EVALUATING FLEXURAL STRENGTH OF RC BEAMS STRENGTHENED BY CFRP USING DIFFERENT ANALYTICAL MODELS

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Abstract

This paper deals with reinforced concrete beams strengthened by CFRP in flexure. The debonding between CFRP and the surface of the beam is the main problem. Many researchers around the world have made extensive efforts to study the phenomenon of debonding for efficient applications. Based on these efforts and different related field applications, code provisions and various models have been proposed for predicting debonding failure. Two code provisions and three typical models are presented in the current study. ACI-440.2R 17, CNR-DT 200 R1/2013, Said and Wu, Lu et al., and Teng et al. have been used to estimate the flexural strength of RC beams strengthened by CFRP with and without grooves. Test results of eleven flexural beams strengthened by CFRP sheet/laminate using externally bonded reinforcement (EBR) and externally bonded reinforcement on grooves (EBROG) methods were used in the current paper. The performance and accuracy of each model were evaluated based on these test results. Most of the prediction models that used in this study give a closer prediction of the flexural strength of beams strengthened by EBR compared to that of the beams strengthened by EBROG. Finally, the prediction results of CNR-DT 200 R1/2013 were the most accurate and approval with test results compared with other models in the current study

Keywords:

CFRP;
 Strengthening;
 Analytical models;
 EBROG;
 Reinforced concrete beam.

1 Introduction

During the past three decades, fiber-reinforced polymer (FRP) has been an increasing interest for using in the strengthening and retrofitting of structural elements. Because of its desired properties such as high tensile strength, low weight, and corrosion resistance [1-3]. Carbon fiber reinforced polymer (CFRP) as externally bonded (EB) is one of the most used techniques to improve the stiffness, strength and structural performance of reinforced concrete (RC) elements. The technique of CFRP external bonding has been confirmed to be an effective and economic solution. Hawileh R. A., et al. [3] conducted an experimental and an analytical study of the flexural behavior of RC beams strengthened with different combinations of fiber reinforced polymer (FRP) using EBR method. The effect of CFRP, GFRP, and hybrid on the load- deflection relationships, strain readings, and failure modes of tested beams have been reported and discussed. Dong J., et al. [4] studied the flexural behavior of RC beams strengthened by external CFRP sheets. The test results showed an increase in the flexural strength of beams strengthened by CFRP sheet ranged by 41 - 125 %.

However, the problem of the debonding CFRP from the surface of the strengthened elements still needs to be overcome in EBR technique. The debonding of CFRP usually occurs before reaching the ultimate capacity of the section. Debonding of the strengthening material may negatively influence its effectiveness and may make such strengthening technique prone to danger due to the high potential of catastrophic failure of the structure.

In order to overcome the problem of CFRP debonding, several attempts have been paid toward exploring alternative strengthening techniques and examining their performance.

Recently, Mostofinejad and Mahmoudabadi [5] have developed Externally Bonded Reinforcement on Grooves (EBROG) as an alternative to externally bonded reinforcement strengthening method.

Several investigations have been conducted to study the effects of shape, number and direction of grooves on the flexure and shear strengths as well as the failure modes of strengthened members. Among these studies, Hosseini A. and Mostofinejad D. [6] carried out an experimental study to assess the effect of groove dimensions (width and depth) on the bond behavior of strengthened specimens by CFRP using EBR and EBROG methods. The ultimate capacity of specimens strengthened by EBROG was increased up to 27.8 % compared with those strengthened by EBR method. Czaderski C., et al., [7] reported that the using of EBROG method leads to increase the load carrying capacity up to 33 % and change the failure mode from debonding to concrete cover separation. Mostofinejad D., et al. [8] experimentally examined the effect of number of CFRP layers (one, two, and three layers) on strengthened beams using EBROG, Externally Bonded Reinforcement in Grooves (EBRIG) and EBR methods. EBRIG method, CFRP is attached directly to the inner surfaces of the grooves as well as the surfaces outside the grooves on the tensile face of the beam. For one layer of CFRP, the performances of three techniques were found to be approximate. While, the beams strengthened with two layers of CFRP using EBROG and EBRIG methods increased up to 25 % compared with one with EBR. For three layers of CFRP using EBROG and EBRIG methods, the increase was 19 % and 24 % respectively. The using of both EBROG and EBRIG methods led to postpone the debonding phenomenon. Sabzi et al. [9] carried out an experimental and analytical study to investigate the behavior of RC beams strengthening by CFRP using EBR and EBROG methods. The results showed that the EBROG method leads to an increase in the ultimate load capacity and ductility of the strengthened elements compared to EBR method. Also, they found that the prediction results of models were in close agreement with test results for beams that were strengthened by EBR method.

A large number of researchers have been proposed models to predict the failure mode and evaluated the existing models of the debonding strength. For example, Lu, X. Z., et al. [10] presented a finite-element (FE) analysis and proposed strength model to investigate the effectiveness and accuracy of theoretical equations to predict the Intermediate crack debonding of beams strengthened by FRP. Both of FE model and simple strength equations are shown to be accurate during comparisons with test results of 42 and 77 beams, respectively. Teng J. G., et al. [11] proposed a model to predict the strength of beams and slabs that fail by Intermediate crack debonding. The proposed model, based on the modification of the bond strength model of Chen and Teng [12] is combined with section analysis. The prediction results of the proposed model is appeared a good agreement with the test results. Kotula P., et al. [13] presented an experimental and numerical study to investigate the anchorage zone of CFRP sheets and lamella using a double tensile test. Kotes, P., et al. [14] the behavior of reinforced concrete beams strengthened by CFRP lamella against bending were studied.

The aim of this study is to assess the accuracy of the theoretical models to predict the flexural strength of RC beams strengthened by CFRP sheet/laminate using EBR and EBROG methods. The performance and accuracy of each model was evaluated based on the experimental results conducted by the authors. The moment capacity equations recommended by the ACI 440.2R-17 [15] and other theoretical models were used to verify the margin of safety provided by the discussed code and to verify its accuracy.

2 Experimental data of existing study

The experimental program is included eleven reinforced concrete beams, all beams were 120 mm · 150 mm in cross-section and 1300 mm in length. Fig. 1 shows the details of beam specimens.

Fig. 1: Details of beam specimens: a) cross section; b) longitudinal section.

Two types of CFRP were used in the current study. The first type is CFRP sheet with 100 mm in width and the second type is CFRP laminate with 50 mm or 100 mm in width. The properties as per the manufacturer are presented in Table 1. The details of flexural strengthening for beams are presented in Table 2. Both types of CFRP are applied to the tension face of the beam surface with a length equal to 1000 mm. All other materials used in the experimental work such as cement, aggregate and steel have been tested according the ASTM Standards.

Table 1: Properties of CFRP.

Material	Property	CFRP-Sheet	CFRP-Laminate
CFRP	Tensile strength [MPa]	4300	2800
	Tensile modulus [GPa]	238	165
	Thickness [mm]	0.131	1.2
	Elongation at break [%]	1.8	1.69

Table 2: Details of flexural strengthening of beam specimens.

Beam specimen	Strengthening type	CFRP type	Number and direction of grooves	Failure mode
BT	EBR	Sheet	-----	Debonding
BTWU	EBR with U anchorage at ends	Sheet	-----	Rupture at anchorage and then debonding
BL1	EBROG	Sheet	Three longitudinal grooves	Concrete cover separation
BL2	EBROG	Sheet	Three longitudinal grooves	Concrete cover separation
BLS1	EBROG	Sheet	Three longitudinal and eight transverse grooves	Rupture of CFRP
BL2S1	EBROG	Sheet	Two longitudinal and eight transverse grooves	Concrete cover separation
BS1	EBROG	Sheet	Only eight transverse grooves in tension side	Concrete cover separation
BT-st	EBR	Laminate (50 mm in width)	-----	Debonding
BL-st1	EBROG	Laminate (50 mm in width)	Two longitudinal grooves	Concrete cover separation
BL-st-S	EBROG	Laminate (100 mm in width)	Three longitudinal grooves	Concrete cover separation
BL-st-SS	EBROG	Laminate (100 mm in width)	Three longitudinal and eight transverse grooves	Concrete cover separation

In this study, two types of grooves longitudinal and transverse have been employed. All longitudinal grooves were cut in length 1000 mm, width 10 mm and depth 7 mm using grinder machine. While, transverse grooves was cut in length 100 mm, width 10 mm and depth 7 mm. Fig. 2 shows the longitudinal and transverse grooves.

Based on the experimental results of existing study, three typical failure patterns of strengthened beams, which observed during the test is presented in Fig. 3. These three failure patterns are termed: a) debonding of CFRP, b) concrete cover separation, and c) CFRP rupture.

Fig. 3: Failure modes of existing study.

The test of existing study has shown that the ultimate capacity of RC beams strengthened by CFRP in flexural is often limited by concrete cover separation. This failure is occurred due to high interfacial stresses nigh the ends of CFRP and then detachment of the CFRP together with a concrete cover.

3 Appraisal of the existing models to predict the flexural strength

3.1 ACI-440.2R 17

In this section, the approach employed by the ACI 440.2R [15] is intended to analyze RC beams strengthened by means of EBR or by NSM FRP reinforcement. In this research, however, the moment capacity of all tested beams has been calculated according to the code equations. Even though some of them are not explicitly among any of the categories above, presence of grooves may make the

strengthened beams be classified in somewhat between EBR beams and NSM beams. Because of FRP materials are linear elastic until failure, the design elasticity modulus for unidirectional FRP can be found by using Hooke's law. The expression of elasticity modulus is given in Eq. (1).

$$E_f = f_{fu} / \varepsilon_{fu} , \quad (1)$$

where E_f is the elasticity modulus of FRP, f_{fu} is the ultimate tensile strength of the FRP, and ε_{fu} is the ultimate rupture strain of FRP reinforcement.

The design approach of flexural strength includes that the design flexural strength of element ϕM_n exceed its required factored moment M_u , as defined in Eq. (2).

$$\phi M_n \geq M_u . \quad (2)$$

This section is intended to verify the accuracy of the analysis and design approach of the ACI 440.2R-17 and to determine the margin of safety provided by the proposed equations. Fig. 4 explains the strain and stress distribution for a rectangular section under flexure at the ultimate limit state. For each individual beam, the nominal moment capacity has been calculated according to the following equations:

$$M_n = A_s f_s \left(d - \frac{\beta_1 c}{2} \right) + \psi_f A_f f_{fd} \left(d_f - \frac{\beta_1 c}{2} \right), \quad (3)$$

where M_n is the bending moment (nominal), A_s and A_f are the areas (cross-sectional) of tensile reinforcement, and FRP sheet/laminate, respectively. f_s is the yield strength in steel reinforcement, and f_{fd} is the design strength of externally bonded FRP reinforcement and can be computed by Eq. (4). c is the neutral axis depth, d and d_f are the effective depth of tensile steel reinforcement and FRP reinforcement. ψ_f is the factor of reduction created by FRP, which is equal to 0.85, this reduction factor is based on the reliability analysis that was dependent on the experimentally measured statistical characteristics of the flexural strength. β_1 is the ratio of depth of equivalent rectangular stress block to the depth of the natural axis. If the controlling failure mode is concrete crushing (after or before yielding of steel reinforcement) β_1 can be taken equivalent to 0.85 for concrete strength f'_c between 17 MPa and 27 MPa. While, if the f'_c exceeding 27 MPa, β_1 should be reduced at rate of 0.05 for each 7 MPa of concrete strength increasing. However, β_1 shall not be taken less than 0.65. If the controlling failure mode is cover delamination, FRP rupture or FRP debonding, the Whitney stress block will give rationally accurate results. f_{fe} can be obtained from equation (5) below

$$f_{fd} = E_f \varepsilon_{fd} , \quad (4)$$

$$f_{fe} = E_f \varepsilon_{fe} , \quad (5)$$

where f_{fe} is effective stress of the FRP (stress obtained at section failure) and ε_{fe} is the effective strain in the FRP reinforcement, as defined in equation (6)

$$\varepsilon_{fe} = \varepsilon_{cu} \left(\frac{d_f - c}{c} \right) - \varepsilon_{bi} \leq \varepsilon_{fd} , \quad (6)$$

ε_{cu} is the ultimate usable strain in concrete (i.e. 0.003), ε_{bi} is the initial stain on the bonded substrate, it can found based on the elastic analysis of the existing element (all loads will be applied on the element during FRP installation). ε_{fd} is the upper limit of the FRP strain at which the debonding failure may occur and can be calculated from equation (7) below

$$\varepsilon_{fd} = 0.41 \sqrt{\frac{f'_c}{n E_f t_f}} \leq 0.9 \varepsilon_{fu} , \quad (7)$$

where f'_c is the compressive strength of concrete, t_f is the thickness of FRP, n is the number of FRP layers and ε_{fu} is the rupture strain of FRP. The Eq. (7) takes a modified formula of the equation of debonding strain proposed by Teng et al. [11, 16]. To find the best-appropriate coefficient of (0.41 in SI) for the proposed equation by Teng, it was calibrated by using average measured values of strains of FRP debonding.

Fig. 4: The strain and stress distribution for a rectangular section under flexure at the ultimate state.

3.2 CNR-DT 200 R1 / 2013

The Italian code CNR-DT 200 R1 / 2013 [17] presents an accurate criterion for design and analysis of members strengthened by FRP using EBR method at the ultimate limit state (ULS). Based on this code, the maximum tensile strain of FRP is calculated by Eq. (8)

$$\varepsilon_{fd} = \min \left\{ \eta_a \frac{\varepsilon_{fk}}{\gamma_f}, \varepsilon_{fdd} \right\}, \quad (8)$$

where ε_{fk} is the FRP characteristic strain at failure (rupture) and can be expressed by Eq. (9), η_a is an environmental coefficient (in this study, equal to 0.95), γ_f is FRP material partial factor (in this study, equal to 1.0). ε_{fdd} is FRP strain at debonding and can be given in Eq. (10).

$$\varepsilon_{fk} = f_{fk} / E_f, \quad (9)$$

$$\varepsilon_{fdd} = \frac{k_q}{\gamma_{f,d}} \sqrt{\frac{2 \cdot k_b \cdot k_{G,2}}{n \cdot E_f \cdot t_f \cdot F_C}} \sqrt{f_{cm} \cdot f_{ctm}}, \quad (10)$$

where f_{fk} is the ultimate tensile strength (provided by manufactory), k_a is a factor that depend on load distributions ($k_a = 1$ for concentrated load and $k_a = 1.25$ for distributed load), γ_{fd} is a safety coefficient equal to 1.0. F_c is the confidence factor equal to 1.0, f_{cm} and f_{ctm} are the concrete compressive and tensile strengths, respectively f_{ctm} can be expressed by Eq. (11). $k_{G,2}$ is a corrective coefficient calibrated from test results and equal to 0.32 and 0.10 mm, respectively (irrespective of the type of reinforcement), k_b is the geometrical corrective factor and can be calculated from the Eq. (12):

$$f_{ctm} = 0.1 f_{cm}, \quad (11)$$

$$k_b = \sqrt{\frac{2 - b_f/b}{1 + b_f/b}} \geq 1 \text{ for } b_f/b \geq 0.25, \quad (12)$$

where, if $b_f/b < 0.25$, $k_b = 1.18$, b_f and b are the width of FRP and beam section respectively. The maximum tensile strength of FRP reinforcement f_{fd} can be calculated by Eq. (13)

$$f_{fd} = f_{deb} = E_f \cdot \varepsilon_{fd}. \quad (13)$$

Based on the following equation can be calculated the maximum tensile moment M_{deb} of FRP reinforcement

$$f_{deb} = \frac{M_{deb} \cdot c}{I}, \quad (14)$$

where I is the moment of inertia of the section.

3.3 Said and Wu model

Said and Wu [18] suggested a simple model to predict the ultimate capacity of CFRP strengthened sections in flexural due to debonding failure. After, the CFRP debonding occurs and the FRP strain at debonding ε_{deb} may be expressed in a general form by the following equation

$$\varepsilon_{deb} = C1(f'_c)^{C2}/(E_f t_f)^{C3}, \quad (15)$$

where $C1$, $C2$ and $C3$ are constants that may be computed base on the available experimental results. By assuming the value of $C2$, the two other constant $C1$ and $C3$ can be calibrated to get the most accurate predictions. The required level of accuracy was occurred by the lowest level of dispersion and the smallest range of predicted to experimental load ratios. The critical value of FRP strain at debonding ε_{deb} can be calculated by the following equation

$$\varepsilon_{deb} = 0.23 (f'_c)^{0.2}/(nE_f t_f)^{0.35}. \quad (16)$$

Based on Eq. (16) and considering that after steel yielding, IC debonding usually occurs, the moment capacity of FRP strengthened members in flexural at debonding can be expressed as follows

$$M_{deb} = A_s f_y y_{ct} + 0.23 b_f (f'_c)^{0.2} (nE_f t_f)^{0.65} (y_{ct} + c^*), \quad (17)$$

where M_{deb} is the bending moment (at debonding), f_y is the yield stress in steel reinforcement, y_{ct} is the distance between the tensile force of steel reinforcement and the compressive force of the compression part of the section. b_f is the width of CFRP, c^* is the distance between the centroid of the tensile steel and the centroid of the FRP. The value of y_{ct} may be carefully computed by applying the equations of strain compatibility and static equilibrium of the section. For quick calculations, rectangular cross sections y_{ct} may be considered equivalent to the effective depth d multiplied by 0.9. So that, Eq. (17) may be rewritten as follows

$$M_{deb} = 0.9 A_s f_y d + 0.23 b_f (f'_c)^{0.2} (nE_f t_f)^{0.65} (0.9d + c^*). \quad (18)$$

For design purposes, a suitable safety factor should be presented. Based on the available experimental results, a reduction factor of strength of 0.75 is recommended. So, Eq. (17) may be modified as follows

$$M_{deb,des} = 0.75 \{ 0.9 A_s f_y d + 0.23 b_f (f'_c)^{0.2} (nE_f t_f)^{0.65} (0.9d + c^*) \}. \quad (19)$$

3.4 Lu et al. model

Lu et al. [10] proposed a model to predict the debonding failure of members strengthened by CFRP in flexural. Based on the stress blocks, the total axial force in the FRP plate at the loaded section is expressed by

$$T = \left(\frac{\tau_{c,max} L_{ee}}{2} + \frac{\tau_{s,max} L_d}{2} \right) b_f, \quad (20)$$

where T is the total axial force of FRP, L_{ee} is the effective bond length corresponding to bond-slip model for the major flexural crack zone, which can be calculated by the Eq. (21). L_d is the distance from the loaded section to the end of the FRP plate. $\tau_{c,max}$ is the maximum value of the crack-induced interfacial shear stress, $\tau_{s,max}$ is the maximum value of the shear force-induced interfacial shear stress, which can be expressed by the equations (22) and (23)

$$L_{ee} = 0.228 \sqrt{E_f t_f}, \quad (21)$$

$$\tau_{s,max} = \alpha \tau_{max}, \quad (22)$$

$$\tau_{c,max} = (1 - \alpha) \tau_{max}, \quad (23)$$

$$\alpha = 3.41 L_{ee}/L_d, \quad (24)$$

where

$$\tau_{max} = 1.5 \beta_w f_t, \quad (25)$$

$$\beta_w = \sqrt{(2.25 - (\frac{b_f}{b})) / (1.25 + (\frac{b_f}{b}))}, \quad (26)$$

$$f_t = 0.53 \sqrt{f'_c}, \quad (27)$$

f_t denotes to the tensile concrete strength, b and b_f are the width of beam and FRP, respectively. According to this model, the debonding strain of FRP ε_f^{IC} can be defined by Eq. (28) below

$$\varepsilon_f^{IC} = \frac{0.114 (4.41 - \alpha) \tau_{max}}{\sqrt{n E_f t_f}}. \quad (28)$$

The maximum tensile strength of FRP reinforcement, f_f can be calculated by Eq. (29)

$$f_f = f_{deb} = E_f \cdot \varepsilon_f^{IC}. \quad (29)$$

Based on the following equation can be calculated the maximum tensile moment M_{deb} of FRP reinforcement

$$f_{deb} = \frac{M_{deb} \cdot c}{I}, \quad (30)$$

where I is the moment of inertia of the section.

3.5 Teng et al. model

Teng et al. [11] proposed a model, based on the effect of concrete compressive strength, axial stiffness of FRP, $nE_f t_f$ and width ratio b_f/b on the debonding failure. In this model, the debonding strain can be expressed by the following equations

$$\varepsilon_{deb} = \begin{cases} 0.48 \beta_w \sqrt{\frac{\sqrt{f'_c}}{n E_f t_f}} & \text{If } L_f \geq \sqrt{\frac{n E_f t_f}{\sqrt{f'_c}}} \\ 0.48 \sin \left\{ \frac{\pi L_f}{2 \sqrt{\frac{n E_f t_f}{\sqrt{f'_c}}}} \right\} \beta_w \sqrt{\frac{n E_f t_f}{\sqrt{f'_c}}} & \text{If } L_f < \sqrt{\frac{n E_f t_f}{\sqrt{f'_c}}} \end{cases}, \quad (31)$$

$$\beta_w = \sqrt{(2 - b_f/b) / (1 + b_f/b)}, \quad (32)$$

where L_f is the distance from the FRP cutoff to the nearest applied load and β_w is the width ratio factor. Based on this model, the ultimate stress for IC debonding σ_{dbic} can be calculated by Eq. (33) below

$$\sigma_{dbic} = 0.48 \beta_p \beta_L \sqrt{\frac{E_f \sqrt{f'_c}}{t_f}}, \quad (33)$$

where β_p and β_L can be expressed by the following equations

$$\beta_p = \sqrt{\frac{2 - b_f/b}{1 + b_f/b}}, \quad (34)$$

$$\beta_L = \begin{cases} 1 & \text{if } L_f \geq L_e \\ \sin \frac{\pi L_f}{2L_e} & \text{if } L_f < L_e \end{cases} \quad (35)$$

in which L_e is the effective bond length and can be defined as follows

$$L_e = \sqrt{\frac{E_f t_f}{f'_c}} \quad (36)$$

Based on the following equation can be calculated the maximum tensile moment M_{deb} of FRP reinforcement

$$\sigma_{dbic} = \frac{M_{deb} \cdot c}{I} \quad (37)$$

4 Comparison of test results with existing models

This section carried out an inclusive comparison among the experimental results P_{Exp} conducted by authors and flexural strength models P_{Pre} in order to check the accuracy and validity of ACI-440.2R 17, CNR-DT 200 R1 / 2013, Said and Wu model, Lu et al. model, and Teng et al. model in estimate flexural strength of RC beams strengthened by CFRP sheet or laminate in presence of grooves. Table 3 summarizes the ratio between prediction load by equations and experimental load P_{Pre}/P_{Exp} of all tested beams. The prediction results that obtained from ACI and Said and Wu models were underestimated results for beams strengthened by CFRP sheet and overestimated for RC beams strengthened by CFRP laminate. The prediction results that obtained based on the models of Teng et al., and Lu et al. have underestimated results for RC beams strengthened by CFRP sheet / laminate and more agreement with those of experimental results compared with ACI and Said models. While, the prediction results of CNR-DT 200 R1/2013 were the most accurate and in agreement with test results compared with other models in current study. From Table 3 can be noted that most of the analytical models used in this study give a closer prediction of the flexural strength of beams strengthened by EBR compared to that of the beams strengthened by EBROG. This may be due to the effect of grooves in EBROG is not taken into consideration in calculations of models.

Table 2: Comparison of results from test and flexural strength models.

No.	Beam	f'_c [N/mm ²]	n	E_f [N/mm ²]	t_f [mm]	b_f [mm]	P_{Exp} [kN]	P_{Pre}/P_{Exp}				
								ACI-440	CNR-DT	Said and Wu	Lu et al.	Teng et al.
1	BT	30.64	1	238000	0.131	100	63.0	0.91	1.03	0.92	0.93	0.84
2	BTWU	30.17	1	238000	0.131	100	64.5	0.88	1.01	0.90	0.91	0.82
3	BL1	31.08	1	238000	0.131	100	70.33	0.82	0.93	0.83	0.84	0.76
4	BL2	31.08	2	238000	0.131	100	73.5	0.67	0.81	1.02	0.75	0.70
5	BLS1	30.64	1	238000	0.131	100	75.1	0.76	0.87	0.78	0.78	0.71
6	BL2S1	30.63	1	238000	0.131	100	70.6	0.81	0.92	0.82	0.83	0.75
7	BS1	30.63	1	238000	0.131	100	63.65	0.90	1.02	0.92	0.92	0.83
8	BT-st	30.17	1	160000	1.2	50	53.04	1.20	0.97	1.44	0.94	0.92
9	BL-st1	29.28	1	160000	1.2	50	56.0	1.13	0.92	1.36	0.89	0.87
10	BL-st-S	29.28	1	160000	1.2	100	56.1	1.70	0.91	2.20	0.87	0.86
11	BL-st-SS	29.28	1	160000	1.2	100	58.5	1.63	0.88	2.11	0.84	0.82

5 Conclusions

In this study, analytical results were presented in order to evaluate the effectiveness and accuracy of the equations used to analysis of RC beams strengthened with CFRP sheets/laminates using the groove method. The moment capacity equations recommended by the ACI 440.2R-17 and other theoretical models were used to verify the margin of safety provided by the discussed code and to verify its accuracy. Based on the results of prediction models, the following conclusions can be noted:

- 1) The results obtained from ACI and Said and Wu models were underestimated results for beams strengthened by CFRP sheet and overestimated for RC beams strengthened by CFRP laminate.
- 2) The results that obtained from the models of Teng et al. and Lu et al. have underestimated results for RC beams strengthened by CFRP sheet / laminate and more agreement with those of test results compared with ACI and Said models.
- 3) CNR-DT 200 R1/2013 results were the most accurate and in agreement with test results compared with other models in current study.
- 4) Most of the analytical models used in this study give a closer prediction of the flexural strength of beams strengthened by EBR compared to that of the beams strengthened by EBROG.

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