Shear strength assessment of FRP pre-tensioned concrete beams

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ABSTRACT: The low susceptibility to corrosion of fiber reinforced composite materials (FRP) has led to considering their use for prestressing reinforcement, as an alternative to steel. However, the effects of FRP fragility, low elastic modulus, and low bond with concrete, when compared to usual prestressing steel, must be accounted for in the design of structures. This paper presents a theoretical model for predicting the shear strength of FRP prestressed pre-tensioned concrete beams, extending a previously developed model (CCCM), to account for the properties of FRP as active reinforcement, and to capture reductions in shear strength caused by bond loss in the FRP tendons. The model has been validated by comparing the theoretical predictions with the results of shear tests available in the literature. Good accuracy has been obtained in predicting the shear capacity and also in identifying those cases where shear failure is associated to loss of anchorage bond.

1 INTRODUCTION

Fibre reinforced polymers might be a suitable alternative to conventional steel reinforcement to avoid corrosion of prestressing reinforcement, which is one of the main causes of deterioration of concrete structures. Although the use of FRP as passive reinforcement has been studied in depth during the last decades, the level of knowledge of FRP prestressed concrete structures is much lower, and there are few design guidelines or recommendations.

Up to date, few analytical models have been proposed to assess the ultimate shear strength of FRP prestressed beams. Kueres et al. (2020) and Küres (2019) extended a variable strut inclination model to determine the shear strength of non-prestressed FRP reinforced beams with shear reinforcement to consider the beneficial effects of prestressing on the shear strength. ACI440.4R-04 (ACI 2004), CSA S806-12 (CSA 2012) and JSCE (Astuhiko Machida 1997) consider that the ultimate strength is provided by the concrete, by the shear reinforcement and by the beneficial vertical component given by the prestressing force, with different provisions for each component.

The aim of this paper is to present an extension of the Compression Chord Capacity Model (CCCM) (Cladera et al. 2016), previously developed by the authors together with Cladera and Ribas from the University of Balearic Islands (UIB), to predict the shear strength of pre-stressed concrete beams with FRP tendons and FRP passive shear reinforcement. This model includes the effects of loss of anchorage of the FRP tendons. The performance of this model is assessed by comparing the theoretical predictions for the ultimate shear strength to the experimental ultimate values of 43 FRP pre-tensioned beams.

2 DESCRIPTION OF THE DEVELOPED SHEAR STRENGTH MODEL FOR CONCRETE MEMBERS PRESTRESSED WITH FRP TENDONS

2.1 The Compression Chord Capacity Model for steel reinforced and prestressed concrete members

It is generally accepted that the shear strength in a RC beam is provided by the contribution of the un-cracked concrete chord \( V_{cc} \), the shear transferred along the cracks, \( V_{cw} \), due to
aggregate interlock and the residual tensile forces transferred across the crack, the shear resisted directly by the transverse reinforcement, $V_s$, crossing the critical crack, and the shear resisted by the longitudinal reinforcement, $V_l$, so called dowel action. However, the relative contribution of each shear transfer action change along the loading process since, as the loading and the critical shear crack width increase, the shear transferred by aggregate interlock and the residual stresses crossing the crack tend to reduce as well as the dowel action. Then, shear stresses redistribute, increasing in the region of the uncracked compression chord; which becomes the most relevant shear transfer action until failure takes place under a biaxial stress state. To account for this experimental evidence, the Multi-action Shear Model (Mari et al. 2014) which included the four shear transfer actions, was simplified, into the Compression Chord Capacity Model (CCCM) (Cladera et al. 2016). In this model, the critical shear crack typically involves two branches, as shown in Figure 1. The opening of the first branch of the critical crack is assumed to be the result of a rotation around its tip. The compression zone above the tip prevents any meaningful contribution of shear slip along the crack interface, thus the contributions of aggregate interlock and dowel action are small, considered as constant values and incorporated into the contribution of the uncracked concrete chord.

The most relevant assumptions of the CCCM are:
- The shear stresses are concentrated in the uncracked compression chord, concrete tensile strength is neglected and the normal compressive concrete stresses distribution is linear.
- The critical shear crack starts at a section where at shear failure, the bending moment reaches the cracking moment of the section. The horizontal projection of the critical shear crack is 0.85$d$, based on experimental observations.
- The weakest section subjected to a combined shear-bending failure is considered to be located at the tip of the first branch of the critical crack, where it reaches the flexural neutral axis.
- Failure is assumed to take place when the pair of principal stresses reach the Kupfer’s Biaxial Failure Envelope (Kupfer & Gerstle 1973).
- For usual values of $a/d = M/(V·d)$, the critical point inside the compression chord, is located at a vertical distance from the neutral axis $z = 0.425c$, being $c$ the neutral axis depth.

The key equation to obtain the contribution of the concrete chord $V_c$, is the following:

$$V_c = \int_0^c \tau(z)b(z)dz = K \zeta b c \sigma_1 \sqrt{1 - \frac{\sigma_x + \sigma_y}{\sigma_1} + \frac{\sigma_x \sigma_y}{\sigma_1^2}}$$

where $\tau(z) = \text{shear stress at a point of the uncracked chord at a distance } z \text{ of the neutral axis; } c = \text{neutral axis depth given by Eq (11b) of Table 1; } b = \text{rectangular section width; } \sigma_1 = \text{principal tensile stress; } \sigma_x = \text{normal longitudinal stress due to bending; } \sigma_y = \text{normal vertical stress; } K = \text{integration constant of value 0.682, and } \zeta = \text{size effect factor given by Eq. (10) of Table 1.}$

Relating the normal stresses with the internal forces, and setting the equilibrium conditions Eq. (1) can be expressed as a function of the internal forces and the components of the shear
transfer actions, see Figure 2. Once solved, Eq (1) provides a solution in which the contribution $V_c$ is almost a linear function of $c/d$, including the effects of the confining stresses in the concrete generated by the stirrups.

![Figure 2](image.png)

Figure 2. Internal forces and stress resultants considered in the shaded free body equilibrium.

The CCCM shear model, initially derived for reinforced concrete members with rectangular cross section, was extended to T sections, in which a non-negligible contribution of the flanges to the shear strength exists. This and other effects produced by the differences in section shape were incorporated by using an effective shear flanges width $b_{v,eff}$ as detailed in Cladera et al. (2016).

The CCCM was also extended to incorporate the effects of longitudinal prestressing of a beam in the shear strength (Mari et al. 2015). The increment of the neutral axis depth due to the axial compressive force, the reduction of inclination angle of the cracks and the increment of the cracking moment and, therefore, of the position of the critical shear crack were accounted for. For inclined tendons, the vertical component of the prestressing force $V_p = P \cdot \sin(\alpha)$, is deducted from the shear force due to external actions.

For simplicity reasons, the effects of confinement of the concrete chord due to stirrups, initially included into $V_c$, were incorporated into $V_s$. These changes and extensions gave place to the following expressions for the concrete and steel reinforcement contributions to the shear strength in reinforced and prestressed concrete beams with rectangular T and I section, in which, for design purposes, the tensile strength was replaced by an equivalent term in terms of the concrete compressive strength:

$$ V_{cu} = 0.3' \frac{c}{d} f_{cd}^{2/3} b_{v,eff} d \neq V_{cu,\text{min}} = 0.25 \left( \zeta K_c + \frac{20}{d_0} \right) f_{cd}^{2/3} b_w d $$

$$ V_{su} = 1.4 \frac{A_{yw}}{s} f_{yw}(d_s - x) \sin(\alpha) \left( \cot \theta + \cot a \right) $$

where $K_c = c/d \geq 0.2$; $d_0 = d \geq 100$ mm; and $b_{v,eff}$ = shear effective web width, which accounts for the flanges contribution, given by Eqs. (12a) and (12b) of Table 1. The neutral axis depth in prestressed concrete sections $c$ is approached by a linear interpolation between the state with zero prestressing ($c = c_0$) and the decompression state ($c = h$), given by Eq (11a) in Table 1.

2.2 Extension of CCCM for beams reinforced with FRP bars as prestressing and transverse reinforcement, without anchorage loss of prestressing reinforcement

Due to the elastic-brittle nature of FRP, the contribution of the shear reinforcement to the shear strength must be modified. As no yielding occurs in FRP reinforcement, the stirrups behave elastically until failure and their individual stress is not constant along the shear crack, but it depends on their deformation. Therefore, the vertical shear force provided by the
stirrups ($V_f$) can be estimated using an average value of their stress along the critical crack (Oller et al. 2015), resulting for vertical stirrups ($\alpha$=90°):

$$V_f = 0.85 \frac{A_{fw}}{s} \sigma_{fw,m} d_s (1 + \Delta V_{cu}) = 0.85 \rho_{fw} E_f \varepsilon_{fw,m} d_s (1 + \Delta V_{cu})$$

(4)

where $A_{fw}/s$ is area of FRP stirrups per unit length of the beam; $\sigma_{fw,m}$ is mean stress of the stirrups at shear failure, $\varepsilon_{fw,m}$ is mean strain of the stirrups at shear failure; $E_f$ is modulus of elasticity of the FRP bars; and $\Delta V_{cu}$ is effect of confinement due to stirrups given by Eq. (14) of Table 1.

The mean strain of the FRP stirrups at failure ($\varepsilon_{fw,u}$) is obtained assuming a linear variation of the stirrups strains along the crack length (Figure 3), as recommended by Oller et al. (2015). The stirrup that fails is assumed to develop a strain of 0.45 $\varepsilon_{fw,u}$, where the 0.45 factor is derived from the strength limit of bent FRP bars proposed in JSCE (Astuhiko Machida 1997) assuming a bent radius of 3 bar diameters and $\varepsilon_{fw,u}$ is the ultimate strain of the FRP stirrup. This results in an average stirrup strain along the shear crack of 0.225 $\varepsilon_{fw,u}$. As a result, the shear force resisted by the stirrups in a length equal to the horizontal projection of the crack until the neutral axis, 0.85 $d_s$, is:

$$V_f = 0.85 \rho_{fw} 0.225 \varepsilon_{fw,u} d_s (1 + \Delta V_{cu}) = 0.191 \rho_{fw} E_f \varepsilon_{fw,u} d_s (1 + \Delta V_{cu})$$

(5)

Table 1 summarizes the proposed formulation for predicting the shear strength of prestressed concrete beams with or without FRP stirrups, assuming that no anchorage failure takes place.

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<tr>
<th>Strength parameter</th>
<th>Equation</th>
<th>Eq. #</th>
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<td>Shear strength</td>
<td>$V_{rd} = V_{cu} + V_f \leq V_{rd,max}$</td>
<td>(6)</td>
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<td>Shear strength limit (diagonal compression)</td>
<td>$V_{rd,max} = \alpha_{cu} b_n 2\nu_{vf} f_{cd} \frac{c_{min}}{T_v \cot \theta}$</td>
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<td>Concrete contribution</td>
<td>$V_{cu} = 0.3 \frac{c}{h} K_w d_{bf} d_{bf}$</td>
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<td>Shear reinforcement contribution</td>
<td>$V_{frp} = 0.191 \rho_{fw} E_f \varepsilon_{fw,str} d_s (1 + \Delta V_{cu})$</td>
<td>(9)</td>
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<td>Size and slenderness effect</td>
<td>$\zeta = \frac{2}{1 + \frac{\rho_d}{\rho_p}}$, $d_0 = d \geq 100$ mm</td>
<td>(10)</td>
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<td>Relative neutral axis depth</td>
<td>$\zeta = \frac{1}{2} + (\frac{\rho_d}{\rho_p}) \frac{\sigma_{eff}}{\sigma_{cub}}$; $A_c = \text{Section Area}$</td>
<td>(11a)</td>
</tr>
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<td>Effective flange width</td>
<td>$b_{ef} = b_n = (1 + \sqrt{1 + \frac{2}{3}\frac{\rho_d}{\rho_p}}) b_n + 2h_f \leq b$</td>
<td>(11b)</td>
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<td>Crack inclination</td>
<td>$\cot \theta = \frac{0.85 l_{av}}{a_2 - c} \leq 2.5$</td>
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<td>Stirrup confinement factor</td>
<td>$\Delta V_{cu} = 0.5 \zeta \frac{c}{d} \left(1 + \frac{b_n}{b} \right) \frac{b_{ef}}{b}$</td>
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2.3 Shear strength considering anchorage loss of FRP prestressing bars (shear-bond failure)

As evidenced by experimental tests, premature shear failures may occur due to slip of the prestressing tendons at the critical shear crack. This type of failure occurs when the available bonded length between the critical shear crack and the free end of the beam ($l_{av}$ in Figure 4a) is smaller than the anchorage length ($l_{req}$) required to develop the tensile force of the prestressing reinforcement at the cracked section. The shear-bond failure mode can be considered by computing $l_{av}$ and $l_{req}$ based on the position of the critical shear crack.

The available bonded length is equal to the sum of the beam offset ($e$) and the distance between the centre of the support and the critical crack ($s_{st}$):

$$l_{av} = e + s_{st} = e + \frac{M_{cr}}{V_u}$$

(15)
The required anchorage length can be computed based on equilibrium of the tensile force and bond forces acting on the FRP reinforcement:

$$l_{req} = \frac{F_p}{u \cdot \tau_{max}}$$  \hspace{1cm} (16)

where $F_p$ = tensile force of prestressing reinforcement at the critical crack; $\tau_{max}$ = average bond strength along the anchorage length; and $u$ = total perimeter of prestressing reinforcement. The value of $F_p$ is obtained by studying moment equilibrium at the critical crack. Based on the moment equilibrium at point O of the free body diagram shown in Figure 4b:

$$F_p = \frac{M_{cr} + V_c \cdot \beta \cdot d + 0.5V_t \cdot \beta \cdot d}{z} = \frac{M_{cr} + V_c \cdot 0.85 \cdot d + 0.5V_t \cdot 0.425 \cdot d}{z}$$  \hspace{1cm} (17)

Assuming no bond failure will occur, the shear strength and critical section can be initially estimated using the equations in Table 1. The available bonded length and required anchorage length are then calculated using Eqs. (15) and (16), respectively, given the position of the critical section. If $l_{av} \geq l_{req}$, the prestressing reinforcement is capable of developing the tension force at the shear crack (no bond loss) and the development of the full shear strength is confirmed. If $l_{av} < l_{req}$, a bond failure will occur prior to reaching the required tension force at this critical section, which implies that a reduced prestressing force ($P_{red}$) is acting at the instant of failure. The reduction of the prestressing force results in a smaller shear strength and a shorter distance to the critical section ($s_{cr}$). Bond failure has to occur for the value of $P_{red}$ that satisfies $l_{av} = l_{req}$. The shear strength for bond loss ($V_{Rd,B}$) is obtained using the equations in Table 1 for $P = P_{red}$. The values of $P_{red}$ and $V_{Rd,B}$ are obtained by iteration since both $l_{av}$ and $l_{req}$ depend nonlinearly on $P$. 

Figure 3. Idealized tensile stress distribution for stirrups crossing the critical shear crack.

Figure 4. Available bonded length and forces acting in critical section.
3 VERIFICATION OF THE PROPOSED MODELS

A database of shear tests on FRP prestressed concrete beams reported in the literature has been compiled in order to assess the accuracy of the proposed model. The database includes 43 tests on FRP pre-tensioned beams, with and without FRP shear reinforcement. The tests correspond to studies conducted by Grace et al. (2015), Kueres et al. (2020), Nabipay & Svečova (2014), Park & Naaman (1999), Preinstorfer et al. (2021), and Whitehead & Ibell (2005). Table 2 provides the list of beam tests included in the database and their key geometrical and reinforcement characteristics. As shown, the beams present a variety of types of FRP as prestressing and shear reinforcement (aramid, glass, carbon, parafil), shear span ratios \((a/d)\) ranging from 1.5 to 6), prestressing stress levels \((\sigma_{cp}/f_c)\) ranging from about 2% to 20%) and amounts of transverse reinforcement \((\rho_{tr})\) ranging from 0 to 1.5%).

The shear strength of the beams in the database has been estimated using the model presented in Section 2. The shear-bond failure mode has been considered only for the nine beam tests by Kueres et al. (2020), since this was the only study providing bond strength data for the FRP bars and in which shear-bond failures were actually observed in beam tests. In addition to the shear failure modes associated to the proposed model, potential shear failures in flexurally uncracked regions have been considered following the equations of Eurocode 2 (European Committee for Standardization 1992).

Table 2 presents the results of the analytical predictions, including the predicted shear strength \(V_{\text{pred}}\) and failure mode, as well as the experimental-to-analytical strength ratios \(V_{\exp}/V_{\text{pred}}\). Figure 5 compares the experimental and analytical strengths, and presents statistics of the \(V_{\exp}/V_{\text{pred}}\) ratios. The average value of \(V_{\exp}/V_{\text{pred}}\) is 1.08 and its coefficient of variation (C.V.) is 23%. Hence, the proposed model provides an overall good match with the experimental results. The level of accuracy of the model is similar to that obtained with the original CCCM formulation for steel reinforced and prestressed concrete presented in Cladera et al. (2016), which provided an average of 1.17 and a C.V. of 19%.

Regarding the shear-bond failure mode, specimens LV-1-1, LV-1-2, LV-2-1, LV-2-2 and LV-4 of Kueres et al. (2020) exhibited this type of failure during testing. This type of failure was correctly predicted for the first four specimens with an average value of the strength ratio of 1.03, as depicted from Table 2. The bond failure observed in the fifth specimen (LV-4) was correctly identified analytically, but the strength prediction is very close to the experimental value.

![Figure 5. Experimental vs. predicted shear strengths.](image)

4 CONCLUSIONS

A mechanically-based theoretical model to predict the shear strength of prestressed concrete beams with FRP reinforcement has been proposed. The model is an extension of the Compression Chord Capacity Model (CCCM) formulation, which has been modified to account for the brittle nature of FRP transverse reinforcement and potential bond loss of prestressing reinforcement. The accuracy of the model has been verified using a database of 43 shear tests from the literature. The analytical strength estimations are in good agreement with experimental data,
providing an average experimental-to-predicted strength ratio of 1.08 and a coefficient of variation of 23%. The model has been also shown to identify shear-bond failures in four of the five cases in which this type of failure was reported experimentally. With the appropriate safety factors, the proposed shear strength formulation can be, therefore, employed to efficiently design and assess for shear resistance FRP-prestressed concrete elements.

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* Predicted failure modes: failure of compression chord (CC), shear-bond (B), diagonal compression (DC), and shear failure in uncracked regions (U).

** Test specimens exhibiting shear-bond failures
ACKNOWLEDGEMENTS

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REFERENCES


Küres, S., 2019. Analysis and Design of Concrete Beams with Pre-tensioned CFRP Reinforcement. Dr. Diss. RWTH Aachen.


