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Experimental study on concrete walls for housing rehabilitated externally with CFRP

J. Carrillo & F. Echeverri

Department of Civil Engineering, Universidad Militar Nueva Granada, UMNG, Bogotá, Colombia.

ABSTRACT: Use of low-rise housing made of reinforced concrete walls and using industrialized systems has incremented in Latin America due to its seismic performance and its many economical and sustainable benefits. However, most houses having reinforced concrete walls and built before updating of the earthquake-resistant codes do not meet the new requirements of seismic performance. Aimed at providing a solution to these deficiencies, an experimental study was carried on seismic strengthening of walls using fabrics of Carbon Fiber Reinforced Polymers, CFRP. The experimental program includes 16 panels and 5 walls having web shear reinforcement ratio lower than that prescribed by NSR-10 Building Code, which is equal to 0.20% for housing having three or lower stories, and specimens strengthened externally using three reinforcement ratios and four layouts of CFRP. The paper shows details of the experimental program and discusses the results of a statistical analysis of the capacity computed using the predictive models available in the literature. The analysis demonstrates that the variation between the models is higher than 40%, and that the walls strengthened with the studied CFRP reinforcement ratios and layouts will reach strengths higher than those of the specimen reinforced with the minimum web reinforcement ratio currently prescribed by NSR-10.

1 INTRODUCTION

In recent years, construction of low-rise housing located in all climates and in all seismic hazard zones has increased significantly in Latin America. The low-rise housing made of reinforced concrete walls and using industrialized systems takes a great push every day due to its seismic performance and its many economical and sustainable benefits. However, many houses having reinforced concrete walls that has been built before updating of the earthquake-resistant codes do not meet the new requirements of seismic performance, i.e. the web steel ratio is smaller than the minimum ratio prescribed by NSR-10 (AIS 2012), which is equal to 0.20% for housing having three or lower stories.

For seismic strengthened, it is necessary to find efficient methods to solve structural deficiencies of structures that do not comply with the new seismic codes. Since these structures may have high risk of experimenting inappropriate performance, or suffering severe damage or even collapse during an earthquake. These consequences may occur due to the fact that the code used for the design of the existing dwelling may not have specified requirements to seismic actions, or may have specified requirements but insufficient, or the quality of the original construction has been deteriorated. A research project is being developed in the Nueva Granada Military University for evaluating the performance of four different layouts and three reinforcement ratios of carbon fiber reinforced polymer, CFRP, for strengthening of concrete walls for low-rise housing. The paper describes the experimental program, presents the construction and rehabilitation process of the specimens, compares some of models available in the literature to evaluate the capacity of the strengthened walls, evaluates the effect of the variables included in the experimental program and shows results of a statistical analysis of the predictive models.

2 PREDICTIVE MODELS

Most of research studies on rehabilitation of walls using CFRP refers to masonry walls. In general, information about the rehabilitation of concrete walls rehabilitated with CFRP is scarce. A comparative analysis of the equations proposed by codes ACI 440 (2008), AC 125 (ICBO ES 2001), Bulletin 14 (FIB 2001), and by five models available in the literature for predicting the strength of elements externally

strengthened with CFRP, is presented in this section. Available models were developed from experimental studies reported by Triantafillou & Antonopoulos (2000), Machado (2005), Li *et al.* (2005), Alcaíno & Santa María (2008), and Babaeidarabad *et al*. (2014). Models proposed by the three seismic codes and the model poposed by Triantafillou & Antonopoulos (2000) are applicable for concrete wall. All models here studied consider that the strength of shear wall strengthened with CFRP, V_n, is calculated as the sum of the contributions of the concrete, V_c, or the contribution of the masonry, V_m, the contribution of the steel, V_s, and the contribution of the CFRP, V_f, as described in the Equation (1). According to Carrillo and Alcocer (2013), the contributions of concrete and web shear reinforcement of the walls for low-rise housing is obtained using Equation
(2).

$$V_n = V_c + V_s + V_f \tag{1}$$

$$V_c + V_s = \left[\alpha_1 \cdot \sqrt{f_c'} + \eta_h \cdot \rho_h \cdot f_{yh}\right] \tag{2}$$

where α_1 is the coefficient that defines the relative contribution of concrete to diagonal tension and is calculated with Equation (3), f'_c is the nominal compressive strength of concrete, ρ_h is the web shear reinforcement ratio, η_h represents the efficiency of ρ_h and its value is equal to 0.8 for deformed steel bars and 0.7 for welded wire mesh, f_{yh} is the yield strength of the horizontal web shear reinforcement, and A_w is the wall thickness, t_w, times wall length, l_w. In Equation (3), $M/(Vl_w)$ is the ratio between the bending moment and shear force times wall length.

$$\alpha_1 = 0.21 - 0.02 \cdot \left[\frac{M}{V \cdot l_w}\right] \tag{3}$$

The **Table 1** summarizes the equations of the models analyzed and compared in this study. Echeverri and Carrillo (2013) presents in detail the models used and analyzed.

Table 1. Equations of the predictive models analyzed.

Model	Main equation	Variables
ACI-440	$V_f = \Psi_f\,[A_{fv}\,f_{fe}$ (sen α + cos α) $d_f/s_f]$	$A_{fv} = 2\,n\,t_f\,w_f$
AC-125	$V_f = 2\,t_f\,f_j\,h_w\,\text{sen}^2\alpha$	
FIB	$V_f = 0.9\,\varepsilon_{fd,e}\,E_f\,\rho_f\,t_w\,d_f$ (cot θ + cot α) sen α	$\varepsilon_{fd,e} = k\,\varepsilon_{f,e}/\gamma_f$ $\varepsilon_{f,e} = \min\,[0.65\,\{f'^{2/3}_c/(E_f\,\rho_f)\}^{0.56}\times10^{-3}, \{f'^{2/3}_c/(E\,\rho_f)\}^{0.30}\varepsilon_f]$ $\rho_f = (2\,t_f/t_w)\,(w_f/s_f)$
Triantafillou & Antonopoulos	$V_f = 0.9\,\varepsilon_{f,e}\,E_f\,\rho_f\,t_w\,d_f$ (1 + cot α) sen α	$\varepsilon_{f,e} = [f'^{2/3}_c/(E_f\,\rho_f)]^{0.30}\,\varepsilon_f$
Machado	$V_f = \varphi\,\Omega\,f_{fe}\,t_f\,l_w$ cot α	
Li *et al.*	$V_f = 0.8\,E_f\,\varepsilon_f\,\rho_f\,(t_w\,l_w)$	
Alcaíno & Santa María	$V_f = \Psi_f\,\kappa\,\tau_f\,b_f$ cos α	
Babaeidarabad *et al*.	$V_f = 2\,n\,A_f\,l_w\,f_{fe}$	

In the ACI 440 (2008) model, Ψ_f is the efficiency factor and depends on the strengthening layout; d_f is considered equal to $0.8l_w$, s_f is the spacing between the strips of fibers, f_{fe} is the effective tensile stress in the CFRP. α is the inclination angle of the strips of CFRP with the horizontal, n is the number of layers of CFRP, t_f is the thickness of the CFRP, and, w_f is the width of each CFRP strip. Tensile stress f_{fe} is strain associated to the ultimate tensile strength of CFRP, ε_f, times the modulus of elasticity of the CFRP, E_f.

In the AC-125 (2001) model, f_j is the tangential stress in the CFRP and is equal to 0.004 E_F. In the Bulletin 14 (FIB 2001) model, θ is the angle between principal fiber orientation and longitudinal axis of member, $\varepsilon_{fd,e}$ is the design value for the effective tensile strain of CFRP, k is the factor that relates flexural to tensile strength of concrete, γ_f is a factor that considers the differences observed in the behavior to long term of the CFRP as regard the influence of the application method, $\varepsilon_{f,e}$ is the effective tensile strain of CFRP, and ρ_f is the reinforcement ratio of CFRP.

In the Machado (2005) model, φ is the efficiency factor of the reinforcement method; Ω is the efficiency factor of the location of the fibers and depends on the number of sides strengthened of the wall. In the model proposed by Alcaíno y Santa María (2008), κ is the efficiency factor and depends of the inclination angle of the strips of CFRP with the horizontal, b_f is the total width of the CFRP, and τ_f is the unit shear strength of the CFRP determined experimentally. In the model proposed by Babaeidarabad *et al*. (2014), A_f is the area of CFRP per unit width (mm^2/mm) in both direction.

3 EXPERIMENTAL PROGRAM

The experimental program comprises two phases: the first phase included diagonal tension tests of 16 concrete panels having nominal compressive strength, f'_c, of 26 MPa, height and length (h_w and l_w) of 600 mm, and thickness, t_w, of 75 mm. This phase was aimed at choosing the strengthening layouts that offer better performance to then evaluate, in a second phase, the behavior of such layouts using walls constructed on larger scale and tested under cyclic load reversals. Dimensions of the five concrete walls included in the second phase were $h_w = l_w = 900$ mm and $t_w = 75$ mm. Figure 1 shows walls's geometry.

The panels were tested using a typical setup of a diagonal compression tests. Web reinforcement of panels was designed for shear demands only. Walls were tested in a cantilever configuration. The walls were designed with enough reinforcement at the ends to avoid the bending failure before the typical shear failure observed in low-rise housing. Based on results of the bending design of the walls, steel ratio for bending at the boundary elements of the wall was equal to $\rho_l = 0.8$ %. The longitudinal reinforcement at each boundary elements of the walls was provided using four No. 4 (d_b = 1/2 in = 12.7 mm) deformed steel bars and stirrups No. 2 (d_b = 1/4 in = 6.4 mm) with spacing of 150 mm (4#4, E#2@150 mm). Stirrups were used for assembling the longitudinal reinforcement only; not for confining. The ratio between shear force associated with the bending strength and the web shear of the wall is 1.7, i.e. the bending strength is 70% higher than the shear strength; so it ensures that the wall exhibits shear failure prior to bending failure. Yield stress of the longitudinal reinforcement, f_y, was equal to 420 MPa.

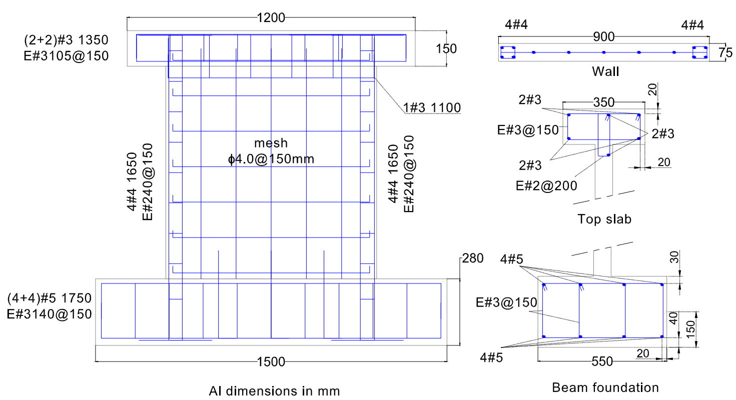


Figure 1. Geometry and internal reinforcement of the walls.

Same steel ratio of horizontal and vertical web shear reinforcement was used in all the specimens, and its value was $\rho_{h,v} = 0.11$ %, which is roughly equivalent to 50% ρ_{min}. Steel ratio $\rho_{min} = 0.20$%, and it is the minimum ratio prescribed by NSR-10 (AIS 2012) for housing having three or lower stories. However, the minimum web shear reinforcement ratio prescribed by NSR-10 (AIS 2012), was used in one panel. The web shear reinforcement of the panels and the walls was provided using a welded wire mesh. For the specimen with 50% ρ_{min}, the diameter and spacing of the vertical and horizontal wires is equal to 4.0 mm (A_s = 12.6 mm^2), and 150 mm, respectively (4.0×4.0-15/15). For the panel with 100% ρ_{min}, spacing of vertical and horizontal wires was 75 mm (4.0×4.0-7.5/7.5). Yield stress of wires used for welded wire mesh was f_{yh} = 500 MPa. The shear friction reinforcement at the base of walls was one No. 3 (d_b = 3/8 in = 9.5 mm) deformed steel bar with spacing of 225 mm (1#3@230 mm).

3.1 Layout of strengthening of the walls

The contribution of CFRP in the models proposed by Bulletin 14 (FIB 2001), Triantafillou & Antonopoulos (2000), and Li *et al.* (2005) is computed with the equations shown in **Table 1**. However, values of ρ_f does not reflect particular layouts as those discussed in this study, because such values changes when the layouts or the specimen dimensions are modified. Therefore, this study proposes to get a volumetric reinforcement ratio of CFRP using Equation (4).

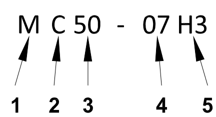
$$\rho_{f-vol} = \frac{Vol_{CFRP}}{Vol_{CON}} \quad (4)$$

where Vol_{CFRP} is the total volume of CFRP in the specimen and Vol_{CON} is the volume of the concrete specimen wrapped. The reinforcement ratio proposed in this study is similar to the volumetric ratio used to determine the confinement effect on concrete columns. Table 2 shows the ratio of CFRP external strengthening.

Initially, 12 panels were strengthened using CFRP reference SikaWrap 300C having thickness t_f = 0.508 mm, and three volumetric reinforcement ratios determined by the Equation (4), $\rho_{f\text{-}vol}$, of 0.07%, 0.17% and 0.27%. As shown in

Figure 2, the four CFRP layout were: one or three horizontal strips, *H1* or *H3*, and one or three 45-degree inclined strips, *D1* or *D3*. Volumetric reinforcement ratios were obtained by varying the width of the strips, w_f, spacing, s_f, and length, L_d. CFRP were wrapped only on one side of the specimens. Aimed at comparing the results with a conventional technique, a panel was rehabilitated conventionally using a one-side overlay of concrete reinforced with 5.0×5.0-15/15 welded-wire mesh equivalent to web steel ratio. Thickness of the overlay, t_{fc}, was equal to 25 mm and the steel ratio of the overlay was ρ = 0.11 %. Therefore, web steel ratio of the rehabilitated (75 mm + 25 mm) wall was ρ = 0.2 %. In addition, two panels were built without external reinforcement; one panel with plain concrete and one panel reinforced with the minimum web shear reinforcement ratio prescribed by NSR-10 (AIS 2012). The unreinforced panel was included for assessing separately the contributions of concrete and steel wire reinforcement. The second panel was built to study the performance of the panel reinforced with ρ_{min}. Before strengthening, the panels were not subjected to any damage level. As discussed earlier, five concrete walls were strengthened in the second phase of the study; four panels were rehabilitated with CFRP and one panel was conventionally rehabilitated with reinforced concrete overlay. The same steel ratio and welded-wire mesh used in panels were used for the rehabilitated wall. For walls strengthened with CFRP, the volumetric reinforcement ratio of $\rho_{f\text{-}vol}$ = 0.17 % was chosen according to the performance observed in the tests of the panels.

Figure 2 shows the layout of the external strengthening of the walls. The specimens were named according to the aspect ratio, h_w/l_w, the internal and external reinforcement, and the layout of the CFRP. The following labeling system was used for the specimens.



where 1 indicates the type of specimen (M = Wall, P = Panel), 2 indicates the aspect ratio of the specimen h_w/l_w (C = Square, h_w/l_w = 1), 3 indicates the percentage of the minimum web shear reinforcement ratio prescribed by NSR-10 (AIS 2012) (100 = ρ_{min}, 50 = 0.5×ρ_{min}). If this letter is omitted, it indicates that plain concrete is used in the specimen (no web shear reinforcement). Number 4 indicates the CFRP reinforcement ratio (07 = 0.07%, 17 = 0.17%, 27 = 0.27%). Number 5 indicates the layout of the CFRP reinforcement; i.e. *H1* is the horizontal layout with a single strip at the height of the specimen, *H3* is the horizontal layout with three strips at the height of the specimen, *D1* is the 45-degree inclined layout with a strip along the diagonals of the specimen, and *D3* is the 45-degree with three strips along the diagonals. If variables located at numbers 4 and 5 are omitted, it indicates that the specimen are not reinforced externally. In addition, if letters CR are included, it indicates that the specimen is reinforced externally through concrete overlay reinforced with steel ratio of 0.11% and made of welded-wire mesh. Table 2 shows the general characteristics of the specimens. In the table, d_f is the distance from the extreme fiber in compression of concrete to the resultant of the forces of all the tension reinforcement, and is considered equal to $0.8l_w$, and A_{fv} is the area of the CFRP strengthening with spacing s_f. The area A_{fv} is calculated according to ACI 440 (2008) model (see **Table 1**).

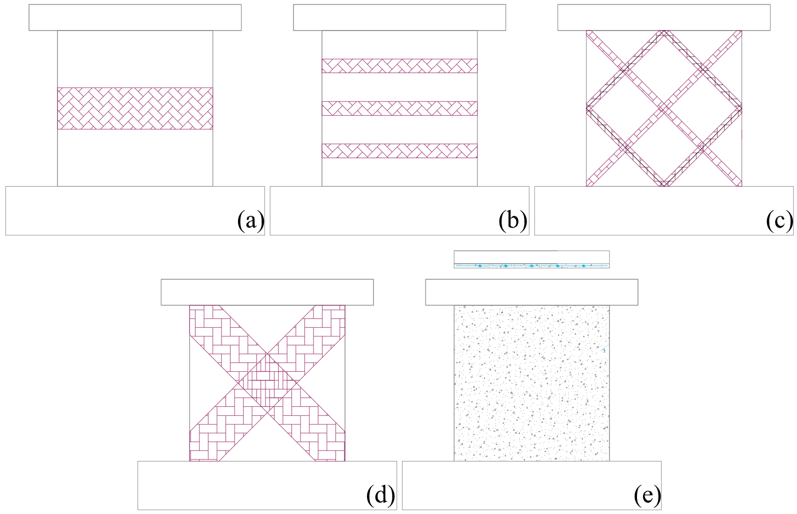


Figure 2. Layout of the external reinforcement: (a) MC50-17H1, (b) MC50- 17H3, (c) MC50-17D1, (d) MC50-17D3, (e) MC50-CR.

3.2 Mechanical properties of materials

The specimens were reinforced externally using two types of reinforcement: CFRP strips and concrete overlay reinforced with welded-wire mesh. Table 3 shows the nominal mechanical properties of the materials used in the study, where ε_c is the maximum usable strain at extreme concrete compression fiber, E_c is the modulus of elasticity of concrete, f'_c is the nominal compressive strength of concrete, f_{yh} is the yield stress of the steel reinforcement, ε_f is the tensile strain associated with the failure of carbon fibers, E_f is the modulus of elasticity of the CFRP, and f_f is the tensile strength of the carbon fibers.

Table 2. General characteristics of specimens.

Model	t_{fc} mm	w_f mm	s_f mm	L_d mm	d_f mm	A_{fv} mm^2	α °	ρ^* %	$\rho_{f\text{-}vol}^{**}$ %
PC					480				
PC50					480				
PC100					480				
PC50-07H1		60	600	600	480	36000	0	0.14	0.07
PC50-07H3		20	200	600	480	36000	0	0.14	0.07
PC50-07 D1		11	212	3394	480	36730	45	0.07	0.07
PC50-07D3		21	849	1697	480	35197	45	0.03	0.07
PC50-17H1		150	600	600	480	90000	0	0.34	0.17
PC50-17H3		50	200	600	480	90000	0	0.34	0.17
PC50-17 D1		27	212	3394	480	87996	45	0.17	0.17
PC50-17D3		55	849	1697	480	90313	45	0.09	0.17
PC50-27H1		240	600	600	480	144000	0	0.54	0.27
PC50-27H3		80	200	600	480	144000	0	0.54	0.27
PC50-27D1		45	212	3394	480	142610	45	0.29	0.27
PC50-27D3		90	849	1697	480	144635	45	0.14	0.27
PC50-CR	25				480				
MC50-CR	25				720				
MC50-17H1		225	225	900	720	202500	0	1.35	0.17
MC50-17H3		75	75	900	720	67500	0	1.35	0.17
MC50-17D1		41	82	5091	720	200333	45	0.68	0.17
MC50-17D3		82	318	2546	720	202014	45	0.35	0.17

* ρ_f is the reinforcement ratio of CFRP used in the models proposed by Bulletin 14 (FIB 2001), Triantafillou & Antonopoulos (2000) and Li *et al.* (2005). ** $\rho_{f\text{-}vol}$ is the volumetric reinforcement ratio of CFRP proposed by this study.

Table 3. Mechanical properties of materials.

Material	f'_c MPa	f_{yh} MPa	E_c MPa	E_f MPa	ε_c %	ε_f %	f_f MPa
Reinforced concrete	26	500	19886	--	0.3	-	-
Reinforced concrete overlay	35	500	23073	-	0.3	-	-
CFRP	-	-	-	51724	-	1.5	4200

3.3 Construction and curing of specimens

Casting of the concrete panels were carried out in three layers. Form vibration was applied through a rubber hammer only to assure that air bubbles were removed. When removing the form, a membrane was used for curing the concrete specimens. A high strength grout was used to fill small-size hollows observed after removing the form.

Recommendations of the manufacturer were followed for bonding carbon fibers to concrete surface. Initially, surface of panels where the CFRP would be bonded was identified, cleaned and prepared. To assure that CFRP develops the specified tensile mechanical properties, the manufacturer recommends that the minimum pull-off strength of a coating system from concrete must be 1.5 MPa. Pull-off of coatings from concrete was verified using a concrete cylinder based on the recommendations of ASTM-D-4541 (1995). Figure 3 shows the test setup. The general pull-off test is performed by securing a loading fixture (dolly, stud) perpendicular to the surface of the coating with an adhesive. After the adhesive is cured, a testing apparatus is attached to the loading fixture and aligned to apply tension normal to the test surface. The force applied to the loading fixture is then gradually increased and monitored until either a plug of material is detached, or a specified value is reached. When a plug of material is detached, the exposed surface represents the plane of limiting strength within the system. The nature of the failure is qualified in accordance with the percent of adhesive and cohesive failures, and the actual interfaces and layers involved. The results of the pull-off test demonstrates that the pull-off strength of a coating system from concrete is higher than 1.5 MPa, with mean equal to 4.32 MPa and coefficient of variation, *CV*, equal to 17%.

Figure 3. Pull-off test.

Bonding of carbon fibers to concrete surface of specimens was conducted after a training course provided by Sika. Figure 4 shows some pictures of the process before the rehabilitation with CFRP. A particular anchoraging method of fibers to concrete was avoided because (i) Tan *et al.* (2003) demonstrated that the behavior of CFRP fibers bonded on rough surface is sufficiently suitable for the expected behavior of concrete walls for low-rise housing, (ii) carbon fibers are difficult to anchorage in walls with difficult access to the edges (corner walls), and (iii) anchoring of CFRP augments the cost of strengthening.

Figure 4. Constructive process: retrofit of concrete voids, surface preparation and bonding of CFRP.

4 RESULTS AND DISCUSSION

This section shows the results of comparing models available in the literature to evaluate the capacity of the strengthening walls and results of evaluating the effect of the variables included in the experimental program. The comparison and analysis of each predictive model are made in terms of the ratio of the contribution of the CFRP, V_f, and the total shear strength of the wall, V_n. The comparison only includes the specimens reinforced externally with CFRP. Therefore, specimens PC and PC50 are not included in the analysis because they were not reinforced externally, and specimens PC50-CR and MC50-CR were reinforced externally with concrete overlay.

Figure 5 shows a box and whisker chart to analyze statistically the results of the predictive models in terms of V_f/V_n. Results of box and whisker chart allow to reveal the following observations:

- According to ACI 440 (2008) and Machado (2005) models, the performance of panel rehabilitated with $\rho_{f\text{-}vol}$ = 0.07% and using a horizontal strip (PC50-07H1), does not match the performance of the panel conventionally reinforced with ρ_{min}. Also, the results of Alcaíno & Santa María (2008) model infers that any proposed CFRP reinforcement ratio is sufficient to match the performance of the the conventionally reinforced specimen with the minimum web shear reinforcement.
- Using the predictive models analyzed, the average contribution of the CFRP to total shear strength varies between 31% and 56%. Additionally, the variation between the models is very high (greater than 40%), which indicates that there are substantial differences and scatter on the prediction of the contribution of CFRP to total shear.
- Highest contribution of CFRP to total shear strength is obtained with Li *et al.* (2005) and Babaeidarabad *et al*. (2014) models, varying between V_f/V_n 56% and 76%. On the other hand, the lowest contribution of CFRP to total shear strength is obtained with Alcaíno & Santa María (2008) model, showing a V_f/V_n value of 12%. Alcaíno & Santa María (2008) model was calibrated for masonry walls. Althoug the value of masonry properties were replaced by concrete properties, characteristics such as bond between the carbon fiber and masonry/concrete affect the results, but they are not included in the model.
- The predictive model that exhibits the lowest coefficient of variation is the model of AC 125 (ICBO ES 2001), showing a *CV* equal to 12%. On the other hand, Machado (2005) model exhibits the higher coefficient of variation (*CV* = 104%). Machado model does not apply when CFRP is bonded completely horizontal or completely vertical.

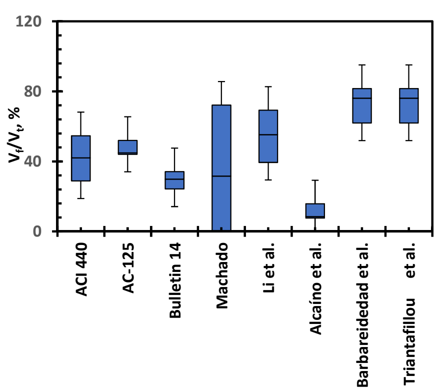


Figure 5. Box and whisker chart for the evaluation of the prediction.

5 CONCLUSIONS

Details of the experimental program and results of the statistical analysis of prediction of seismic performance of concrete walls for low-rise housing rehabilitated with CFRP, was presented in the paper.

The statistical analysis demonstrated that the variation of the fiber contribution in the prediction models is greater than 40%. It indicates that there are substantial differences and scatter on the prediction of the contribution of CFRP to shear strength of walls.

In terms of the variables included in the predictive models, it was observed that not all the models found in the literature include the orientation of the fibers completely parallel or completely perpendicular to the specimen axis. Furthermore, the models that include a CFRP reinforcement ratio do not include layouts that are effective but are not common, as the layouts of 3 bands that cross on their diagonals.

According to the prediction of the models available, the contribution of CFRP to total shear is greater than 30%. The AC 125 (ICBO ES 2001) model exhibits the lowest coefficient of variation to modify the studied variables. However, the ACI 440 (2008) model will be used to estimate the specimens strength before tests, because: (i) it is proposed for strengthening of reinforced concrete structures, (ii) it exhibited (among the studied codes) the intermediate value of the average of the CFRP contribution, (iii) it exibited the major contributions of the CFRP, and thus, it will be possible to define the maximum capacity for testing actuators, (iv) it is the base document of the software used by the company that supplied the CFRP.

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