THE ROLE OF THE BOND ON THE STRUCTURAL BEHAVIOUR OF FLEXURAL FRP REINFORCED CONCRETE MEMBERS

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ABSTRACT

The paper focuses on the bond between fibre-reinforced polymer (FRP) reinforcements and concrete including its modelling (local bond-slip law) and influence on the structural behaviour of FRP reinforced concrete members. The analysis, both theoretical and experimental, refers to flexural concrete beams upgraded by externally bonded FRP sheets, considering different bonding systems such as the commonly used resin-FRP system and novel bonding technologies (near surface mounted system, cementitious-FRP system). The structural behaviour of strengthened beams is analyzed by means of a non-linear model derived from a cracking analysis based on slip and bond stress. By using some bond-slip models, the performances of beams under service conditions (cracking, deformability) are evaluated varying parameters governing the FRP-to-concrete bond behaviour. Results of the analysis furnish useful information to find interface bonding systems which can offer suitable bond characteristics to optimize the performances of FRP reinforced concrete members.

KEYWORDS

FRP, RC beams, strengthening, bond, structural behaviour.

INTRODUCTION

The use of fibre-reinforced polymer (FRP) materials for reinforcing, strengthening or retrofitting concrete structures has extensively increased in the last decades due to its relevant advantages over the conventional materials. Researches carried out on this topic have shown that the use of FRP can enhance both serviceability and ultimate limit state of concrete elements. The performances of concrete structures, however, are different varying the purpose of the FRP’s use. When FRPs are used in form of rebars as internal reinforcements of concrete structures in substitution of traditional steel rebars, in fact, a remarkable improvement of the ultimate strength, an increase of the deformability and a decrease of the ductility, are observed (Aiello and Ombres, 2000). On the contrary, using the FRPs as sheets or plates externally bonded to reinforced concrete structures, if premature failure mechanisms are avoided, enhances both serviceability and ultimate strength (Teng et al., 2002a). In strengthening or retrofitting existing reinforced concrete structures different bonding techniques can be used: the well-known system in which an adhesive (polymeric resin) is used to bond the FRP plates/sheets to the concrete substrate, the Near Surface Mounted system in which FRP rods or strips were placed into grooves precut into the concrete cover (Taljsten et al., 2001), and the novel system in which a special cementitious mortar is used to bond an FRP mesh to the concrete substrate (Aiello et al., 2005). The success of the FRP technique is strictly related to the bond between the FRP materials and the concrete. A lack of bond, in fact, produces high slip values between FRP and concrete and, consequently, an increase of the deformability of concrete members and premature debonding of the FRP sheets from the concrete. Researches have shown that bond properties of FRPs are dependent on numerous parameters both geometrical and mechanical such as the outer surface treatment and diameter of rebars, thickness and mechanical properties of the adhesive, concrete strength, environmental conditions. Extensive studies and researches have been carried out during the last decade on the analysis of the bond properties of FRPs. Among those experimental studies on FRP rebars (Benmokrane et al., 1998; Cosenza et al., 1997), externally bonded FRP sheets (De Lorenzis et al., 2001; Teng et al., 2005) and FRP NSMR (De Lorenzis and Nanni, 2002); theoretical and numerical studies (Smith and Teng, 2001; Teng et al., 2002b; Aprile et al., 2001). Many Codes provisions are, also, available (ACI, 2002; CEB, 2001; CNR, 2004). A good understanding of the bond properties is, then, a fundamental requisite for a reliable and rational design of concrete structures reinforced or strengthened with FRPs. At this aim becomes essential both the analytical modelling of the interface bond between FRP and concrete substrates and the definition of adequate methodologies for the structural analysis of concrete members. Traditional methodologies referring to steel reinforcements are, in fact, unable to predict the real behaviour of concrete members reinforced or
strengthened with FRP materials (Aiello and Ombres, 2000). The paper, focuses on the bond between fibre-reinforced polymer (FRP) reinforcements and concrete including its modeling (local bond-slip law) and influence on the structural behavior of concrete beams upgraded by externally bonded FRP sheets, considering different bonding systems. Some existing bond-slip models were considered and the performances of concrete beams under service conditions (deflections, cracking) were analyzed. The structural analysis was carried out by means of a non linear model (Aiello and Ombres, 2004) derived from a cracking analysis founded on slip and bond stresses. Analytical predictions were compared with experimental results available in the literature. Results of the analysis furnish useful information to find interface bonding systems which can offer suitable bond characteristics to optimize the performances of FRP reinforced concrete members.

BONDING SYSTEMS AND STRUCTURAL BEHAVIOUR OF FRP STRENGTHENED RC BEAMS

Commonly, FRP reinforcements in form of sheets or plates are externally bonded to the concrete by means of an adhesive (polymeric resin); this system is easy to apply and furnishes good results both in terms of strength and durability. The behaviour of strengthened beams, however, involves premature debonding of the FRP sheet from the beams; debonding failure modes are brittle and related to the interfacial stresses between the FRP and the existing concrete. The structural behaviour of reinforced concrete (RC) beams upgraded with FRP sheets bonded with adhesive has been well established by means of a large number of experimental and theoretical researches. The mechanisms involved in the premature failure modes and their effects on strength capacity and ductility of strengthened beams are well known; analytical procedures to determine the stresses at the interface are well defined and can be used in design (Teng et al., 2005). Codes provisions are also available. An analogous behaviour characterizes the structural behaviour of RC beams upgraded with FRP rebars or strips bonded by the Near Surface Mounted system. Compared to the sheet bonding system, the NSM system furnishes higher fracture energy at failure and a better protection against fire, vandalism and impact (Taljsten et al., 2001).

Recently a novel bonding system was proposed to upgrade RC beams (Aiello et al., 2005). The system (Fiber Reinforced Cementitious Mortar, FCRM) is composed of a Carbon FRP mesh, made of filaments of nominal thickness 0.047 mm, oriented at 0°/90°, at a distance of 100 mm, embedded into an inorganic stabilized matrix made of pozzolanic hydraulic binder and specific additives. The matrix, mixed with water, acts as a binding agent between the carbon fiber mesh and the concrete substrate. The FRCM system is applied to the concrete substrate in two phases: a first layer of mortar is applied on the surface, then, the carbon fiber net is placed on the mortar bed with a slight pressure and a subsequent layer of mortar is placed as a cover. Following the same procedure it is possible to place other carbon fiber nets. Compared with the other two bonding systems, previously described, the FRCM system can result more invasive with higher value of the thickness. Some experimental studies on RC beams upgraded with the FRCM system have been carried out (Di Tommaso et al., 2004, Aiello et al. 2004). Results obtained by tests are the following: i) in all tests only flexural failure modes were observed (i.e. premature failure modes are avoided); ii) the increase in strength capacity was limited (10-20% respect to values corresponding to unstrengthened beams); iii) a relevant ductility was observed.

In the Figures 1 and 2 are shown the failure modes of a RC beam upgraded with the adhesive and FRCM systems (Aiello et al. 2005); in the first case the failure was due to debonding, while a flexural failure mode was observed for the FRCM strengthened beam. Even if further researches are needed, it is evident as the structural
MODELLING OF THE FRP-TO-CONCRETE BOND

As experimentally evidenced, the FRP-to-concrete bond behaviour is governed from some geometrical and mechanical parameters such as the concrete strength, the adhesive stiffness, the adhesive strength and the FRP sheet axial stiffness. When a polymeric resin is used as adhesive, the failure at the interface FRP-concrete is by cracking of the concrete adjacent the adhesive layer; this is due to mechanical properties of the adhesive that are higher than that of the concrete substrate. For each load level, experimental tests evidence the presence of slips between FRP and concrete substrate strictly related to the shear deformability of the adhesive layer. As a consequence local bond-slip relationships, derived from experimental results, are representative of the behaviour at the interface between FRP sheets and concrete. Because the difficulties in defining the local bond-slip curves directly from tests (pullout tests), generally the interface behaviour is described by bond-slip models developed on the basis of strain measurements or load-slip curves. Several bond-slip curves have been proposed from researchers; all these curves present an ascending branch until to reach the maximum bond stress, \( \tau_{\text{max}} \), at the interface and a descending branch ending at the maximum slip value, \( u_{\text{s}} \), corresponding to the zero value of the bond stress. Both branches of the bond-slip curves are non linear; however to simplify the analysis some models adopt a linear shapes both for ascending and descending branches. In the literature have been proposed other models such as a brittle linear model (Neubauer and Rostasy, 1999) and elasto-plastic or rigid-plastic models; these models, however, are unrealistic and do not furnish correct predictions. An exhaustive and accurate review of existing bond-slip models was presented in a recent paper (Teng et al. 2005). For the analysis described in this paper the bond-slip models adopted are: the Savoia et al. model (Savoia et al., 2003), the Teng et al. simplified model (Teng et al., 2005), the Monti et al. model (Monti et al., 2003) and the Ueda et al. model (Ueda et al., 2005). The models by Savoia et al. and Ueda et al. are of non linear type and analytically described by a single equation; models of Monti et al. and Teng et al. are of bilinear type.

<table>
<thead>
<tr>
<th>Bond-slip model</th>
<th>Ascending branch ( u \leq u_0 )</th>
<th>Descending branch ( u &gt; u_0 )</th>
<th>( \tau_{\text{max}} )</th>
<th>( u_s )</th>
<th>( u_f )</th>
<th>( \beta_e )</th>
</tr>
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<tbody>
<tr>
<td><strong>Bilinear curve</strong></td>
<td>( \tau_{\text{max}} w/ u_0 )</td>
<td>( \tau_{\text{max}} (u_s - u)/(u_s - u_0) )</td>
<td>1.8 ( \beta_e ), ( f_c )</td>
<td>2.5 ( \tau_{\text{max}} ), ( A )</td>
<td>0.33 ( \beta_e ), ( B )</td>
<td></td>
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<tr>
<td><strong>Teng et al.</strong></td>
<td>( \tau_{\text{max}} w/ u_0 )</td>
<td>( \tau_{\text{max}} (u_s - u)/(u_s - u_0) )</td>
<td>1.5 ( \beta_e ), ( f_c )</td>
<td>0.0195 ( \beta_e ), ( f_c )</td>
<td>2G/ ( \tau_{\text{max}} )</td>
<td></td>
</tr>
<tr>
<td><strong>Savoia et al.</strong></td>
<td>( \tau_{\text{max}} (w/ u_0)(2.86/(1.86 + (u/ u_0)^{2.86})) )</td>
<td></td>
<td>3.5 ( f_c ), ( ^{3.19} )</td>
<td></td>
<td>0.051</td>
<td></td>
</tr>
<tr>
<td><strong>Ueda et al.</strong></td>
<td>2KG/ ( e^{u_u/(u_f - u_0)} )</td>
<td></td>
<td></td>
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Parameters involved in the equations are: \( A = (t_f/E_f + 50/E_c) \), \( B = ((1.5 - (2b_2/100))/((1-b_1/100)))^{0.5} \), \( G_f = 0.308 C f_{c'}^{0.5} \), \( C = ((0.25-b_1/b_2))/((1.25 + b_2/b_1))^{0.5} \), \( K = 4.446 (G_f t_f)^{0.5} \), \( T = 6.846 (E_f t_f)^{0.5} \), \( E_t = 10^{6} \), \( G_t = 10^{6} \), \( f_r \) the compression and tensile strength of the concrete, \( t_f \) and \( t_t \) the thickness of the adhesive and the FRP sheet, \( b_1 \) and \( b_2 \) the width of the FRP sheet and the concrete, \( E_t \) and \( E_c \) the elastic modulus of the FRP sheet and the elastic modulus of the concrete, \( E_a \) and \( G_a \) the elastic modulus and the shear modulus of the adhesive, respectively. The use of the FRCM bonded system in which the tensile strength of the cementitious mortar, eventually added with short randomly distributed fibers, is higher than that of the existing concrete, allows to sensibly reduce the slips between the FRP sheet and the substrate. Slips between the FRP and the cementitious mortar are too low and can be neglected; in addition, as experimentally evidenced (Di Tommaso et al., 2004; Aiello et al. 2005), debonding failure modes are avoided. For this reason the structural analysis of reinforced concrete beams strengthened with the FRCM system can be carry out by using a bond strength model, that is, supposing a perfect bond between strengthening system and existing concrete.

THE STRUCTURAL ANALYSIS

Due to the presence of slips between the FRP sheets and the concrete substrate, the structural analysis of reinforced concrete beams strengthened with FRP sheets bonded with an adhesive layer, imposes the use of models founded on a bond-slip analysis. A non linear model, extensively described in previous papers (Aiello and Ombres, 2000; Aiello and Ombres, 2004), founded on a block model approach, is used in this paper. To analyse the structural behaviour, some regions are considered in the beams: the uncracked regions \( (M < M_{cr}, \) being \( M \) the applied bending moment and \( M_{cr} \) the first cracking bending moment) in which a perfect bond between the reinforcements and the concrete is assumed, and the cracked regions in which the influence of the
slip between reinforcements and concrete is considered. In the cracked regions, the model, derived from a cracking analysis based on slip and bond stress, refers to a beam element between two consecutive cracks (block) subjected to a constant bending moment higher than the first cracking bending moment. The analysis refers to the stabilized crack formation phase and considers two limit cracking configurations corresponding to the maximum and minimum crack spacing that bound all possible cracking configurations. The following equations are used to solve the structural problems:

**i) Equilibrium conditions on the cross-sections (translational and rotational)**

\[
\int_{\Omega_c} \sigma_c \, d\Omega + \sum_{i=1}^{n} \sigma_{ri} y_i \, d\Omega = 0
\]

\[
\int_{\Omega_c} y \sigma_c \, d\Omega + \sum_{i=1}^{n} \sigma_{ri} y_i \, d\Omega = M
\]

where \(c\) is the concrete area, \(ri\) is the area of the \(i\)th reinforcement (FRP sheet, steel rebars), \(yi\) is the distance between the neutral axis of the cross section and the centroid of the \(i\)th reinforcement. The section is divided into finite layers for integration.

**ii) Strain compatibility between two points, initially fully bonded, belonging to the steel rebar and the concrete**

\[
u_s(z) = \frac{d u_s}{d z} = \epsilon_s(z) - \epsilon_{cts}(z)
\]

and to the FRP sheet and the concrete

\[
u_r(z) = \frac{d u_r}{d z} = \epsilon_r(z) - \epsilon_{ct}(z)
\]

where \(u_s(z)\) is the slip between the concrete and the steel rebar; \(\epsilon_s(z)\) and \(\epsilon_{cts}(z)\) are strains of the steel rebar and the concrete in tension at the level of the steel rebars, respectively; \(u_r(z)\) is the slip between the concrete and the FRP sheet; \(\epsilon_r(z)\) and \(\epsilon_{ct}(z)\) are strains of the FRP sheet and the concrete at the tensile side of the cross-section, respectively.

**iii) Longitudinal stress equilibrium of the concrete reinforcements:**

\[
\sigma_s = \frac{d \sigma_s}{d z} = \frac{4}{d_b} \tau_s(z)
\]

\[
\sigma_r = \frac{d \sigma_r}{d z} = \frac{1}{t_f} \tau_r(z)
\]

where \(d\) is the steel rebar diameter; \(t_f\) is the thickness of the FRP sheet; \(\sigma_s(z)\) and \(\sigma_r(z)\) are tensile stresses in the steel rebar and the FRP sheet respectively; \(\tau_s(z)\) and \(\tau_r(z)\) are bond stresses between steel rebar and concrete and FRP sheet and concrete, respectively.

The equations 1-4 furnish a system of differential equations that cannot be solved in closed form because the inhomogeneity of the boundary conditions. A numerical procedure is used to solve the system; the boundary values being imposed at the midway section of the block (the slip is zero for symmetry) and at the cracked section in which the tensile concrete strain is zero at the level of the reinforcements (Aiello and Ombres, 2004). By using the constitutive relations of materials and an explicit form of the bond-slip law, the model allows to evaluate the crack width, the crack spacing (minimum and maximum values of the crack spacing) and the mean curvature of the block. Adopting this model it is possible to define completely the moment-curvature diagram that can be used for evaluating the beam deflections.

For beams strengthened with the FRCM system, in which can be neglected the slippage between FRP and concrete, the structural analysis was carried out by means of classical models founded on the hypothesis of perfect bond between the reinforcements (FRP and steel) and the concrete (Aiello et al., 2005).

**NUMERICAL INVESTIGATION AND COMPARISONS**

In order to evaluate the influence of the bond properties of the bonding system on the structural behaviour of beams, the above-mentioned nonlinear model has been used to carry out a numerical investigation. Reinforced concrete beams considered in the analysis have been tested from other researchers and experimental results are available in the literature. Theoretical predictions are obtained by using the bond-slip models analytically described in the Table 1. For each considered beam, predictions of models adopted from the ACI (ACI Committee, 2002) and the Eurocode (Eurocode 2, 1992) that suppose a perfect bond between the FRP and the
concrete, have been considered for comparisons. In addition, curves obtained by using a bilinear bond-slip model (BL model) in which the ascending branch is described by the slip modulus $E_b = G_b/t_b$, being $G_b = E_b / (1 + 2\nu b)$ the shear modulus, and $t_b$ the the thickness of the adhesive, are also reported in the figures. The first part of the investigation is devoted to a comparison, in terms of curvature, deflections and crack widths, of results obtained using the above mentioned bond-slip models and tests results. Analysed beams have been strengthened by CFRP sheets externally bonded to the concrete by means of polymeric resin as adhesive. For all beams debonding failure modes have been observed; as a consequence it is reasonable to suppose that the structural behaviour of beams has been strongly governed from slips between the FRP reinforcements and the concrete.

The Figure 3 shows the comparison between theoretical predictions and experimental values for the beam A2 tested from Ceroni et al. (Ceroni et al., 2000). The A2 beam was tested under four points loading, had a rectangular cross section 150 mm width, 100 mm high and was 1800 mm long. The beam was reinforced with two steel bars of 8 mm diameter both in tension and compression; yield strength and ultimate strength of steel were 590 MPa and 690 MPa, respectively. The beam failure was due to the peeling of the CFRP sheet.

Analysing obtained results it appears as the Code relationships, founded on the hypothesis of perfect bond between the FRP sheets and the concrete are unable to predict experimental crack widths values. Better predictions are obtained taking into account the slip between FRP and concrete. All bond-slip laws considered furnish similar prediction values; for the case examined, however, the bilinear model (BL model) and the Monti model are more accurate than other models. The comparison between predictions in terms of load-midspan curves evidences as the differences of values obtained using the various bond-slip models are very low.

Moment-curvature and load-deflection diagrams relative to the beam A3.1 tested from Spadea et al. (Spadea et al., 1998) are shown in the Figures 4.

The A3.1 beam with rectangular sections was 140 mm width, 300 mm high and 5000 mm long, reinforced with two 16 mm steel bars both in tension and compression. The mean compressive strength of the concrete was 30 MPa and the yield strength of the steel was 435 MPa. One layer of CFRP sheet was used for strengthening the beam before cracking. The adhesive used in bonding of CFRP sheet to the concrete surface was 2 mm thick; its mechanical properties were 12,80 GPa for the Young’s modulus and adhesive strength on concrete upper than 4 MPa. The failure of the beam was due to the CFRP sheet debonding.
The comparison in terms of curvature and midspan deflection for this beam shows as the Ueda bond-slip model furnishes predictions that are in very good agreement with experimental values; for load values until 0.5-0.6 \( P_u \) (\( P_u \) being the ultimate load value) the curve obtained using the Ueda model is perfectly coincident with experimental curve. Very good predictions are obtained also by using the bilinear bond-slip model (BL model); other considered bond-slip models furnish predictions similar to those of Code models.

In order to analyse the distribution of bond stress and slip along the concrete block between two consecutive cracks a numerical investigation has been carried out by using the bilinear model (BL model) as bond-slip law. The analysis refers to beams tested from various researchers: the distributions of bond stresses and slip have been evaluated supposing that the strengthening with the FRP sheets is made in presence of existing loads. In particular, with reference to the serviceability conditions, the analysis has been carried out considering four levels of applied loads: 0.2 \( P_u \), 0.3 \( P_u \), 0.4 \( P_u \), 0.5 \( P_u \) being \( P_u \) the ultimate load value experimentally determined. In the Figures 5-7 are shown the distributions of the bond stress in the concrete block between two consecutive cracks for three beams; the \( A2 \) beam tested from Ceroni et al. for which the failure was due to the peeling mode, the \( FC1 \) beam tested from Almusallam and Al-Salloum (Almusallam and Al-Salloum 1998) for which a flexural failure mode was observed and the \( C1 \) beam tested from Juvandes et al. (Juvandes et al. 2001) for which a shear failure mode was observed. Numerical results were obtained assuming \( \tau_{\text{max}} = 6.64 \text{ MPa} \), \( u_f = 0.2 \text{ mm} \) and \( E_b = 200 \text{ N/mm}^2 \).

Analyzing curves it appears as the maximum value of the bond stress is reached at a distance from the midway of the cracked block less than the development length \( l_b = l_{\text{min}} \) in all examined cases. However for the \( A2 \) beam for which a premature failure mode is observed (peeling of the FRP sheet) the \( \tau_{\text{max}} \) is reached near to the cracked sections while for \( FC1 \) and \( C1 \) beams in which failure modes are due to concrete collapse, \( \tau_{\text{min}} \) is reached in proximity of the midway section.

The influence of parameters characterizing the bilinear bond-slip law, \( \tau_{\text{max}} \) and \( u_f \), on the structural response of strengthened beams is evidenced in the Figures 9 and 10. The Figure 9 shows the load-deflection diagrams varying the \( \tau_{\text{max}} \) value while in the Figure 10 moment-curvature curves are drawn varying the value of the ultimate slip value; results are obtained with reference to the \( A3.1 \) beam tested from Spadea et al.. The analysis of results evidences as the variation of the maximum bond value influence the performances of strengthened beams.
beams; increasing $\tau_{\text{max}}$, being $E_s$ and $u_f$ fixed, the deflections of the beam are increasing. On the contrary, if $\tau_{\text{max}}$ and $E_s$ are fixed, the variation of the $u_f$ value is un-influential on the structural response of the beam.

The bond value is then the most important parameter governing the interface behaviour between the FRP sheets and the concrete. This result is confirmed from diagrams reported in the Figures 11 and 12 relative to the A2 beam. The Figure 11 shows diagrams applied load-maximum crack width varying $\tau_{\text{max}}$: it is evident as the variation of $\tau_{\text{max}}$ modifies the shape of the curves. This effect is more pronounced for high values of the applied loads: an increase of $\tau_{\text{max}}$ values produces a reduction of crack widths. A same result is obtained in terms of crack spacing: as shown in the Figure 12, increasing $\tau_{\text{max}}$ the crack spacing is reducing.

**CONCLUSIONS**

The role of the bond between the FRP materials and the concrete on the structural performances of FRP strengthened reinforced concrete beams, is investigated in the paper. The results of the structural analysis, allows to draw the following concluding remarks:

i) the bond behaviour of FRP reinforced concrete beams is depending on the adopted bonding technique:
   - a perfect bond between FRP materials and existing concrete is observed if the Fiber Reinforced Cementitious Mortar (FCRM) system is adopted while slips between FRP and concrete characterize the behaviour of beams if the adhesive system (polymeric resin) is adopted;

ii) a reliable structural analysis imposes the use of non linear models founded on the local bond-slip relationship;

iii) existing bond-slip models are able to predict with good accuracy the performances of strengthened beams, mainly in the serviceability conditions. Results presented in the paper highlight as the Ueda and the bilinear model (BL model) are more effective than other examined models;

iv) the distribution of the bond stress and slips along the strengthened beam under service conditions is associate to the failure mode. Referring to a concrete block between two consecutive cracks, the maximum value of the bond stress is reached near to the cracked section when premature failure modes

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**Figure 9. Load-midspan deflection diagrams varying $\tau_{\text{max}}$**

**Figure 10. Moment-curvature diagrams varying $u_f$**

**Figure 11. Load- crack width curves varying $\tau_{\text{max}}$**

**Figure 12. Load- crack spacing curves varying $\tau_{\text{max}}$**

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take place while it is attained in proximity of the midway if traditional failure modes, such as flexural and shear modes, take place.

v) the maximum bond value is the most important parameter governing the interface behaviour between the FRP sheets and the concrete and its influence on the structural response of strengthened beams is relevant; the influence of of the ultimate slip value, on the contrary, is neglecting.

Further investigations, both theoretical and experimental, are needed for a better understanding of the FRP-to-concrete bond behaviour; however, it is evident as the role of the local bond behaviour is conditioning for the structural performances of strengthened or retrofitted reinforced concrete members both at failure and under service conditions.

REFERENCES


American Concrete Institute- Committee 440 (2001). “Guide for the design and construction of externally bonded FRP systems for strengthening of concrete structures”, ACI, Farmington Hills, MI, USA.

Benmokrane B., Ghao D. and Tighiouart B: (1998).”Investigation on the bond of fiber reinforced polymer (FRP) to concrete"Proceedings of the 2nd International Conference on Composites in Infrastructures, University of Arizona, Tucson


