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Development Length of GFRP Rebar Based on Non-uniform Bond Stress

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Abstract

Due to the assumption of uniform bond stress, the development length of FRP bars by design standards can be unnecessarily long and difficult to provide in practice. Hence, the bond stress distribution and required development length of a GFRP rebar is investigated. Four beam-bond specimens, two following RILEM specifications and two based on a procedure by ACI are tested to evaluate the effect of test method on bond strength. Ten pullout tests are also performed using the same bar. The two test methods yield similar results, but the ACI test is easier to perform. The bond stress distribution in the beams is highly nonlinear but in the pullout tests approaches uniformity. The actual development length is found to be 50% to 250% less than that required by the foregoing standards. Consequently, a new equation is proposed based on the logistic growth function to model the non-uniform bond stress distribution and estimate the required development length.

Keywords: Analysis; beam (support); bond stress test; development length; GFRP bar.

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1. Introduction

One of the disadvantages of conventional reinforced concrete is its lack of durability in aggressive environments due to steel reinforcement corrosion. Fibre Reinforced Polymer (FRP) reinforcing bars, termed rebar for brevity, made of continuous fibres of glass or carbon embedded in a thermoset polymer/resin, are corrosion-resistant and can be used instead of steel rebars in such environments. An essential property of a rebar in reinforced concrete (RC) is its ability to transfer the bar force to the adjoining concrete at all stages of loading, without excessive slip at the concrete-bar interface. The stress transfer between the bar and the concrete occurs through interfacial shear stress, commonly referred to as bond stress, which is accompanied by circumferential normal stress in the concrete annulus surrounding the bar. The bond strength of the bar is a function of the magnitude and distribution of bond stress as well as the ability of the interface to redistribute them. The normal stress being tension causes radial cracks in the concrete surrounding the bar which, if left unrestrained, allows the cracks to widen and propagate to the concrete surface, negatively affecting the shear resistance of the interface and leading to bond loss. Bond stress transfer occurs via adhesion, friction and mechanical action (bearing) at the bar-concrete interface (ACI 408 R03 2003), but increase in transfer by bearing and friction normally occurs at the expense of reduction in the transfer by adhesion. In addition, the relative elastic moduli of concrete and the bar will affect both the maximum bond stress and its distribution along the embedded length.

A recent study by Yan et al. 2016 revealed large scatter among the reported bond strength values for GFRP bars and the inability of the current standards (ACI 440.1 2006; CSA S806 2012 and CSA S6 2006) to adequately predict the bars' actual bond strength and development length. The study by Quayyum & Rteil 2012, which involved the analysis of 541 beam-bond tests, where bond loss occurred due to either concrete splitting or bar pullout, showed that none of the methods in

current standards can consistently predict the actual bond strength of FRP bars and they are often highly conservative. Furthermore, none of the standards consistently gives better estimate of the actual bond strength and development length than the others.

Although using results from many sources can be useful in arriving at design equations, when using the results, one must ensure that the tests have been performed according to a standard test method. Otherwise, significantly different development lengths may be obtained for the same bar. Common bond test methods include beam-bond test and pullout test. It is well known that the pullout test does not provide the same bond strength and development length as the beam test due to the differences in the stress field around and along the bar. Different beam-bond tests are available, including the midspan hinged beam (RILEM TC-RC5 1994), the notched beam (ACI 408 R03 2003), the spliced bar beam (ACI 408 R03 2003) and the beam end (ASTM A944-10 2015) tests.

In conventional RC design, it is common practice to proportion a section so steel yielding occurs at critical sections at the ultimate state while in FRP reinforced concrete (FRP-RC) design, the recommended practice is to over-reinforce the members and let failure initiate by concrete crushing. Consequently, for FRP bars the embedded length required to achieve the design bar stress at ultimate state is the important quantity. Finally, current bond strength and development length equations are based on the uniform bond stress assumption along the embedded length notwithstanding the fact that the stresses are highly non-uniform, especially in the region between flexural cracks (ACI 408 R03 2003). This assumption per se is not a problem because as long as the equilibrium of the forces acting on the bar is satisfied by the assumed bond stress distribution and the stress does not exceed the maximum resistance of the interface, safety is assured, but depending on the assumed bond stress distribution, it can often lead to quite conservative results.

Furthermore, if both the shape and maximum value of bond stress become functions of the stress level in the bar, then extension of the development length provisions from one stress level to another will become quite cumbersome because to check the adequacy of the provided bond length at any section and stress level, in each case one must integrate the bond stress along the provided length.

Researchers have investigated the FRP rebar-concrete bond considering the characteristics of the bar, the concrete and the influence of the environment (Tighiouart et al. 1998; Tighiouart et al. 1999; Katz et al. 1999; Achillides and Pilaoutas 2004; Wambeke and Shield 2006; Lee et al. 2008; Davalos et al. 2008; Menezes et al. 2008; Hao et al. 2009; Baena et al. 2009; Esfandeh et al. 2009; Desnerck et al. 2010; Robert and Benmokrane 2010; Soong et al. 2011; Zhou et al. 2012; Mazaheripour et al. 2013; Lin and Zhang 2013; Yan et al. 2016; Rolland et al. 2018). Despite these studies, some issues have not been sufficiently investigated. Accordingly, the goals of the current research are (1) comparing the results of the RILEM and ACI beam-bond tests to determine if only one of the two methods can be adopted as the standard beam-bond test method in future investigations (2) the development length corresponding to different stress levels in the bar and the validity of the assumption of direct proportionality between the bar stress and its required development length, (3) examination of the degree of conservatism of some FRP design standards for bond and development length and the extent of the conservatism in relation to the bar stress, (4) the actual bond stress distribution along the bar embedded length and its effect on development length.

2. Experimental Program

2.1 Test Materials

The materials used in this investigation include GFRP, steel and concrete.

2.1.1 GFRP Bar

The GFRP bars had a nominal diameter of 15 *mm* and fibre content of 80% embedded in a

polyester resin. The bar had a ribbed surface, with rib height and pitch being 0.35 and 10 mm $\pm$ 10%, respectively. To obtain its tensile strength, 30 samples were tested using the recommendations of ASTM D7205/D7205M 2011. Based on the above sample size, the mean values of bar diameter, ultimate strength and strain at failure were determined to be 14.8 mm, 1066 MPa and 0.0213, respectively, with corresponding standard deviation of 0.03 mm, 60.96 MPa and 0.002.

2.1.2 Steel bar

The transverse reinforcement in the beam-bond specimens and the longitudinal hanger bars used to make the reinforcement cage were No.10 deformed steel bars. Three specimens were tested in accordance with ASTM A370 2014 to obtain their mechanical properties. Their average yield stress, ultimate strength and elastic modulus were determined to be 564 MPa, 735 MPa and 216 GPa, respectively.

2.1.3 Concrete

All specimens were made from the same batch of concrete obtained from a ready-mix plant and had specified compressive strength of 30 MPa, maximum aggregate size of 20 mm and slump of 100 mm. Concrete casting and finishing were performed per the recommendations of ASTM C192/C192M 2014. Average 28-day compressive strength of the concrete, obtained in compliance with ASTM C39/C39M 2014, was 35.2 MPa while the average split cylinder tensile strength was 2.4 MPa. At the time of performing the bond tests, concrete specimens reached average compressive and tensile strength of 36.5 MPa and 2.6 MPa, respectively. Following ASTM C469/C469M 2014 procedures, the concrete elastic modulus was determined to be 24 GPa, comparable to the expected value of 26 GPa calculated using CSA A23.3 2014.

2.2 Test specimens

The testing program involved four large scale beams and 10 pullout prisms. Two replicate beams, designated as BR1 and BR2, referred to as hinged beams, were constructed and tested in

compliance with the RILEM TC-RC5 1994 recommendations while the other two replicate beams, designated as BN1 and BN2, termed notched beams, were constructed and tested as recommended by ACI 208 1958. The ten replicate pullout prisms, designated as P1 to P10, were tested in compliance with CSA S806 2012 specifications.

With reference to Figure 1, the four test beams had identical dimensions, viz. 2000 *mm* length, 400 *mm* width and 500 *mm* height, with a clear cover of 35 *mm* over the GFRP test bar. Note that the choice of bar size, spacing and clear cover for the transverse reinforcement was based on the RILEM TC-RC5 specifications and the values of these parameters were kept the same for all four beams. It is worth mentioning that the presence of transverse reinforcement does not increase the flexural bond strength (Wambeke and Shield 2006) via concrete confinement because rectangular ties bow out at mid-length of their sides (Park and Paulay 1975). However, the ties mitigate the negative impact of bond splitting on flexural bond strength. Bar axial deformation was measured in the "Test End" via strain gauges installed along the 600 *mm* bonded length. Before applying the strain gauge to the bar, a thin layer of epoxy was applied on the bar surface at the location of the gauge installation and after it dried, it was leveled with two grades of sandpaper. Thereafter, the gauge was bonded to the levelled surface and was covered with a tape with foam backing to avoid damage to it during concrete casting.

Some beam-bond test guidelines, such as those by RILEM TC-RC5, allow collection of two sets of bond data from a single rebar, symmetrically bonded to the concrete along the two halves of the beam length, however, Moreno and Bastos 2006 found that the bond strength values obtained from the two halves of the beam were highly inconsistent, an occurrence that was attributed to the sudden deformation caused by bar pullout from one side of the beam. Hence, similarly to Mazaheripour et al. 2013, in this study, to force bond failure in the designated half of the beam,

i.e., the "Test End", different embedment lengths were used in the two halves of each beam.

For the notched beams, the height of the notch was 50 *mm* while its width, consistent with the gap in the hinged beams, was 60 *mm*. Note that longitudinal steel rebars running along the corner of the transverse reinforcement at the bottom of each beam were necessary to maintain the reinforcement cage rigidity during concrete casting, however, they were not bonded to the concrete and were terminated before midspan. With reference to Figure 1, debonding of the bottom longitudinal steel bars and segments of the GFRP bar was accomplished by wrapping a layer of electrical tape around them, followed by a layer of duct tape with the outside surface of the duct tape brushed with form oil.

The ten replicate pullout specimens, had cross section of 150 X 150 *mm* and embedment length of 60 *mm* or four times the bar nominal diameter, which were selected in accordance with CSA S806 recommendations.

3. Instrumentation

Strain gauges were used to capture the GFRP bar axial deformation along its 600 *mm* bonded length in the "Test End". To allow for direct comparison of the strain gauge data between the two test methods, the number and location of the gauges along the bar were maintained unchanged. With reference to Figure 1, beside the strain gauge at the centerline of the beam, designated as SG_C , four strain gauges labeled from the loaded-end, SG_1 , SG_2 , etc., were placed along the 600 *mm* embedded length at 200 *mm* spacing. In addition, LVDTs labeled $L1$, $L2$, etc. were used to measure the test bar slip at its loaded and unloaded ends in both the pullout specimens and the test beams. Finally, three string pot potentiometers were used to measure each beam's vertical deflection at its quarter points and midspan.

4. Test procedure

As indicated in Figure 1, the beams were tested under four-point bending and the load was applied via a servo-controlled hydraulic actuator attached to a load cell. Since hinged beams were pre-cracked at midspan, they were tested via displacement control at a constant rate of 0.5 mm/min . The notched beams were tested under load-control at the rate of 5 kN/min up to the onset of flexural cracking, thereafter, the loading switched to displacement-control at an initial rate of 1 mm/min with subsequent rapid decrease to 0.5 mm/min . The loading was then kept constant at 0.5 mm/min up to failure.

For the pullout specimens, the loading rate was 1.27 mm/min , which corresponds to the maximum rate allowed by CSA S806. During testing of each specimen, an automatic data acquisition system recorded data from the load cell, strain gauges, LVDTs and string-pot potentiometers at a rate of 0.5 second.

5. Experimental Results

5.1 Failure mode

As intended by design, all four beams experienced flexural tension failure initiated by rupture of the GFRP bar. The overall behaviour of the two sets of beams in terms of maximum moment, $M_{max}^{mid.}$ as well as the corresponding midspan deflection, $\Delta_{max}^{mid.}$, and the uncorrected loaded-end bar slip, s_{uc}^l , were comparable, with the notched beams on average having slightly higher moment capacity than the beams with the midspan hinge. The 600 mm embedment length was adequate since the bar achieved its ultimate capacity, without any slip being recorded by the LVDTs $L2$ to $L4$ placed at the unloaded ends of the beams. As can be noticed in Figure 2(a), BR1 exhibited a non-midspan flexural crack at its "Non-test End" while no visible cracks were observed in BR2 until failure. Both notched beams had the characteristic midspan flexural crack, Figure 2(b) and 2(d), that initiated at one of the corners of the notch and propagated upward. At failure, the depth of the uncracked compression zone, observable by naked eye, diminished to 50 and 40 mm in

BN1 and BN2, respectively. These beams also experienced another crack at their "Non-test End". Rupture of the GFRP bar occurred at or near midspan, but post-test exposure of the GFRP bar in the maximum moment region revealed that besides midspan, the most significant damage to the bar, in terms of its fibre detachment, occurred in the "Non-test End" part of the beams.

All ten pullout specimens experienced pullout failure due to their short embedment length of 150 mm and the LVDTs at both ends of the bar recorded substantial slip. The average maximum pullout load along with its corresponding bond stress and corrected loaded-end bar slip were 40.1 kN, 14.4 MPa and 4.5 mm, respectively, with corresponding standard deviation of 5.5 kN, 2.0 MPa and 0.5 mm.

5.2 Beam load-deflection behaviour

Figure 3 shows the load-midspan deflection curves of the hinged and notched beams. The notched beams had on average 10% higher moment capacity and 0.05 mm more deflection than the hinged beams at the same load level. Neither of these small variations is the result of differences in the bond behaviour of the two types of beams and the testing procedure. The differences are mainly due to the presence of the full-depth crack and the hinge at the midspan section in BR1 and BR2.

5.3 Strain distribution along the bar embedded length

To examine the strain distribution as a function of the bar stress level, which is a function of the applied load level, the strain data at 10% increments of each specimen's rupture load, P_{max} , were extracted and are plotted in Figure 4. With reference to this figure, prior to the midspan flexural cracking, below approximately 50% of P_{max} , the bar strain in the notched beams is practically negligible due to the small flexural rigidity of the GFRP bar compared to that of the uncracked concrete section. Based on the shown distributions, it could be argued that the development length for this GFRP bar is slightly larger than 400 mm but noticeably less than 600 mm.

6. Analysis and Discussion of Test Results

6.1 Flexural response of the test beams

Table 1 provides a comparison among the maximum moment at midspan, $M_{max}^{mid.}$, midspan cracking moment, $M_{cr}^{mid.}$, non-midspan cracking moment $M_{cr}^{non-mid.}$ and the midspan deflection at maximum moment $\Delta_{max}^{mid.}$ obtained experimentally. The table also shows the theoretical moment capacity of the two sets of beams and the computed cracking moment of the notched beams. The theoretical moment capacity of BR1 and BR2 was obtained by multiplying their measured moment arm of 433 and 429 mm, respectively, by the 183.4 kN average rupture strength of the GFRP bar obtained from direct tensile tests. Note, the internal moment arm, designated as z , is shown in Figure 1. The lower theoretical moment capacity of the RILEM beams than that of the notched beams is due to their smaller internal moment arm. For the notched beams, the theoretical calculations were based on strain compatibility analysis and the assumption of perfect bond. Notice that the computed and measured moments are in reasonable agreement and the differences can be explained by the usual variability in the bar and concrete strengths. Thus, the assumption of full bond used in the calculation seems reasonable and the lack of measured slip at the bar unloaded ends further supports this assumption.

Due to the constant internal moment arm of the hinged beams, at any load level, the GFRP bar stress at midspan is linearly proportional to the externally applied moment at the same location. Thus, one can calculate the stress corresponding to increasing moment, and plot it against the companion strain measured by the midspan gauge. The slope of the resulting stress-strain curve gives the elastic modulus of the bar, which was found to be 68 GPa. This value is used in all the relevant theoretical calculations in the present work. It should be mentioned that Benmokrane et al. 1996 used a similar procedure for calculating the bar elastic modulus.

Note that based on the shear span to depth ratio of the test beams, one might suspect that deep

beam action dominates their behaviour, but as shown in Table 1, the notched beams calculated ultimate moment capacity, based on conventional beam theory, is only 0.5% higher than the average of their observed strengths. In the case of a deep beam, the GFRP bar would serve as a tie, and according to the strut-and-tie model, the tie force at the support node equilibrates the horizontal component of the compression force in the inclined strut. Based on this model a large tie force must exist at failure. For example, in BN1, this force needs to be roughly 172 kN, which corresponds to over 90% of the bar rupture stress. However, the bar is debonded at the support and is stress free while the measured strain in the embedded part near the support is also practically zero at all load levels in all four beams, an observation that is consistent with conventional beam theory. Hence, it is reasonable to assume that despite the absence of multiple flexural cracks, the beams' behaviour follows conventional flexural theory.

6.2 Measured versus computed development length

The ACI 440.1, CSA S806 and CSA S6 standards provide the required development length based on the maximum bond stress that would initiate bond loss by pullout and/or concrete splitting. According to ACI, as recommended by Wambeke and Shield 2006,

$$l_d = \frac{\left(\alpha \frac{f_F}{0.083\sqrt{f'_c}} - 340 \right)}{\left(13.6 + \frac{C}{d_b} \right)} d_b \quad (1)$$

The CSA S806 and CSA S6 provide the semi-empirical development length equations as given by Eqs. (2) and (3), respectively.

$$l_d = 1.15 \frac{k_1 k_2 k_3 k_4 k_5 f_F}{d_{cs} \sqrt{f'_c}} A_F \quad (2)$$

$$l_d = \frac{0.45k_1k_4}{\left(d_{cs} + k_{tr}\frac{E_F}{E_s}\right)} \frac{f_F}{f_{cr}} A_F \quad (3)$$

Inserting the GFRP bar and the beam parameters in this study into the relevant development equations, the ACI 440, CSA S806 and CSA S6 expressions result in Eqs., (4), (5) and (6), respectively.

$$l_d = (1.8012)f_F - 304.98 \quad l_d \geq 20d_b \quad (4)$$

$$l_d = (0.8981)f_F \quad l_d \geq 300 \quad (5)$$

$$l_d = (0.6973)f_F \quad l_d \geq 250 \quad (6)$$

Note that contrary to the two CSA standards relations, the ACI expression does not make the bar development length directly proportional to its stress. Since in FRP reinforced concrete flexural members, failure is not supposed to be initiated by rupture of the bar, the maximum bar stress at member failure will be less than the bar tensile strength. Hence, in practice the development length needs to be determined for a bar stress less than its strength. To get a sense of the required development length for the different levels of maximum stress in the bar, in Table 2, the required development lengths at 10% increments of P_{max} are estimated using the measured strain values and are compared with the corresponding computed values using the latter three equations.

It is evident that irrespective of the test method, the standards' recommended values are comparable to the experimental values at the beginning (up to 40% of bar strength), but with increasing bar stress, they start to deviate significantly. The ACI 440 requires much longer length than the other two standards as the stress level in the bar keeps increasing. The CSA S6 gives more reasonable estimates than the other two standards. The difference between the two CSA standards values is

primarily due to the imposition of the 5 MPa limit on the $\sqrt{f'_c}$ by CSA S806, irrespective of the actual concrete strength. This limit needs to be revisited to improve the accuracy of the recommended expression for the development length. If this limit is disregarded, the computed lengths by the CSA S806 would drop by 20% and would become closer to the value given by CSA S6.

On the other hand, to develop the bar stress less than 50% of its strength, a stress level which may not be uncommon at failure of FRP reinforced concrete members, the CSA S6 values may not be sufficiently conservative. Consequently, more detailed studies are required in this regard because FRP bar rupture is not the preferred failure mode in FRP reinforced concrete design and obtaining development values for reaching bar rupture may not be relevant in many applications. For developing bar stress up to 70% of its strength, the CSA S806 recommended values seem quite reasonable.

Finally, for the Notched beams, the onset of beam cracking between 40% to 50% of P_{max} transfers a significant amount of tensile stress from the concrete to the GFRP, therefore, it warrants longer development length. However, compared to CSA S806 and CSA S6, the nearly doubling of the required development length by the ACI 440 seems unexpectedly high, the basis for which requires explanation. It is also important to compare the required development length at 20% of its strength with the value obtained in the pullout tests because pullout occurred at 20% of the bar strength. Clearly, the results of the pullout test do not agree with those of the beam tests. This means that the pullout test results cannot be extrapolated to the beams and its utility for design purposes is questionable.

6.3 Actual versus the assumed uniform bond stress distribution

Based on the strain data gathered along the embedded length in the current test beams, the bond stress variation along the bar can be estimated using Eq. (7). Given the positioning of the strain

gauges along the embedment length this equation produces average experimental bond stress values at 100, 300 and 500 *mm* from the loaded-end.

$$\mu_j = \left(\frac{|\varepsilon_{SGi} - \varepsilon_{SGi+1}|}{\Delta l} \right) \left(\frac{E_b A_b}{\pi d_b} \right) \quad (7)$$

Figure 5 shows the bond stress distribution for the four test beams at various stress levels in the GFRP rebar and the corresponding assumed uniform bond stress. It is evident that the actual bond stress distribution along the embedment length is nonlinear and it seems that for bar stresses up to 80% of strength, the maximum stress occurs at nearly the same distance from the loaded-end; thereafter, the shape changes and the location of maximum stress shifts away from the loaded-end. This shift is likely due to the slip occurring in the region near the bar loaded-end. This phenomenon has been observed by previous researchers (Perry and Thompson 1966; Benmokrane et al. 1996; Feldman and Bartlett 2007). Consequently, if the maximum design stress in the bar is less than 80% of its strength, it is appropriate to focus on the shape of the bond stress curve at or below this stress level.

With reference to Figure 5, at $P_{\max}$, the bond stress, $\mu_{\max}$ for BR1 and BR2, was 9.7 and 12.2 *MPa*, respectively while for BN1 and BN2, the corresponding values were 9.7 and 15.8 *MPa*. As observed in the figure, for BR1, the location of the mentioned bond stress occurs at 100 *mm* from the loaded-end while for the rest of the beams this location is 300 *mm* from the loaded-end. Obviously, these values are significantly larger than the average value computed based on the recommendations of each design standard. Using the results presented in Figure 5, one can obtain the average uniform bond stress, μ_{avg}^u , in each beam and then compare it with the values recommended by the standards, $\mu_{\text{avg}}^{\text{code}}$. Table 3 presents such a comparison and it is evident that all three standards, particularly the ACI 440.1, underestimate the actual bond stress, although

overall the CSA S6 recommendation seems reasonable, particularly if factors such as long-term performance and certain epistemic uncertainties are considered. Note that μ_{avg}^u is obtained by using the bar stress at failure equal to f_{Fu} and the embedment length, l_e equal to 600 mm while μ_{avg}^{ucode} is calculated in a similar fashion but l_e is determined using Eqs. (4), (5) and (6), respectively.

6.4 Proposed Bond Stress Distribution Equation

In some analytical and numerical analyses, one needs more accurate representation of the bond stress distribution. In the following, a new relationship is proposed, and it is compared with the current test data. Theoretically, the bond stress can be expressed as:

$$\mu(x) = \left(\frac{d\epsilon_b}{dx} \right) \left(\frac{E_b A_b}{\pi d_b} \right) \quad (8)$$

Note that x is the distance from the loaded-end of the embedment length. Observation of the strain variation in Figure 4 may initially appear to follow an exponential function, but the strain distribution in some specimens, such as BR2 and BN2 at P_{max} suggest a pattern that does not resemble any exponential form. In BR2 and BN2 at P_{max} , the extent of bond degradation has nearly reached the first 200 mm and as a result, the difference between the strain readings of SG_1 , located at the start of the embedment, i.e., the loaded-end, and SG_2 , positioned 200 mm from the loaded-end decreased. Observe that all of the specimens undergo a certain degree of bond degradation with increase in bar stress, however, depending on the spacing of the strain gauges in the experimental program, the extent of this bond degradation may not be recognized by analysis of the distribution plots. Considering the patterns obtained in this experimental program as well as those provided by researchers in the past (Perry and Thompson 1966; Benmokrane et al. 1996; Feldman and Bartlett 2007), it is suggested that the actual strain distribution be taken as indicated in Figure 6. Based on the preceding bar strain profile, the bar strain, ϵ_b , variation along its embedded length can be represented by a modified logistic growth function (MLGF) as given by

Eq. (9). This expression can accommodate the expected bond degradation that occurs and in addition is able to capture the rapid decrease in strain beyond the point of significant bond degradation.

$$\varepsilon_b(x) = \frac{1}{a + be^{cx}} \quad (9)$$

Note that e is the base of natural logarithm and the function contains three parameters, a , b and c . To obtain a unique solution, two parameters need to be expressed in terms of the third one and this can be achieved by the constraints of $\varepsilon_b(0)$, the strain at the onset of the embedment from the loaded-end and $\varepsilon_b(x_2)$, the strain at a known position x_2 along the embedment length from the loaded-end which results in:

$$a = \frac{1}{\varepsilon_b(0)} - b \quad (10)$$

$$c = \frac{\ln \left[\frac{1/\varepsilon_b(x_2) - 1/\varepsilon_b(0) + b}{b} \right]}{x_2} \quad (11)$$

Note that due to the nature of the MLGF, if the strain at a chosen point along the embedment length is zero, then a negligible non-zero value must be assigned to it. Inserting Eqs. (10) and (11) into Eq. (9), results in an expression that is a function of x and the constant b which can be optimized using the measured strain data.

Figure 7 presents the experimental data points at P_{max} captured by the strain gauges, the MLGF curves and the segmentally linear fits based on assumed uniform bond stress distribution between two consecutive gauges spaced at 200 mm. With reference to this figure, the values of the parameters, a , b and c generating these curves were based on the constraint choice of 0 mm and 400 mm from the loaded-end.

The derivative of the function in Eq. (9), presented in Eq. (12), can be inserted into Eq. (8) to obtain an expression, i.e., Eq. (13), which defines the bond stress distribution as a function of the distance from the loaded-end. The proposed MLGF, is not expressed as a function of bar slip but its shape can accommodate the extent of bond degradation.

$$\left| \frac{d\varepsilon_b}{dx} \right| = \frac{bce^{cx}}{(a + be^{cx})^2} \quad (12)$$

$$\mu(x) = \frac{bce^{cx}}{(a + be^{cx})^2} \left(\frac{E_b A_b}{\pi d_b} \right) \quad (13)$$

Using Eq. (13), the analytical bond stress distribution at P_{max} is presented in Figure 8 and for comparison, the uniform bond stress distributions between two consecutive strain gauges, based on the assumption of linear strain variation between the two gauges, is also plotted. It can be observed that the MLGF generated curves are much closer in shape to the experimentally observed curves. For bar equilibrium to be satisfied, the area under the graph of the bond stress curve A_{gr} should be equal to the measured force, F_b in the bar. As evident in Figure 8, F_b from the MLGF (obtained by numerical integration using the Simpson's rule) and from the uniform bond stress distribution are both comparable to the 183.4 kN direct tensile strength of the bar. With reference to BN2 in Figure 4, the strain values at 0 mm and 200 mm from the loaded-end are converging, indicating that debonding has occurred within this region, which is manifested by the sharp decline in the curve fitted to the strain data. This occurrence is responsible for the high bond stress in Figure 8. Note, Eq. (13), which depends on the derivative of Eq. (9), was used to produce the bond stress curves in Figure 8. Ideally one needs to obtain strain data at short intervals, but surface strain gauges disturb the bond stress flow and applying too many gauges may significantly distort the bond stress distribution. To avoid this problem, in the case of steel rebars, some investigators (Feldman & Bartlett 2007) drilled a hole into the bar cross-section, split it into two halves, installed

the gauges on the inside and rejoined the two by welding, but this procedure cannot be applied to FRP bars. Although the number of measured strain points may not be sufficient to capture the actual strain variation of the bar with complete certainty, still, the shown curve fits reasonably well with the measured data points. Furthermore, the nearly similar variation reported by Perry and Thompson 1966, and Benmokrane et al. 1996 supports the assumption of a nonlinear variation. The linear variation corresponds to a uniform bond stress, which based on the current tests requires a longer development length than necessary.

7. Conclusions

The bond performance of a GFRP bar is investigated using two types of beam-bond tests, with focus on the bond stress distribution along the bar embedded length. The results support the following conclusions:

1. The notched beam-bond test and the hinged beam test, give similar results, but due to the easier construction of the ACI recommended test specimens, the use of the notched beam test is recommended.
2. The test bar achieved its full strength, without pullout, with a provided embedment length of 600 mm but the two CSA standards for FRP design and the ACI guidelines overestimated its required development length. Overall, the CSA S6 recommendations seem reasonable while those of the ACI appear to be highly conservative, requiring more than 2.5 times the provided length. The CSA S806 required length was roughly 25% higher than that required by the CSA S6, mainly due to limiting $\sqrt{f'_c}$ to maximum 5 MPa, irrespective of the concrete strength. Therefore, this limit needs to be revisited to reduce the conservatism of the standard.
3. In conformity with the findings of previous researchers, a state of uniform bond stress was

not observed at any load level.

4. The strain distribution along the embedment length of a GFRP tensile rebar is highly nonlinear. Based on the strain values measured along the embedded length of the bar, it is found that the distribution fits well to a modified form of the logistic growth function. However, due to the relatively small number of strain measurement points in this study, further investigation into the general applicability of this function is warranted, especially in slender beams experiencing multiple distributed flexural cracks.
5. The derivative of the proposed modified logistic growth function, which is theoretically proportional to the bond stress, exhibits the bond stress distribution to be far from uniform and highly nonlinear, but in view of the small number of points along the embedded length where strain was measured, further investigation is needed to confirm the generality of the proposed distribution.

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2 Table 1: Comparison of experimental and theoretical flexural response of test beams

Beam Designation	Experimental				Theoretical	
	$M_{max}^{mid.}$	$M_{cr}^{mid.}$	$M_{cr}^{non-mid.}$	$\Delta_{max}^{mid.}$	$M_{max}^{mid.}$	$M_{cr}^{mid.}$
	(kN.m)		(mm)		(kN.m)	
BR1	77.2	N/A	57.6	15.1	79.3	N/A
BR2	72.8	N/A	None	13.9	78.6	N/A
BN1	80.2	40.1	66.8	13.7	82.0	48.2
BN2	83.0	39.0	61.1	15.4	82.0	48.2

3 **Note.** N/A = Not Applicable as BR1 and BR2 were precracked at midspan

4 Table 2: Development length relation to bar maximum stress

	Required Development length l_d (mm)							
	Experimental		ACI 440		CSA S806		CSA S6	
	RILEM beams							
% of P_{max}	BR1	BR2	BR1	BR2	BR1	BR2	BR1	BR2
10	< 200	< 200	296	296	300	300	250	250
20	200 to 400	200 to 400	296	296	300	300	250	250
30	200 to 400	200 to 400	309	296	306	300	250	250
40	200 to 400	200 to 400	502	434	402	368	312	286
50	200 to 400	400 to 600	702	618	502	460	390	357
60	400 to 600	400 to 600	920	806	611	554	474	430
70	400 to 600	400 to 600	1,097	976	699	639	543	496
80	400 to 600	400 to 600	1,294	1,154	797	728	619	565
90	400 to 600	400 to 600	1,489	1,332	895	816	695	634
100	400 to 600	400 to 600	1,685	1,511	992	905	770	703
ACI Notched beams								
% of P_{max}	BN1	BN2	BN1	BN2	BN1	BN2	BN1	BN2
10	< 200	< 200	296	296	300	300	250	250
20	200 to 400	200 to 400	296	296	300	300	250	250
30	400 to 600	200 to 400	296	296	300	300	250	250
40	400 to 600	200 to 400	296	296	300	300	250	250
50	400 to 600	400 to 600	626	661	464	482	360	374
60	400 to 600	400 to 600	820	844	561	573	435	445
70	400 to 600	400 to 600	1,007	1,052	654	677	508	525
80	400 to 600	400 to 600	1,194	1,246	748	774	580	601
90	400 to 600	400 to 600	1,381	1,440	841	870	653	676
100	400 to 600	400 to 600	1,570	1,635	935	967	726	751

Table 3: Experimental versus theoretical average bond stress assuming uniform bond stress at bar failure

Specimen Designation	$\mu_{avg.}^u$	$\mu_{avg.}^{u_{code}} \text{ (MPa)}$			$\mu_{avg.}^u / \mu_{avg.}^{u_{code}}$		
		(ACI 440.1, 2006)	(CSA S806, 2012)	(CSA S6, 2006)	(ACI 440.1, 2006)	(CSA S806, 2012)	(CSA S6, 2006)
BR1	6.8	2.4	4.1	5.3	2.8	1.7	1.3
BR2	6.2	2.5			2.5	1.5	1.2
BN1	6.4	2.5			2.6	1.6	1.2
BN2	6.6	2.4			2.8	1.6	1.2

Note. Stresses in MPa

(a)

(b)

(c)

(d)

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Figure 2: Beam-bond test specimens after failure

(a)

(b)

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Figure 3: Load-midspan deflection curves

(a)

(b)

(c)

(d)

Figure 4: Strain distribution along the embedded length of beam-bond specimens

(a)

(c)

(b)

(d)

Figure 5: Experimental versus assumed uniform bond stress distribution

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Figure 6: Suggested actual strain distribution

(a)

(b)

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Figure 7: Experimental data, MLGF curves and segmentally linear fits: (a) RILEM beams; (b) Notched beams

(a)

(b)

Figure 8: MLGF versus segmentally uniform bond stress distribution: (a) RILEM beams; (b) Notched beams