard deviation and a coefficient of variation of 0.071 and 7.71, respectively. On the other hand, taking the concrete axial strain at peak stress ε_{co} equal to 0.003 when computing $P_{bar,FRP}$ provided predictions for P_o with the lowest percentage of error giving a mean absolute percentage error $MAPE$ of 7.542. Furthermore, taking ε_{co} equal to 0.0035 when computing $P_{bar,FRP}$ provided predictions with the highest but rather safe mean value μ for $(P_o/P_{exp.})$ of = 0.97, which is very close to the unity, but with high SD and COV of 0.082 and 8.47, respectively (Fig. 1).

Figure 2 shows the relationship between the $(P_o/P_{exp.})$ for the specimens presented in Table 2 and the compressive strength of the concrete. In Fig. 2, the P_o was either obtained from Eq. 6 assuming α_f equal to 0.35 as recommended in Tobbi et al [13] or from Eq. 8 taking ε_{co} equal to 0.003 or 0.0035 or computed using the formula proposed by Legeron and Paultre [29].

Assuming α_f equal to 0.35 (Eq. 6), as recommended in Tobbi et al [13], over-predicts the axial load carrying capacity for most of the FRP bar reinforced NSC and HSC columns presented in Table 1, as shown in Fig 2a. However, using Eq. 8, assuming ε_{co} equal to 0.003 for computing the contribution of the FRP bar over-predicts the axial load carrying capacity of 23% of the FRP bar reinforced specimens. But in general it provides reasonable predictions with mean value μ for $(P_o/P_{exp.})$ of 0.95. On the other hand, using Eq. 8, considering the formula proposed in Legeron and Paultre [29] (Eq. 11) for ε_{co} in computing the contribution of the FRP bar over-predicts the axial load carrying capacity of only 6% of the total number of the specimens presented in Table 2 with a mean value μ for $(P_o/P_{exp.})$ of 0.93, hence, it provides reliable and safe prediction for nearly all the FRP bar reinforced NSC and HSC column specimens (Fig 2d).

5 Conclusions

The present study proposes a theoretical equation for predicting the maximum axial load carrying capacity of FRP bar reinforced concrete columns. In the proposed equation, the contribution of the FRP longitudinal bars was computed based on the axial strain and the stiffness (modulus of elasticity) of the FRP longitudinal bars. An empirical equation based on the concrete compressive strength was developed to compute the axial strain in the FRP longitudinal bars at the maximum axial load carrying capacity of the FRP bar reinforced concrete columns. The proposed equation was validated using a large set of experimental data available in the literature. The equation proposed in this study provided more accurate and safe predictions of the experimentally tested FRP bar reinforced columns. The theoretical equation proposed in this study can be easily applied in predicting the axial load carrying capacity of normal strength and high strength concrete columns reinforced with different types of FRP bars.

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Fig. 1: Experimental versus predicted axial load carrying capacity of FRP bar reinforced concreter columns obtained using: a) Eq. 6 (α_f = 0.35); b) Eq. 8 (ε_{co} = 0.003); c) Eq. 8 (ε_{co} = 0.0035) and d) Eq. 8 ($\varepsilon_{co} = 0.005(f_c')^{0.4}$).

Fig. 2: The relationship between $P_o/P_{exp.}$ of the FRP bar reinforced concrete column and the compressive strength of the concrete f_c'. Note: P_o were obtained using: a) Eq. 6 (α_f = 0.35); b) Eq. 8 (ε_{co} = 0.003); c) Eq. 8 (ε_{co} = 0.0035) and d) Eq. 8 ($\varepsilon_{co} = 0.005(f_c')^{0.4}$).

Table 1: Experimental data of FRP bar reinforced concrete columns taken from available previous research studies.

Research study	Specimen cross-section			FRP longitudinal reinforcement					Concrete
	Specimen	Column shape	Dimensions* (mm)	Type	No. of bars	Diameter (mm)	f_{fu} (MPa)	E_f (MPa)	f_c' (MPa)
Afifi et al. [14]	SP-1	Circular	300	CFRP	6	12.7	1899	140000	42.9
	SP-2	Circular	300	CFRP	10	12.7	1899	140000	42.9
	SP-3	Circular	300	CFRP	10	12.7	1899	140000	42.9
	SP-4	Circular	300	CFRP	10	12.7	1899	140000	42.9
	SP-5	Circular	300	CFRP	10	12.7	1899	140000	42.9
	SP-6	Circular	300	CFRP	10	12.7	1899	140000	42.9
	SP-7	Circular	300	CFRP	10	12.7	1899	140000	42.9
	SP-8	Circular	300	CFRP	10	12.7	1899	140000	42.9
	SP-9	Circular	300	CFRP	14	12.7	1899	140000	42.9
Afifi et al. [26]	SP-10	Circular	300	GFRP	4	15.9	934	55400	42.9
	SP-11	Circular	300	GFRP	8	15.9	934	55400	42.9
	SP-12	Circular	300	GFRP	8	15.9	934	55400	42.9
	SP-13	Circular	300	GFRP	8	15.9	934	55400	42.9
	SP-14	Circular	300	GFRP	8	15.9	934	55400	42.9
	SP-15	Circular	300	GFRP	8	15.9	934	55400	42.9
	SP-16	Circular	300	GFRP	8	15.9	934	55400	42.9
	SP-17	Circular	300	GFRP	8	15.9	934	55400	42.9
	SP-18	Circular	300	GFRP	12	15.9	934	55400	42.9
Mohamed et al. [2]	SP-19	Circular	300	GFRP	8	15.9	934	55400	42.9
	SP-20	Circular	300	GFRP	8	15.9	934	55400	42.9
	SP-21	Circular	300	GFRP	8	15.9	934	55400	42.9
	SP-22	Circular	300	CFRP	10	12.7	1899	140000	42.9
	SP-23	Circular	300	CFRP	10	12.7	1899	140000	42.9
	SP-24	Circular	300	CFRP	10	12.7	1899	140000	42.9

Table 1: (Continued)

Research study	Specimen cross-section			FRP longitudinal reinforcement					Concrete
	Specimen	Column shape	Dimensions* (mm)	Type	No. of bars	Diameter (mm)	f_{fu} (MPa)	E_f (MPa)	f_c' (MPa)
Karim et al. [31]	SP-25	Circular	205	GFRP	6	12.7	1600	66000	32
	SP-26	Circular	205	GFRP	6	12.7	1600	66000	32
	SP-27	Circular	205	GFRP	0	0	0	0	32
	SP-28	Circular	205	GFRP	0	0	0	0	32
Hales et al. [32]	SP-29	Circular	305	GFRP	6	16	715	44000	90
Hadhood et al. [15]	SP-30	Circular	305	GFRP	8	15.9	1289	54900	70.2
	SP-31	Circular	305	GFRP	12	15.9	1289	54900	70.2
Hadhood et al. [33]	SP-32	Circular	305	CFRP	8	15.9	1680	141000	35
Hadi et al. [16]	SP-33	Circular	210	GFRP	6	12.7	1548	67800	85
	SP-34	Circular	210	GFRP	6	12.7	1548	67800	85
Tobbi et al. [13]	SP-35	Square	350 x 350	GFRP	8	19.1	728	47600	33
	SP-36	Square	350 x 350	GFRP	12	15.9	751	48200	33
	SP-37	Square	350 x 350	GFRP	4+4	12.7, 15.9	1040, 751	46300, 48200	33
	SP-38	Square	350 x 350	GFRP	8	12.7	1040	46300	33

* Represents the diameter for circular columns and the length times the width of the square columns
f_{fu}= The ultimate tensile strength of FRP bars
E_f= The modulus of elasticity of FRP bars
f_c'= The compressive strength of the concrete

Table 2: Comparison between the experimental and theoretical axial load carrying capacity of FRP bar reinforced concrete columns available in the previous research studies

Study	Specimen	$\boldsymbol{P_{exp.}}$ [a]	$\boldsymbol{P_o/P_{exp.}}$ [b]						
			Eq. (8)						Eq. (6)
			Popovics [23] [c, e]	Wee et al. [24] [c, e]	Legeron and Paultre [25] [c, e]	Yang et al. [26] [c, e]	ACI 318-14 [16] [d]	CSA A23.3-14 [17] [d]	Tobbi et al. [9] [f]
Afifi et al [14]	S-1	2905	0.95	0.95	0.96	0.96	0.99	1.01	1.05
	S-2	3148	0.91	0.92	0.93	0.93	0.97	1.00	1.07
	S-3	2948	0.97	0.98	0.99	0.99	1.04	1.07	1.14
	S-4	3070	0.93	0.94	0.95	0.95	1.00	1.03	1.10
	S-5	3013	0.95	0.96	0.97	0.97	1.02	1.05	1.12
	S-6	2981	0.96	0.97	0.98	0.98	1.03	1.06	1.13
	S-7	3147	0.91	0.92	0.93	0.93	0.97	1.00	1.07
	S-8	2941	0.97	0.98	1.00	0.99	1.04	1.07	1.15
	S-9	3107	0.96	0.97	0.99	0.99	1.05	1.09	1.19
Afifi et al [26]	S-10	2826	0.93	0.93	0.94	0.94	0.95	0.96	0.99
	S-11	2951	0.90	0.91	0.92	0.92	0.94	0.96	1.03
	S-12	2857	0.93	0.94	0.95	0.95	0.97	0.99	1.06
	S-13	2964	0.90	0.91	0.92	0.92	0.94	0.95	1.03
	S-14	2920	0.91	0.92	0.93	0.93	0.95	0.97	1.04
	S-15	2804	0.95	0.96	0.97	0.97	0.99	1.01	1.08
	S-16	3019	0.89	0.89	0.90	0.90	0.92	0.94	1.01
	S-17	2865	0.94	0.94	0.95	0.95	0.97	0.99	1.06
	S-18	2998	0.90	0.92	0.93	0.93	0.96	0.98	1.09
Mohamed et al. [2]	S-19	2840	0.95	0.95	0.96	0.96	0.98	1.00	1.07
	S-20	2871	0.94	0.94	0.95	0.95	0.97	0.98	1.06

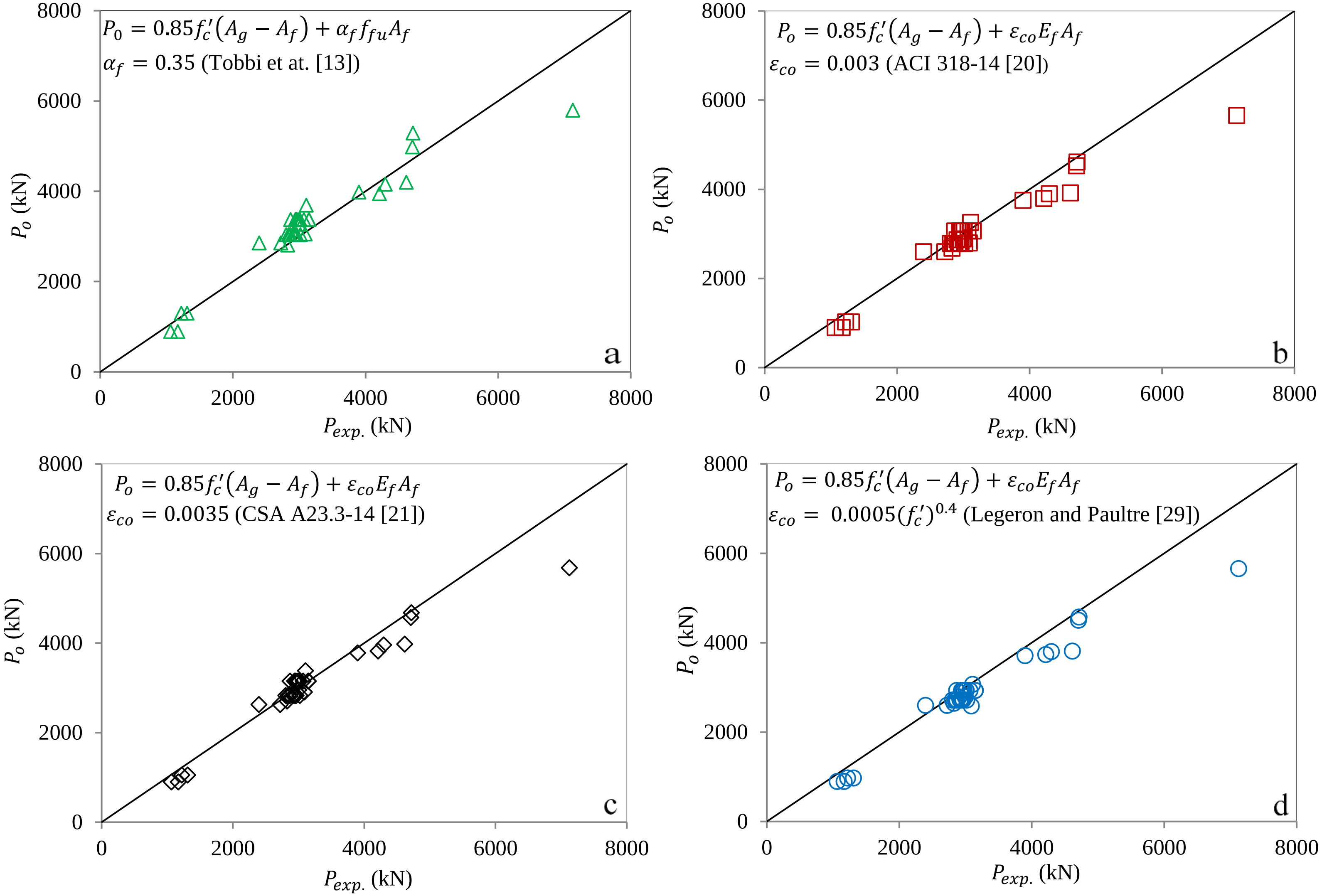
Table 2: (Continued)

Mohamed et al. [2]	S-21	2935	0.91	0.92	0.93	0.92	0.95	0.96	1.04
	S-22	2869	1.00	1.01	1.02	1.02	1.07	1.10	1.18
	S-23	2960	0.96	0.97	0.99	0.99	1.03	1.06	1.14
	S-24	3008	0.95	0.96	0.98	0.97	1.02	1.05	1.12
Karim et al. [31]	S-25	1220	0.79	0.80	0.80	0.81	0.84	0.86	1.07
	S-26	1309	0.73	0.74	0.74	0.75	0.79	0.80	1.00
	S-27	1063	0.84	0.84	0.84	0.84	0.84	0.84	0.84
	S-28	1170	0.77	0.77	0.77	0.77	0.77	0.77	0.77
Hales et al. [32]	S-29	7126	0.78	0.79	0.79	0.79	0.79	0.80	0.81
Hadhood et al. [15]	S-31	4709	0.94	0.95	0.96	0.95	0.96	0.97	1.06
	S-32	4716	0.95	0.96	0.97	0.97	0.98	0.99	1.12
Hadhood et al. [33]	S-30	3090	0.82	0.83	0.84	0.84	0.91	0.94	0.99
Hadi et al. [16]	S-33	2721	0.94	0.94	0.96	0.95	0.96	0.97	1.05
	S-34	2398	1.06	1.07	1.09	1.08	1.09	1.10	1.19
Tobbi et al. [13]	S-35	4297	0.87	0.88	0.88	0.89	0.91	0.92	0.97
	S-36	4615	0.81	0.82	0.83	0.83	0.85	0.86	0.91
	S-37	4212	0.88	0.88	0.89	0.89	0.90	0.91	0.94
	S-38	3900	0.94	0.95	0.95	0.95	0.96	0.97	1.02
Mean			0.91	0.92	0.93	0.93	0.95	0.97	1.05
SD			0.068	0.068	0.071	0.070	0.076	0.082	0.096

Table 2: (Continued)

COV (%)	7.39	7.45	7.71	7.55	7.99	8.47	9.17
MAPE	9.642	9.305	8.614	8.612	7.542	7.478	9.692

[a] $P_{exp.}$ is the experimental axial load carrying capacity of FRP bar reinforced concrete columns.
[b] P_o is the theoretically computed axial load carrying capacity of FRP bar reinforced concrete columns.
[c] Refers to the formula used in computing ε_{co} (Eq. 9 - Eq. 12)
[d] equal to ε_{cu} (0.003 or 0.0035 as defined in ACI 318-14 [20] and CSA A23.3-14[21], respectively).
[e] The contribution of the FRP longitudinal bars in P_o was computed based on the formula defined in the footnote "c" above.
[f] The contribution of the FRP longitudinal bars in P_o was assumed to be equal to $0.35 f_{fu} A_f$ (Tobbi et al. [13])



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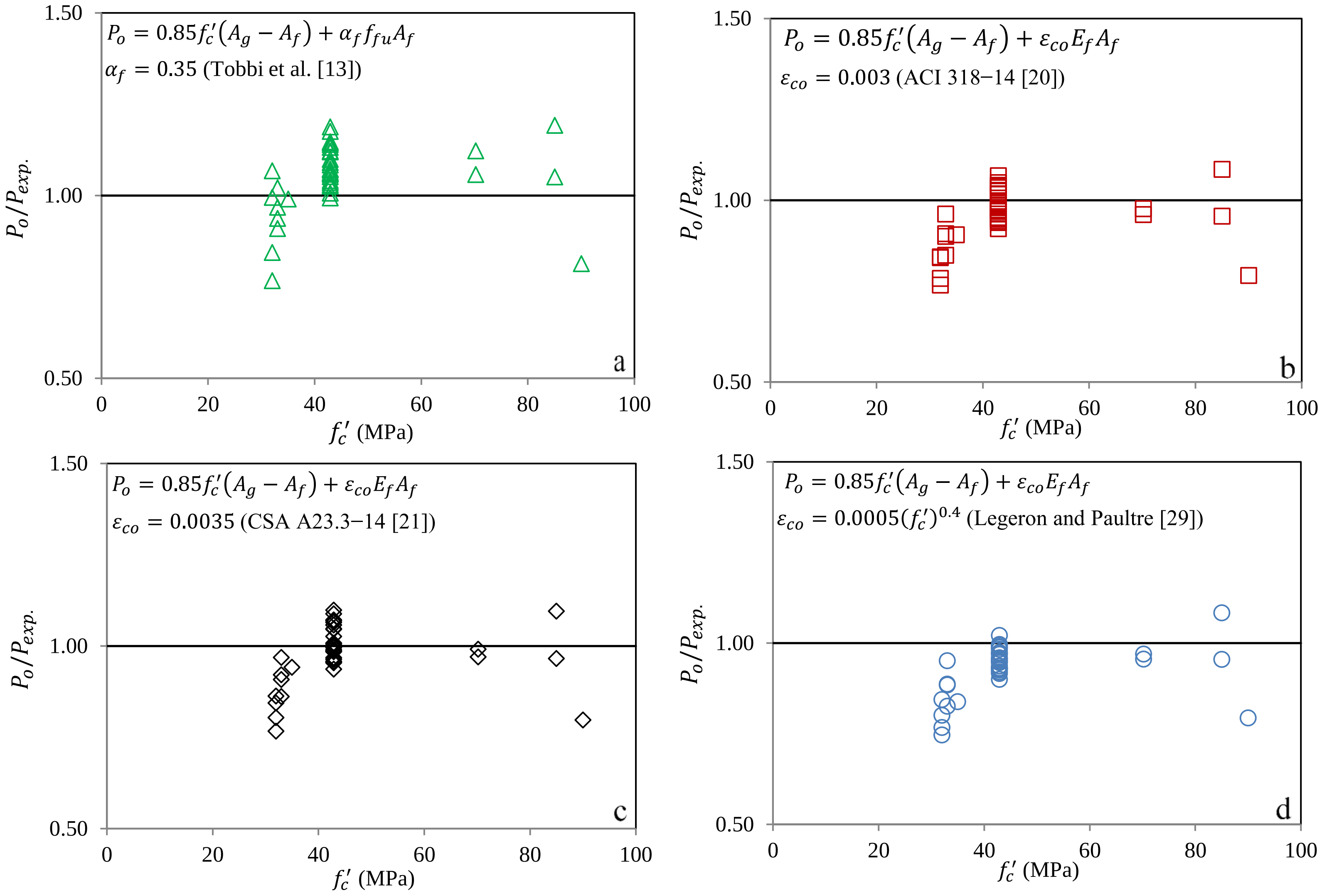

Fig. 2: The relationship between $P_o/P_{exp.}$ of the FRP bar reinforced concrete column and the compressive strength of the concrete f_c'. Note: P_o were obtained using: a) Eq. 6 (α_f = 0.35); b) Eq. 8 (ε_{co} = 0.003); c) Eq. 8 (ε_{co} = 0.0035) and d) Eq. 8 (ε_{co} = $0.005(f_c')^{0.4}$).