Construction and Building Materials 113 (2016) 77–89

Contents lists available at ScienceDirect

Construction and Building Materials

journal homepage: www.elsevier.com/locate/conbuildmat

Bond-dependent coefficient of glass- and carbon-FRP bars in normal- and high-strength concretes

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HIGHLIGHTS

- Investigating the bond-dependent coefficient, k_b, of FRP bars in normal and high strength concretes.
- Evaluating the dependency of k_b values on FRP bar type, diameter, and concrete type, and strength.
- Evaluating the determined k_b values against the current recommendations of FRP design codes and guides.

ARTICLE INFO

Article history:
Received 21 October 2015
Received in revised form 29 February 2016
Accepted 2 March 2016
Available online 12 March 2016

Keywords:
Concrete
Fiber-reinforced polymer (FRP)
Beam
Strain
Crack width
Bond-dependent coefficient
Serviceability

ABSTRACT

The design of concrete members reinforced with fiber-reinforced-polymer (FRP) bars is typically governed by serviceability state rather than ultimate state. This necessitates verifying the crack width in FRP-reinforced concrete (FRP-RC) members at service load. Recent developments in the FRP industry led to the introduction of FRP bars with different surface configurations and mechanical properties, which are expected to affect their bond performance. In the absence of test data, the design codes and guides, however, recommend bond-dependent coefficient (k_b) values considering the surface configurations. Thus, this study aims at investigating the k_b values and verifying the dependency of the k_b values on FRP bar type (glass [GFRP] and carbon [CFRP]), diameter, and concrete type, and strength limited to the first critical flexural-crack width. The investigation included 16 beams measuring 4250 mm long × 200 mm wide × 400 mm deep. The beams were reinforced with sand-coated GFRP bars, helically-grooved GFRP bars, and sand-coated CFRP bars and were fabricated with normal- and high-strength concretes (NSC and HSC). The measured first critical-crack widths and measured strains were used to assess the current k_b values recommended in FRP design codes and guides. The findings did not support using the same k_b value for FRP bars of different types (carbon and glass) with similar surface configurations.

1. Introduction

Steel bars have been traditionally used as main reinforcement in concrete structures. Durability issues and corrosion-related problems and costs, however, have driven the urgency for introducing alternative reinforcement. Fiber-reinforced-polymer (FRP) bars are corrosion resistant by nature and they are becoming more common reinforcing materials in concrete structures and bridges. Due to the lower modulus of elasticity of FRP bars, the design of FRP-reinforced concrete (FRP-RC) members is governed by serviceability state (deflection and crack width) rather than ultimate state. Crack-width calculations, however, include the effect of bond between FRP bars and surrounding concrete. This is normally taken into account in FRP design codes and guides through the so-called bond-dependent coefficient (k_b), while the interpretation of this coefficient remains ambiguous [1].

The recent development of the FRP industry led to the introduction of different FRP bars with different surface configurations and mechanical properties (such as sand-coated, helically-grooved, deformed, and indented). These differences affect FRP-bar bond performance, which relies mostly on friction and mechanical interlock [2]. Baena et al. [3] reported that, in addition to bar mechanical properties, the bond between FRP bars and concrete

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http://dx.doi.org/10.1016/j.conbuildmat.2016.03.005
0950-0618/

Nomenclature

A	effective tension area of concrete surrounding the flexural tension reinforcement and bearing the same centroid as the reinforcement, divided by the number of bars (mm^2)
A_f	area of FRP tension reinforcement (mm^2)
d_b	bar diameter (mm)
d_c	distance from extreme tension fiber to the center of the longitudinal bar or wire located closest thereto according to the code or guideline (mm)
E_f	modulus of elasticity of longitudinal FRP reinforcement (MPa)
E_s	modulus of elasticity of longitudinal steel reinforcement (MPa)
f'_c	compressive strength of the concrete (MPa)
f_f	stress in FRP reinforcement under specified loads (MPa)
f_{frpu}	characteristic tensile strength (average – 3 standard deviations (SD))
f_{fu}	ultimate strength of FRP longitudinal reinforcement (MPa)
f_t	tensile strength from cylinder-splitting test (MPa)
h_1	distance from neutral axis to center of tensile reinforcement (mm)
h_2	distance from neutral axis to extreme tension fiber (mm)
k_b	bond-dependent coefficient
M_n	nominal moment of the reinforced-concrete section (kN·m)
s	spacing between the longitudinal reinforcement bars (mm)
w	maximum crack width (mm)
ρ_f	longitudinal reinforcement ratio
ρ_{fb}	balanced longitudinal reinforcement ratio
ε_{cu}	ultimate compressive strain of the concrete = 0.003

depends on many factors, including concrete compressive strength, bar diameter, and bar surface treatment. An increase in bond strength and changes in failure mode and failure surface have been observed with varying concrete compressive strengths. Analysis of the influence of surface treatment on bond behavior has confirmed that different bond mechanisms are involved with different surface treatments. In addition, there has been a tendency for FRP bars of larger diameter to show lower bond strength. Thus, the variations in surface configuration, bar diameter, and concrete strength are expected to affect the bond performance of FRP bars and, consequently, k_b.

While many studies have been conducted on the bond behavior of FRP bars in different concrete types, there have been no consistent interpretations of k_b, which is required for checking the crack width under serviceability condition. In addition, a previous study demonstrated that the reason for variability of k_b values is a lack of correlation between the actual cracking behavior and the basic mathematical form of the crack prediction equation [1]. In absence of significant test data, design codes and guides provide recommendations for k_b values that may be used in predicting the crack widths. These values, however, depend solely on the surface configurations of the FRP bars (i.e., CSA [4] recommends k_b of 0.80 for sand-coated FRP, regardless of fiber type).

Thus, this paper presents an investigation to determine the bond-dependent coefficient (k_b) of different types and diameters of FRP bars with different surface configurations in normal- and high-strength concretes (NSC and HSC). Moreover, it compares the current k_b values (recommended in absence of test results) provided by FRP design codes and guides with the experimental values determined with reference to CSA [5], Annex S "Test Method for Determining the Bond-Dependent Coefficient of FRP Rods."

2. Prediction of the bond-dependent coefficient (k_b)

The determination of k_b was introduced in ACI [6] by modifying the Gergely–Lutz [7] equation to account for FRP instead of steel bars. Some typical k_b values for deformed GFRP bars cited in ACI [6] are between 0.71 and 1.83. In addition, ACI [6] suggested that designers assume a value of 1.2 for deformed GFRP bars unless more specific information were available for a particular bar. Later, ACI Committee 440 (Fiber-Reinforced Polymer Reinforcement) adopted a modified version of the crack-width equation proposed by Frosch [8] in place of the modified Gergely–Lutz equation and introduced it in ACI [9]. Based on assembled experimental data, Subcommittee 440-0H (Fiber-Reinforced Concrete), it was reported [10] that the k_b in Frosch's equation was 19% greater than the k_b resulting from the equation attributed to Gergely–Lutz. The k_b values based on Frosch's equation ranged from 0.60 to 1.72 with an average of 1.10 ± 0.31 [10]. Data for rough sand-coated FRP bar surface treatments trended towards the lower end of this range [10]. CSA [4] adopted the ACI [9] crack-width equation, while the k_b values were not updated to account for the 19% increase in k_b values with Frosch's equation. It should be mentioned that the recommended k_b values provided by design codes and guides in the absence of experimental test data depend only on the surface configurations of the FRP bars.

According to CSA [5], Annex S, k_b should be determined from the measured crack widths and strains in the FRP bars (at service stage) during testing and using Eq. (1).

$$w = 2\frac{f_f}{E_f}\beta \times k_b \times \sqrt{d_c^2 + \left(\frac{S}{2}\right)^2} \quad (\text{CSA [4]; ACI [9]}) \tag{1}$$

In addition to Eq. (1) provided by ACI [9] and CSA [4], Eq. (2) by ISIS [11] was also used to evaluate the k_b values for comparison.

$$w = 2.2k_b\frac{f_f}{E_f}\frac{h_2}{h_1}\sqrt[3]{d_cA} \quad (\text{ISIS [11]}) \tag{2}$$

3. Experimental program

3.1. Materials

Since k_b is affected by the surface configuration and bond performance, three types of GFRP bars (two sand-coated and one helically grooved) and one type of CFRP bar (sand-coated) were used. The FRP bars were classified according to CSA [12] as Grade I and III. The GFRP bars are referred to as GFRP-1 for sand-coated, normal-modulus bars (Grade I), GFRP-2 for sand-coated, high-modulus bars (Grade III), and GFRP-3 for helically grooved, high-modulus bars (Grade III). Furthermore, the CFRP bars are referred to as CFRP-1 for sand-coated bars (Grade III). The tensile properties of the FRP reinforcing bars were determined by testing five representative specimens according to ASTM [13]. Fig. 1 shows the GFRP bars used in this study and Table 1 provides the properties of the FRP bars.

The beam specimens were fabricated with commonly used NSC and HSC with target compressive strengths of 35 and 65 MPa, respectively. The compressive and splitting concrete strengths were determined by testing three 150 × 300 mm cylinders on the day of testing. The measured compressive and splitting concrete strengths of the NSC ranged from 33.8 to 48.1 MPa and from 3.11 to 3.96 MPa, respectively. In contrast, the measured compressive and splitting concrete strengths of the HSC ranged from 76.5 to 81.5 MPa and from 4.62 to 5.45 MPa, respectively. Table 2 presents the concrete compressive and tensile strengths.