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INDIRECT CRACK CONTROL PROCEDURE FOR FRP-REINFORCED CONCRETE BEAMS AND ONE-WAY SLABS

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ABSTRACT

This paper reports an alternative model for flexural crack control of FRP-reinforced members in which, consistently with current ACI 318 recommendations for steel-reinforced members, cracks are controlled indirectly through a maximum bar spacing requirement instead of being calculated directly. The proposed procedure results from the impracticalities associated with direct crack width measurement in concrete structures due to the high variability of both concrete cracking and crack width measurements. The proposed model explicitly accounts for the dominant effects that bar cover, FRP reinforcement stress, stiffness and bond properties have on cracking of FRP-reinforced concrete beams and one-way slabs. The procedure is seen as a simplification of the existing ACI 440.1R-06 direct crack control recommendations for serviceability design of FRP-reinforced members, rather than a modification.

KEYWORDS

Cracking, Crack widths, FRP Reinforcement, Serviceability, Slabs.

1. INTRODUCTION

1.1 Flexural Crack Control in Steel-reinforced Concrete Members

According to Frosch (1999), the maximum crack width at the tension face of a reinforced concrete beam or one-way slab can be calculated as:

$$w = 2\frac{f_r}{E_r}\beta\, k_b\sqrt{d_c^2 + \left(\frac{s}{2}\right)^2} \qquad (1)$$

where f_r is the reinforcing bar stress, calculated assuming elastic-cracked conditions, E_r is the modulus of elasticity of the reinforcement, β is the ratio of the distance from the neutral axis to the tension face of the member to the distance from the neutral axis to the centroid of the tensile reinforcement, d_c is the cover thickness from the tension face to the center of the closest rebar, s is the bar spacing (taken as the member width for a single bar case) and k_b is a coefficient that accounts for the bond characteristics of the bars. For steel-reinforced concrete members, β can be taken as 1+ 0.0031 d_c, with d_c in mm (Frosch, 1999). Equation 1 is expressed herein in generic form because it is valid regardless of the type of reinforcement.

Based on Frosch's work, ACI 318 introduced in 1999 changes to the crack control rules in which a maximum bar spacing, rather than a z-factor (related to crack width), is prescribed. The exposure condition dependence was also eliminated. These changes resulted from the high variability of concrete cracking and crack width measurements, unacceptable results with the traditional Gergely-Lutz model for large bar covers, and research showing no direct relationship between bar corrosion and crack width. The crack control equation in ACI 318M-05 is defined as

$$s = 380\left(\frac{280}{f_s}\right) - 2.5\,c_c \le 300\left(\frac{280}{f_s}\right) \qquad (f_s \text{ in MPa}) \tag{2}$$

where f_s is the steel reinforcement stress at service level and c_c is the clear cover. This approach is "indirect" because if Eq. 1 (which is the basis for Eq. 2) were to be solved for the maximum bar spacing, the resulting equation (Eq. 3) becomes constrained by a crack width limit, w.

$$s = 2\sqrt{\left(\frac{w\,E_r}{2 f_r \beta\, k_b}\right)^2 - d_c^2} \tag{3}$$

Although ACI 318M-05 does not explicitly link Eq. 2 to a particular crack width, Frosch (1999) showed that Eq. 2 is indirectly tied to a crack width that varies between 0.4 mm (0.016 in.) to about 0.52 mm (0.02 in.). The latter is just a 30% variation from the former. In lieu of more precise calculations, f_s can be taken as $0.67 f_y$ which exceeds the $0.6 f_y$ value assumed in earlier ACI 318 code versions. This is because of the new load factors in ACI 318. The higher stress level leads to a control crack width lower bound of (0.4 mm)(0.67/0.6) = 0.44 mm (0.018 in.).

Figure 1 shows Eq. 3 predictions in terms of concrete cover for crack widths between 0.44 mm and 0.58 mm. The figure shows that Eq. 2 is a discontinuous representation of Eq. 3 for this crack width range.

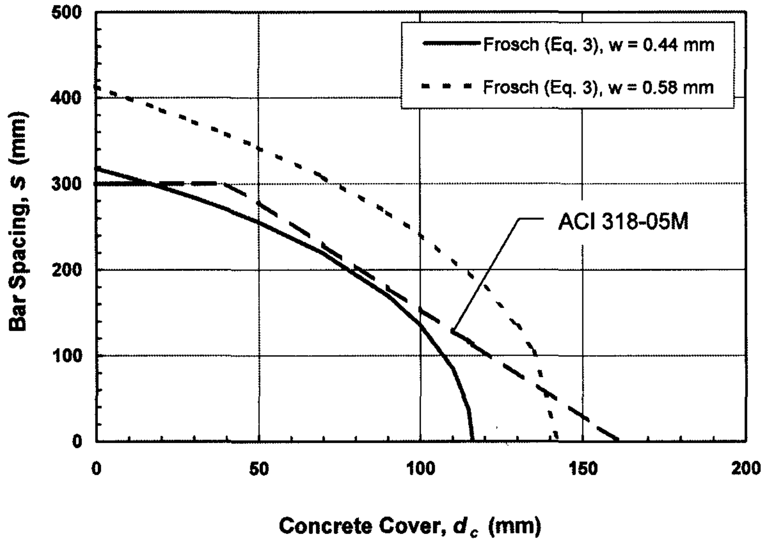

Figure 1: ACI 318M-05 Crack Control Provisions

1.2 Flexural Crack Control in FRP-reinforced Concrete Members

In ACI 440.1R-06, cracks are controlled directly by comparing maximum crack widths per Eq. 1 with crack width limits equal to 0.5 mm (0.020 in.) and 0.7 mm (0.028 in.), for exterior and interior exposure conditions, respectively. For FRP bars with bond similar to that of black steel bars, k_b is equal to one. For FRP bars with better bond, k_b is less than one. For FRP bars with inferior bond, k_b is greater than one. If k_b is unknown, it shall be taken as 1.4 for non-smooth bars. The crack width limits in ACI 440.1R-06 are taken from the Canadian Highway Bride Design Code (2000). These limits are more relaxed than those associated with conventional reinforced concrete design due to the corrosion-free nature of FRP.

The main objective of this paper is to propose an indirect crack control model for FRP-reinforced concrete beams and one-way slabs in which a maximum bar spacing is prescribed in lieu of direct calculation of crack widths. The goal is to keep the format of Eq. 2, including the dominant variables already identified by ACI 318M-05, together with the relevant mechanical properties of FRP that influence cracking in FRP-reinforced concrete members.

2. PROPOSED MODEL

Using Eq. 2 as a starting point, the maximum spacing of FRP bars in FRP-reinforced concrete beams and one-way slabs can be expressed as:

$$s = 380\left(\frac{280}{f_r}\right)\left(\frac{E_r}{200{,}000}\right)\left(\frac{w}{0.44}\right)\frac{1}{k_b} - 2.5\,c_c \le 300\left(\frac{280}{f_r}\right)\left(\frac{E_r}{200{,}000}\right)\left(\frac{w}{0.44}\right)\frac{1}{k_b} \qquad (4)$$

with f_r and E_r in MPa. The terms E_r and w have been normalized by 200,000 MPa and 0.44 mm, respectively, to enable the calculation of the maximum bar spacing in members reinforced with FRP bars. In turn, Eq. 4 leads to

$$s = 1.2\frac{E_r w}{f_r k_b} - 2.5\,c_c \le 0.95\frac{E_r w}{f_r k_b} \qquad (f_r \text{ and } E_r \text{ in MPa}) \qquad (5)$$

which is conceptually consistent with the approach recommended by Frosch (2001) and suitable to comply with any target crack width limit. For instance, substituting $w = 0.7$ mm into Eq. 5, and simplifying, renders

$$s = 0.8\frac{E_r}{f_r k_b} - 2.5\,c_c \le 0.7\frac{E_r}{f_r k_b} \qquad (f_r \text{ and } E_r \text{ in MPa}) \qquad (6)$$

Figure 2 shows maximum bar spacing predictions from Eqs. 3 and 5 as a function of d_c for a member with GFRP bars with E_r = 40 GPa, f_r = 80 MPa, and k_b = 1.4, for limiting crack widths of 0.7 and 0.91 mm. In the bar spacing predictions for w = 0.7 mm, three β expressions are examined: $\beta = 1 + 0.0063\,d_c$ (shallow members, d = 230 mm), $\beta = 1 + 0.0008\,d_c$ (deeper members, d = 1800 mm), and $\beta = 1 + 0.0031 d_c$, as assumed by Frosch for steel-reinforced members. These β values were derived using Eq. 7, assuming $k = 0.3$.

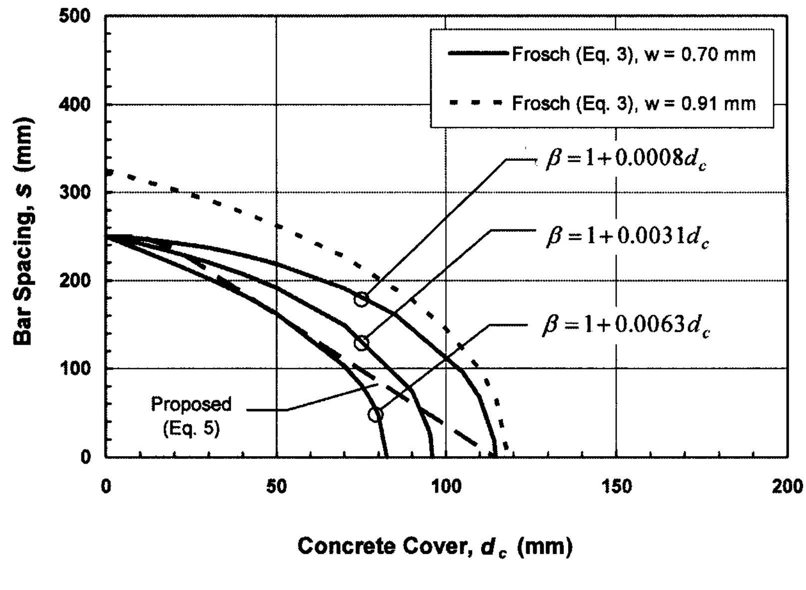

Figure 2: Proposed Crack Control Provisions for GFRP-reinforced Concrete Members

$$\beta = 1 + \frac{d_c}{d(1-k)} \qquad (7)$$

The evaluation of the β effect is necessary because the flexural depth, d, can vary significantly in concrete construction. The bar spacing predictions for w = 0.91 mm are based on $\beta = 1 + 0.0031 d_c$ only.

Figure 2 shows that, in theory, it is possible to explicitly account for the β effect in the maximum bar spacing calculation. However, for reasons of simplicity, Frosch's assumption for β looks appropriate as a balance between shallow and deep FRP-reinforced concrete members. It is worth noting that the proposed indirect crack control procedure implicitly uses $\beta = 1 + 0.0031 d_c$. Figure 2 also shows that Eq. 5 provides a reasonable representation of Eq. 3 for crack control of FRP-reinforced concrete members for the specific conditions assumed.

3. DISCUSSION

Because of its general nature, Eq. 5 can be applied to both steel- and FRP-reinforced concrete beams and one-way slabs, provided k_b is defined accordingly. In addition to the clear cover, the model accounts for the elastic stiffness and bond characteristics of FRP, which are key parameters for serviceability design of FRP-reinforced members.

The model is meant for use in conjunction with allowable crack width values that are considered satisfactory for the designated function of the structure. Equation 5 can be used to ensure compliance with any crack width value to be controlled. For steel-reinforced concrete members, the indirect crack control equation of ACI 318M-05 may not be readily used in situations in which crack widths much smaller than 0.4 mm are to be controlled, as is the case of structures where water tightness control is essential and structures subject to the use of deicing chemicals. Equation 5 also gives freedom to designers to select a target FRP strain to ensure compliance with both the serviceability and ultimate limit states, especially in those cases where either excessive deflections or brittle failure due to GFRP reinforcement creep rupture may be caused by allowing cracks that are too wide.

Rather than being a drastic departure from the ACI 440.1R-06 model, Eq. 5 provides a discontinuous representation of Eq. 3. The procedure is simpler because the impracticalities associated with crack width measurements are avoided. Using indirect procedures for serviceability design of FRP-reinforced members is not new. In fact, ACI 440.1R-06 provides an indirect procedure for deflection control of beams and one-way slabs with FRP bars.

4. CONCLUSION

This paper presents an indirect procedure to control cracking in FRP-reinforced concrete beams and one-way slabs in which a maximum bar spacing is prescribed instead of a maximum crack width. The procedure is indirect because the maximum bar spacing requirement is indirectly linked to a target crack width value that is to be complied with. The proposed procedure can also be applied to steel-reinforced concrete beams and one-way slabs.

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