Crack parameters in normal section of FRP strengthened RC elements

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Abstract. Contrary to existing studies, this paper presents a prediction model of crack parameters in normal section of FRP strengthened RC elements neglecting crack spacing. A relation between normal crack width, depth and strains in the level of FRP reinforcement, established by Slaitas et al. (Slaitas, Daugevičius, Valivonis, & Grigorjeva, 2018a) and Jokūbaitis et al. (Jokūbaitis & Juknevičius, 2013; Jokūbaitis, Juknevičius, & Šalna, 2013), allowed authors to describe the full development of the crack up to the element failure. Numerical results are compared with experimental ones from the tests of 9 RC beams, strengthened with externally bonded and near surface mounted FRP reinforcements.

Keywords: reinforced concrete, FRP, crack width.

Introduction

The use of FRP materials for retrofitting of existing reinforced concrete structures has many advantages comparing to classical methods. FRP materials are lighter and easier to install, what leads to the reduction of labour cost, they are thin and more resistant to corrosion than for instance steel. But to use all the potential of this material, external reinforcement should deform up to 1.6%, in this case high plastic deformations will occur in the element. In accordance with design provisions (EN 1992-1-1:2004; Fib bulletin 14, 2001) the cracks in concrete open when a limiting tensile strain of concrete is reached, therefore crack width can be calculated by multiplying the mean values of a difference between tensile reinforcement and concrete strains with crack spacing, what works quite well in elastic stage and there are various researches conducted on improving design standards in elastic stage (Ceroni & Pecce 2007, 2009; Zomrodian, Yang, Belarbi, & Ayoub, 2016). A different approach is proposed in this paper, i.e. crack model for direct calculation of flexural crack parameters which neglect the crack spacing. Slaitas et al. (Slaitas et al., 2018a) and Jokūbaitis et al. (Jokūbaitis & Juknevičius, 2013; Jokūbaitis et al., 2013) have established a relation between normal crack width, depth and strains in the level of FRP reinforcement. Such a relation allowed authors to describe the full development of the crack up to the element failure.

Numerical results are compared with experimental ones from the tests of 9 RC beams, strengthened with externally bonded (EBR) and near surface mounted (NSM) FRP reinforcements. The experimental results are collected from different scientific publications (Slaitas et al., 2018a; Xing, Chang, & Ozbulut, 2018).

Method of solution

Crack model

In accordance with Slaitas et al. (Slaitas et al., 2018a) and Jokūbaitis et al. (Jokūbaitis & Juknevičius, 2013; Jokūbaitis et al., 2013) proposed crack development models, flexural reinforced concrete elements crack has two peaks, one leads to the crack spread toward the beam neutral axis, and the other refers to the tensile reinforcement. The crack apex closer to the neutral axis opening width is critical to giving rise to further crack spread, for what concrete and steel, concrete and FRP bond strengths equal to the tensile strength of concrete (f_{ct}), steel and FRP reinforcement stresses (σ_s and σ_f respectively) resist. Parts of the element separated by the crack rotates about the intersection point of crack surface and neutral axis (see Figure 1a).

Figure 1. The model for calculation of normal crack propagation a) crack model; b) strain distribution (source: Slaitas et al., 2018a)

Proposed crack model is presented in Figure 1a, where A_{s1} , A_{s2} , A_f are the areas of inner tensile, compressive steel and external FRP reinforcements respectively; d_1 , d_2 are the distances to centroids of inner reinforcements; h_{cr} and h_{ct} are the crack depth and the depth of tensile zone above it respectively; w_{cr} and δ_{cr} are the crack width and the critical width of a normal crack tip respectively. Strain distribution in a cross section is reflected in Figure 1b, where x_{cc} is a depth of compression zone of concrete; ϵ_{cr} and ϵ_{fe} are the concrete cracking strain and the strain in the level of FRP respectively. The dependence of crack parameters and strains can be expressed from a condition of similarity of triangles (Slaitas et al., 2018a):

$$\frac{h_{ct}}{h_{cr}} \approx \frac{\delta_{cr}}{w_{cr}} \approx \frac{\epsilon_{cr}}{\epsilon_{fe}}. \quad (1)$$

At the concrete cracking point $w_{cr} = 0$ mm, it means strain at the level of FRP must be equal to zero too, therefore this strain in the Eq. (1) is replaced by the strain increment from concrete cracking point:

$$\frac{h_{ct}}{h_{cr}} \approx \frac{\delta_{cr}}{w_{cr}} \approx \frac{\epsilon_{cr}}{\Delta \epsilon_f}, \quad (2)$$

where $\Delta \epsilon_f = 0$, when $M_a = M_{cr}$.

The value of concrete cracking strain:

$$\epsilon_{cr} \approx \frac{f_r}{E_{cm}}, \quad (3)$$

where E_{cm} is a secant modulus of elasticity.

The values of concrete's modulus of rupture (ACI, 318) and secant modulus of elasticity (EN 1992-1-1:2004) for normal weight concrete are estimated by following Equation:

$$f_r = 0.623 \sqrt{f_{cm}}; \quad E_{cm} = 22 \left(\frac{f_{cm}}{10} \right)^{0.3}. \quad (4)$$

The empirical expression of the critical width of a normal crack tip (Slaitas et al., 2018a):

$$\delta_{cr} \approx 0,00012 d_{1,eff} \sqrt[3]{\phi_{eq}}, \quad (5)$$

where $d_{1,eff}$ and ϕ_{eq} are the distance to tensile steel and FRP reinforcement resultant (in mm) and the equivalent factor of tensile zone respectively, which can be found in Slaitas et al. (2018a).

Figure 2. Bending moment-crack width dependency graph

There are 3 main stages in the bending moment-crack width dependency graph (see Figure 2): I. from concrete cracking to yield of steel reinforcement; II. from yield of steel reinforcement to load close to bearing capacity of the element; III. further crack propagation without load increment.

Crack width development law changes 2 times: when the steel reinforcement stress reaches yield strength (stage II) and when the load is close to its ultimate value due to slip at the interface of FRP and concrete (stage III).

I stage $M_{cr} < M_a \leq M_y$

The triangular concrete stress diagram is used for the analysis (see Figure 3).

Figure 3. Stress-strain state in the element a) EBR; b) NSM

Equilibrium condition of internal forces:

$$\frac{1}{2} b x_{cc}^2 + \alpha_{s2} A_{s2} (x_{cc} - d_2) = \alpha_{s1} A_{s1} (d - x_{cc}) + \alpha_f A_f (d_f - x_{cc}) + \alpha_f A_f \frac{\varepsilon_p - \varepsilon_0}{\varepsilon_c} x_{cc} + \frac{1}{2} b h_{ct} \frac{\varepsilon_{cr}}{\varepsilon_c} x_{cc}. \quad (6)$$

When the hypothesis of plane sections is valid, there will be following relation between concrete strain:

$$\varepsilon_c = \frac{\varepsilon_{cr} x_{cc}}{h_{ct}}. \quad (7)$$

From Eqs (6) and (7):

$$\frac{1}{2} b x_{cc}^2 + \alpha_{s2} A_{s2} (x_{cc} - d_2) = \alpha_{s1} A_{s1} (d - x_{cc}) + \alpha_f A_f (d_f - x_{cc}) + \alpha_f A_f \frac{\varepsilon_p - \varepsilon_0}{\varepsilon_{cr}} h_{ct} + \frac{1}{2} b h_{ct}^2. \quad (8)$$

The depth of compression zone of concrete:

$$x_{cc} = \frac{\sqrt{B_{cc}^2 + 2bC_{cc}} - B_{cc}}{b}; \quad (9)$$

$$B_{cc} = \alpha_{s1} A_{s1} + \alpha_{s2} A_{s2} + \alpha_f A_f; \quad (10)$$

$$C_{cc} = \frac{1}{2} b h_{ct}^2 + \alpha_{s1} A_{s1} d + \alpha_{s2} A_{s2} d_2 + \alpha_f A_f \left(d_f + \frac{\varepsilon_p - \varepsilon_0}{\varepsilon_{cr}} h_{ct} \right), \quad (11)$$

where the ratios of modules of elasticity:

$$\alpha_{s1} = \frac{E_{s1}}{E_{cm}}; \quad \alpha_{s2} = \frac{E_{s2}}{E_{cm}}; \quad \alpha_f = \frac{E_f}{E_{cm}}. \quad (12)$$

Concrete strains in Eq. (7) can be expressed as follows:

$$\varepsilon_{cr} \approx \frac{(M_{cr} - M_p) y_t}{E_{cm} I_g}; \quad \varepsilon_c \approx \frac{(M_a - M_p) x_{cc}}{E_{cm} I_{cr}}. \quad (13)$$

Then the height of tension zone of concrete can be expressed from Eqs (7) and (13):

$$h_{ct} \approx \frac{(M_{cr} - M_p) I_{cr}}{(M_a - M_p) I_g} y_t, \quad (14)$$

where the depth of tensile zone of concrete corresponding to cracking moment can be taken as $y_t \approx h / 2$; cracking moment of concrete:

$$M_{cr} = \frac{f_r I_g}{y_t} + M_p. \quad (15)$$

Prestressing moment of FRP reinforcement:

$$M_p = P \left(e_p + \frac{I_g}{b h y_t} \right), \quad (16)$$

where P is prestressing force of FRP reinforcement; eccentricity of FRP prestressing force can be taken as $e_p \approx d_f - h / 2$; gross moment of inertia:

$$I_g = \frac{b h^3}{12}. \quad (17)$$

Moment of inertia of cracked cross-section:

$$I_{cr} = \frac{b x_{cc}^3}{3} + \alpha_{s1} A_{s1} (d - x_{cc})^2 + \alpha_{s2} A_{s2} (x_{cc} - d_2)^2 + \alpha_f A_f (d_f - x_{cc})^2. \quad (18)$$

Equilibrium between concrete compression and tension zone depths is found iteratively. Acting bending moment (see Figure 3):

$$M_a = E_{s1} A_{s1} \varepsilon_{fe} \left(\frac{d - x_{cc}}{d_f - x_{cc}} \right) \left(d - \frac{x_{cc}}{3} \right) + E_{s2} A_{s2} \varepsilon_{fe} \left(\frac{x_{cc} - d_2}{d_f - x_{cc}} \right) \left(\frac{x_{cc}}{3} - d_2 \right) + E_f A_f (\varepsilon_{fe} + \varepsilon_p - \varepsilon_0) \left(d_f - \frac{x_{cc}}{3} \right) + f_r b h_{ct} \frac{(h_{ct} + x_{cc})}{3}. \quad (19)$$

The strain at the level of FRP reinforcement can be expressed from Eq. (19):

$$\varepsilon_{fe} = \frac{M_a - E_f A_f (\varepsilon_p - \varepsilon_0) \left(d_f - \frac{x_{cc}}{3} \right) - f_r b h_{ct} \frac{(h_{ct} + x_{cc})}{3}}{E_{s1} A_{s1} \left(\frac{d - x_{cc}}{d_f - x_{cc}} \right) \left(d - \frac{x_{cc}}{3} \right) + E_{s2} A_{s2} \left(\frac{x_{cc} - d_2}{d_f - x_{cc}} \right) \left(\frac{x_{cc}}{3} - d_2 \right) + E_f A_f \left(d_f - \frac{x_{cc}}{3} \right)}, \quad (20)$$

where ε_p and ε_0 are the FRP prestressing strain and initial strain at the level of FRP respectively.

The FRP reinforcement stress:

$$\sigma_f = E_f (\varepsilon_{fe} + \varepsilon_p - \varepsilon_0) \leq f_f. \quad (21)$$

The tensile steel reinforcement stress:

$$\sigma_{s1} = E_{s1} \varepsilon_{fe} \frac{d - x_{cc}}{d_f - x_{cc}} \leq f_y. \quad (22)$$

If initial bending moment (moment of strengthening) $M_0 \leq M_{cr}$, then the strain increment in the level of FRP in Eq. (2):

$$\Delta \varepsilon_{f, M_{cr} < M_a \leq M_y} = \frac{M_a - M_{cr}}{E_{s1} A_{s1} \left(\frac{d - x_{cc}}{d_f - x_{cc}} \right) \left(d - \frac{x_{cc}}{3} \right) + E_{s2} A_{s2} \left(\frac{x_{cc} - d_2}{d_f - x_{cc}} \right) \left(\frac{x_{cc}}{3} - d_2 \right) + E_f A_f \left(d_f - \frac{x_{cc}}{3} \right)}. \quad (23)$$

Otherwise, the following Equation should be used instead of Eq. (2).

$$\frac{\delta_{cr}}{\Delta w_f} \approx \frac{\varepsilon_{cr}}{\Delta \varepsilon_f^*}; \quad (24)$$

$$\Delta \varepsilon_{f.M_{cr} < M_a \leq M_y}^* = \frac{M_a - M_0}{E_{s1} A_{s1} \left(\frac{d - x_{cc}}{d_f - x_{cc}} \right) \left(d - \frac{x_{cc}}{3} \right) + E_{s2} A_{s2} \left(\frac{x_{cc} - d_2}{d_f - x_{cc}} \right) \left(\frac{x_{cc}}{3} - d_2 \right) + E_f A_f \left(d_f - \frac{x_{cc}}{3} \right)}. \quad (25)$$

Finally, the crack width will be the sum of initial crack width and the increment after strengthening.

$$w_{cr} = w_0 + \Delta w_f. \quad (26)$$

II stage $M_y < M_a < M_u$

After the stress of internal steel reinforcement reaches yield strength, equilibrium condition of external and internal forces in Eq. (8), taking into account Eq. (7), will be as follows:

$$\frac{1}{2} b x_{cc}^2 + \alpha_{s2} A_{s2} (x_{cc} - d_2) = A_{s1} f_y \frac{h_{ct}}{f_r} + \alpha_f A_f (d_f - x_{cc}) + \alpha_f A_f \frac{\varepsilon_p - \varepsilon_0}{\varepsilon_{cr}} h_{ct} + \frac{1}{2} b h_{ct}^2. \quad (27)$$

The depth of compression zone of concrete:

$$x_{cc} = \frac{\sqrt{B_{cc}^2 + 2bC_{cc}} - B_{cc}}{b}; \quad (28)$$

$$B_{cc} = \alpha_{s2} A_{s2} + \alpha_f A_f; \quad (29)$$

$$C_{cc} = \frac{1}{2} b h_{ct}^2 + A_{s1} f_y \frac{h_{ct}}{f_r} + \alpha_{s2} A_{s2} d_2 + \alpha_f A_f \left(d_f + \frac{\varepsilon_p - \varepsilon_0}{\varepsilon_{cr}} h_{ct} \right). \quad (30)$$

Acting bending moment:

$$M_a = A_{s1} f_y \left(d - \frac{x_{cc}}{3} \right) + E_{s2} A_{s2} \varepsilon_{fe} \left(\frac{x_{cc} - d_2}{d_f - x_{cc}} \right) \left(\frac{x_{cc}}{3} - d_2 \right) + E_f A_f (\varepsilon_{fe} + \varepsilon_p - \varepsilon_0) \left(d_f - \frac{x_{cc}}{3} \right) + f_r b h_{ct} \frac{(h_{ct} + x_{cc})}{3}. \quad (31)$$

The strain at the level of FRP reinforcement:

$$\varepsilon_{fe} = \frac{M_a - E_f A_f (\varepsilon_p - \varepsilon_0) \left(d_f - \frac{x_{cc}}{3} \right) - f_r b h_{ct} \frac{(h_{ct} + x_{cc})}{3} - A_{s1} f_y \left(d - \frac{x_{cc}}{3} \right)}{E_{s2} A_{s2} \left(\frac{x_{cc} - d_2}{d_f - x_{cc}} \right) \left(\frac{x_{cc}}{3} - d_2 \right) + E_f A_f \left(d_f - \frac{x_{cc}}{3} \right)}. \quad (32)$$

The stress of FRP reinforcement is calculated by Eq. (21) and the stress of internal steel reinforcement will be equal to its yield strength.

The strain increment at the level of FRP can be expressed by following Equation.

$$\Delta \varepsilon_{f.M_a > M_y}^* = \frac{M_a - M_y}{E_{s2} A_{s2} \left(\frac{x_{cc} - d_2}{d_f - x_{cc}} \right) \left(\frac{x_{cc}}{3} - d_2 \right) + E_f A_f \left(d_f - \frac{x_{cc}}{3} \right)}. \quad (33)$$

The crack width will be the sum of crack width corresponding to yield moment of steel reinforcement and the increment of crack width after it.

$$w_{cr} = w_{cr.y} + \frac{\delta_{cr} \Delta \varepsilon_{f.M_a > M_y}^*}{\varepsilon_{cr}}. \quad (34)$$

III stage $M_a \sim M_u$

When the acting moment is getting close to load bearing capacity, a clear slip will appear in concrete and FRP contact surface, what will give rise for further crack propagation without load increment. The slip of FRP reinforcement can be taken into account with contact stiffness reduction factor based on built up bars theory (Slaitas et al., 2018a, 2018b; Skuturna & Valivonis, 2014; Valivonis & Marčiukaitis, 2006):

$$\psi_f = 1 - \frac{sh(\lambda \cdot L_s) \cdot ch\left(\lambda \cdot \left(\frac{L}{2} - L_s\right)\right)}{\lambda \cdot L_s \cdot ch\left(\frac{\lambda \cdot L}{2}\right)}, \quad (35)$$

where λ is a factor evaluating the stiffness of the interface (read more in Skuturna & Valivonis, 2014).

Subsequently, ultimate value of crack width is calculated by following equation:

$$w_{cr,u} = w_{cr,y} + \frac{\delta_{cr} \Delta \varepsilon_{f,M_a > M_y}}{\psi_f \varepsilon_{cr}}. \quad (36)$$

Experimental beams

Four-point bending test results (see Figure 4) of nine full scale RC beams strengthened with externally bonded CFRP reinforcements (EBR) and with near surface mounted GFRP reinforcements (NSM) were used in the analysis. Experimental beams varied by length, material properties, reinforcement ratio and strengthening. CFRP layer of beams BS-0-1, B1-0 and B2-P was additionally anchored with steel clamps. Beam B5-7 was strengthened under an external load action. FRP reinforcement of the beams B3-0, B4-0, B5-7, GCB-1, GCB-2, GCB-3 was unanchored. Properties of tested beams are listed in Table 1 below.

Figure 4. Loading scheme

Table 1. Properties of specimens and materials

Reference	Beam ID	b (m)	h (m)	L (m)	L_s (m)	A_{s1} (mm ²)	f_y (MPa)	A_f (mm ²)	f_f (MPa)	A_{s2} (mm ²)	f_{cm} (MPa)	Method
*	BS-0-1	0.18	0.3	2.81	0.955	308	500	60	2800	157	38.00	EBR
Slaitas et al., 2018	B1-0	0.15	0.3	2.7	0.9	308	569	144	2334	226	25.40	EBR
Slaitas et al., 2018	B2-P**	0.15	0.3	2.7	0.9	308	569	144	2334	226	25.40	EBR
Slaitas et al., 2018	B3-0	0.1	0.2	1.2	0.4	226	318	16.7	4800	100	28.70	EBR
Slaitas et al., 2018	B4-0	0.1	0.2	1.2	0.4	226	318	16.7	4800	100	28.70	EBR
Slaitas et al., 2018	B5-7***	0.1	0.2	1.2	0.4	226	318	16.7	4800	100	28.70	EBR
Xing et al., 2018	GCB-1	0.2	0.35	1.8	0.6	402	498	308	966	226	46.80	NSM
Xing et al., 2018	GCB-2	0.2	0.35	1.8	0.6	402	498	462	966	226	47.30	NSM
Xing et al., 2018	GCB-3	0.2	0.35	1.8	0.6	628	505	308	966	226	49.20	NSM

* Beam BS-0-1 was tested by authors;

** Beam B2-P was prestressed, $\sigma_p = 120$ MPa;

*** Beam B5-7 was strengthened under external load action, $M_0 = 7$ kNm.

All beams in Table 1 were strengthened with externally bonded carbon fibre reinforced polymer plates (BS-0-1, B1-0, B2-P), sheets (B3-0, B4-0, B5-7) and near surface mounted glass fibre reinforced polymer rods (GCB-1, GCB-2, GCB-3).

Results and discussion

The crack width of the beams strengthened with externally bonded CFRP reinforcements (EBR) was measured in I and II stages (see Figure 5). While Xing et al. (2018) has established full load-crack width response of the beams strengthened with near surface mounted GFRP reinforcements (NSM) (see Figure 6).

Figure 5. Bending moment-crack width dependency graphs (EBR) a) BS-0-1; b) B1-0; c) B3-0, B4-0

Figure 6. Bending moment-crack width dependency graphs (NSM) a) GCB-1; b) GCB-2; c) GCB-3

There is a strong relation between predicted and experimental bending moment-crack width dependency (coefficient of correlation $R = 0.95 \div 1.00$). After the stress in tensile steel reinforcement of FRP strengthened RC element has reached its yield strength, the structure does not collapse, but the behaviour changes. This study confirms the 3 stage crack width development theory. The comparison of experimental and numerical results has shown that proposed calculation model allows us to predict full response of maximum crack width propagation.

Additionally, comparison of experimental and numerical results of crack width corresponding to bending moment $M_a = 0.5 \div 0.8 M_u$ of all beams in Table 1 is presented in Figure 7.

Figure 7. Comparison of experimental and numerical results of crack width

2 additional beams were added to those in Figures 5 and 6, one of which was strengthened under external load action with initial moment, crack width of it was approximately 2 times higher than in similar beams. Experimental and numerical results have a good agreement (coefficient of correlation $R = 1.00$) with low systematic error (1.04) and coefficient of variation (16.03%), which demonstrates a sufficient level of accuracy in terms of crack width prediction.

Conclusions

Prediction model of crack parameters in normal section of FRP strengthened RC elements was presented in this paper. The analysis of experimental and numerical results revealed:

- Crack development of FRP strengthened RC elements can be divided into 3 main stages: I. from concrete cracking to yield of steel reinforcement; II. from yield of steel reinforcement to load close to bearing capacity of the element; III. further crack development without load increment. The analysis of the results has shown that crack width in all stages can be predicted sufficiently precisely (coefficient of correlation $R = 0.95 \div 1.00$).
- Various RC beams strengthened with externally bonded and near surface mounted FRP reinforcements were analysed. Experimental and numerical results at the load close to service load ($M_a = 0.5 \div 0.8 M_u$) had a good agreement (coefficient of correlation $R = 1.00$) with low systematic error (1.04) and coefficient of variation (16.03%), which demonstrates a sufficient level of accuracy in terms of crack width prediction.
- Proposed analytical model can be used for prediction of concrete crack parameters in normal section of the RC beams strengthened with prestressed and not prestressed FRP reinforcements.

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