RILEM State-of-the-Art Reports

Carlo Pellegrino
José Sena-Cruz *Editors*

Design Procedures for the Use of Composites in Strengthening of Reinforced Concrete Structures

State-of-the-Art Report of the RILEM Technical Committee 234-DUC

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RILEM STATE-OF-THE-ART REPORTS
Volume 19

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Editors

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ISSN 2213-204X ISSN 2213-2031 (electronic)
RILEM State-of-the-Art Reports
ISBN 978-94-017-7335-5 ISBN 978-94-017-7336-2 (eBook)
DOI 10.1007/978-94-017-7336-2

Library of Congress Control Number: 2015945616

Springer Dordrecht Heidelberg New York London

Printed on acid-free paper

Springer Science+Business Media B.V. Dordrecht is part of Springer Science+Business Media (www.springer.com)

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Chapter 1
Introduction

Carlo Pellegrino and José Sena-Cruz

Abstract This book analyses the current knowledge on structural behaviour of RC elements and structures strengthened with composite materials (experimental, analytical and numerical approaches for EBR and NSM), and the comparison of the predictions of the current available codes/recommendations/guidelines with selected experimental results. The book shows possible critical issues (discrepancies, lacunae, relevant parameters, test procedures, etc.) related to current code predictions or to evaluate their reliability, in order to develop more uniform methods and basic rules for design and control of FRP-strengthened RC structures. General problems/critical issues are clarified on the basis of the actual experiences, detect discrepancies in existing codes, lacunae in knowledge and, concerning these identified subjects, provide proposals for improvements. The book will help to contribute in promoting and consolidating a more qualified and conscious approach towards rehabilitation and strengthening existing RC structures with composites and their possible monitoring.

Keywords FRP · FRCM · Composites · Strengthening · Reinforced concrete

Introduction

Strengthening and retrofitting of existing structures have been widely discussed topics for the last few decades. A great number of existing structures need rehabilitation or strengthening because of improper design or construction, change of the design loads, damage caused by environmental and/or human factors, seismic events, etc. Several different systems have been developed and used to strengthen

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C. Pellegrino and J. Sena-Cruz (eds.), *Design Procedures for the Use of Composites in Strengthening of Reinforced Concrete Structures*, RILEM State-of-the-Art Reports 19, DOI 10.1007/978-94-017-7336-2_1

existing structures. They include replacing structural members, adding new material to improve their performance, modifying the restraint conditions, introducing post-tension, etc. These techniques have been proven to be effective, but in some cases they can be expensive and difficult to apply. The use of fibre reinforced composites applied to existing structural elements may represent a cost-effective alternative to such traditional strengthening techniques. Among fibre reinforced composites, strengthening by means of fibre reinforced polymers (FRP) has gained great popularity because of its high mechanical properties and relatively low cost. FRP composites are comprised of high strength fibres (e.g. carbon, glass, aramid) applied to the element surface through thermosetting organic matrices, usually epoxy resin. FRP composites can be externally bonded (ER) to the element surface or placed within groove carved into the element and filled with organic matrices (Near Surface Mounted technique, NSM). The fibres are meant to carry the tensile forces, whereas the matrix transfers the stress to the concrete support. They are easy to install, have a high strength-to-weight ratio, and have suitable mechanical properties. The structural behaviour of FRP composites applied to reinforced concrete (RC) elements and structures has been widely studied over the last decades and these studies have resulted in some design guidelines. American ACI 440.2R-08 (2008), European fib T.G. 9.3 (2001), and Italian CNR-DT 200 R1 (2013) are examples of such guidelines. Although a large number of experimental, analytical, and numerical studies regarding FRP composites are available in the literature, the predictions provided by guidelines and analytical models are sometimes contrasting and disagreeing with the experimental results. For this reason, the scientific community is still discussing about some important issues and new improved guidelines are under preparation all around the world. It has been observed that the use of organic resins, though effective, may represent an issue for the durability of the intervention. Indeed, organic matrices degrade when exposed to UV radiation and loose most of their mechanical properties when subjected to temperatures close (or higher) to their glass transition temperature. Promising newly-developed types of matrix that potentially represent a valid alternative to organic resins are the so-called inorganic matrices. Within the broad category of inorganic matrices, polymer-modified cement-based mortars have raised some interest in recent years. Composite materials that employ modified cement-based mortars are usually referred to as fiber-reinforced cementitious matrix (FRCM) composites. Although several works about FRP strengthening are available in the literature very few studies can be found regarding FRCM composites.

The international Rilem Technical Committee 234-DUC was created in 2009 with the aim of facing the issues connected with the use of fibre reinforced composites to strengthen RC elements and structures. This committee is composed by a team of experts representing most of the main international institutions working on the subject. Members become from international academic and research institutions, other Rilem technical committees working on reinforced concrete and composites, standardization groups, and national and international groups who have contributed to the development of the current codes/recommendations/guidelines. The committee includes members from Italy, Cyprus, United States, England, Portugal, China, Germany, Switzerland, Sweden, Spain, and other countries. Through the fruitful collaboration between members, promoted by the annual meetings, the committee could analyse many different aspects of the FRP strengthening technique and give insights for improvement and development of the existing analytical models. Furthermore, newly-developed promising fibre reinforced inorganic composites (FRCM) were analysed as well.

This book collect the results of 4 years of work by the Technical Committee 234-DUC. Chapter 2 provides the outcomes of a statistical analysis based on the indication of EN1990 and extended to the case of EBR FRP system. This chapter shows a procedure to evaluate the statistical parameters of the capacity models and to evaluate its characteristic values, which is the aim for application in design. Furthermore, some applications are reported to prove the feasibility of the proposed procedure. Chapter 3 provides a deep analysis of the bond behavior of FRP composites externally applied onto RC structures. The bond is described through a fracture mechanics approach and the theoretical models are compared with the experimental results available in the literature. An assessment of the most important analytical models for the estimation of the bond strength of FRP-concrete joints is provided as well. Finally, the chapter faces the critical issues of models and experimental procedures employed to investigate the FRP bond behavior. In Chap. 4 shear strengthening by means of EBR FRP is analyzed. The main analytical formulation for the evaluation of the shear strength of shear strengthened RC elements are recalled and new improved formulations are provided. The models analyzed are assessed through a wide experimental database to evaluate their accuracy with respect to the experimental evidences. Chapter 5 describes the use of FRP jackets for confining RC members mainly subjected to axial loading. The current formulations for the evaluation of the effective ultimate strain in the FRP are provided and discussed. The influence of the internal steel on the EBR FRP jackets and particularly the effects of possible bar buckling are discussed as well. Finally, experimental results available in the literature are compared with analytical provision to assess the accuracy of the proposed models. Chapter 6 gives an overview on the state-of-the-art about verifications of reinforced concrete structures using Externally Bonded (EB) Fibre Reinforced Polymers (FRP) under serviceability loading conditions (long term behaviour, durability under severe conditions), fatigue load, and fire and high temperature. Furthermore, the use of anchoring systems and mechanically fastened system is described. Chapter 7 describes the use of prestressed EBR FRP system. The chapter provides information regarding commercially available prestressing systems and their anchorage procedures. The newly-developed technique of "gradient anchorage" and various current prototypes at the laboratory-scale level are shown as well. Chapter 8 gives an overview on the state-of-the-art of the NSM technique for structural retrofitting of reinforced concrete structures using FRP composites. The chapter firstly describes the technique and addresses the existing knowledge on the bond behaviour. Furthermore, two formulations for predicting the NSM shear carrying capacity are provided. Finally, the needs for future research on this topic are identified. Chapter 9 describes the use of FRCM composites for strengthening existing RC and masonry structures. After introducing the commercially available composites, the chapter gives a state-of-the-art about the test methods under development for characterizing this materials and provides a fracture mechanics approach that allows for describing FRCM-concrete joints bond behaviour. Finally, the effectiveness of the FRCM technique for flexural strengthening and confinement of RC elements is shown and some experimental campaigns are described.

Chapter 2
Design by Testing and Statistical Determination of Capacity Models

Giorgio Monti, Antonio Bilotta, Annalisa Napoli, Emidio Nigro, Floriana Petrone and Roberto Realfonzo

Abstract In this chapter, the procedure proposed in EN1990 is adopted and extended to the case of EBR FRP systems, with the aim of attaining a uniform reliability level among all equations developed in this technical report. This approach will allow comparing experimental results and theoretical predictions in a consistent manner, and also identifying possible sources of error in the formulations. Any capacity model should be developed on the basis of theoretical considerations and subsequently fine-tuned through a regression analysis based on tests results. The validity of the model should then be checked by means of a statistical interpretation of all available test data. The formulation should include in the theoretical model a new variable that represents the model error. This variable is assumed to be normally distributed whit unit mean and standard deviation to be evaluated from comparison with experimental results. Once the statistical parameters of the model error are known, it is possible to define the statistical parameters of the capacity model and to evaluate its characteristic value, which is the aim for application in design. Some applications are shown to prove the feasibility of the proposed procedure.

Introduction

When developing a design equation, the predictive ability of the analytical capacity model, regardless of how it has been obtained, whether through a mechanics-based approach or through a regression from test data, must be validated over a reasonably large(r) set of experimental data. Thus, the definition of a reliable capacity

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C. Pellegrino and J. Sena-Cruz (eds.), *Design Procedures for the Use of Composites in Strengthening of Reinforced Concrete Structures*, RILEM State-of-the-Art Reports 19, DOI 10.1007/978-94-017-7336-2_2

model to be used in practical design applications requires to follow a rigorous procedure that eventually will aim at calibrating the safety factor to apply to the equation so that it meets an assigned reliability target. This procedure requires the model to be formulated in a probabilistic way, so that both inherent and epistemic uncertainties of the underlying basic variables (geometry and materials, essentially) can be dealt with, as well as the uncertainties associated to the capacity equation itself. All of these uncertainties can be easily incorporated into a model, through the adoption of a random variable that represents the difference between actual and predicted response.

Probabilistic Capacity Models: Analytical Definition

As widely illustrated by Monti et al. (2009) (and also by Monti and Petrone 2014 who extended the procedure to the case of additive uncertainties), a capacity model should be defined as:

$$C\{f;a\} = R\{f;a\} \cdot \delta\{X_i\} \tag{2.1}$$

where: R is a function that "explains" the resisting mechanism capacity, given certain mechanical properties f and geometrical properties a, and represents the "deterministic" part of the capacity model; $\delta\{X_i\}$ is a random variable containing information about the overall model error prediction with respect to experimental results, and represents the "random" part of the capacity model. It is characterized by mean $\mu_\delta = b$ and variance σ_δ^2 and is a function of all the X_i parameters characterized by uncertainties. In the following it will be assumed: $\{X\}_i = \{f;a;m\}$, where f, a and m are parameters related to materials, geometry and model, respectively.

In common practice, only the deterministic part of the capacity model is considered, usually given as:

$$C_{\text{det}}\{f;a\} = b \cdot R\{f;a\} \tag{2.2}$$

Many interpret b as a model fine-tuning coefficient (sometimes called "ignorance" coefficient), so that, when the "deterministic" (mean) part of the capacity model is used, see Eq. (2.3), it predicts the experimental capacity with zero error "in the average". The coefficient b is computed through a least-square approach, by minimizing "in the average" the difference between predicted and experimental values. Pictorially speaking, the coefficient b brings the "cloud" of "theoretical-experimental" points closer to the bisectrix of the Cartesian plan.

Therefore, once the functional form $R\{f;a\}$ is found, in order to completely describe the random variable δ, we perform n experimental tests T, by "appropriately" changing the values of f and a. So, by selecting n sets of values $\{f_{\exp,i};a_{\exp,i}\}$, we obtain n experimental values of the capacity, expressed as:

$$C = T\left\{f_{\exp,i}; a_{\exp,i}\right\} \cdot \tau \quad (i = 1 \ldots n) \tag{2.3}$$

where τ represents a random error, with unit mean and assigned variance, the latter being due to test setup imprecisions, load application modality, and measurement errors that may affect the results. Usually, this term is disregarded (in the sense that the measured values $C_{\exp,i}$ already include it).

The corresponding values predicted by the "deterministic" model are:

$$C_{\det,i}\left\{f_{\exp,i}; a_{\exp,i}\right\} = b \cdot R\left\{f_{\exp,i}; a_{\exp,i}\right\} \tag{2.4}$$

By comparison of n experimental and theoretical values, b is found from the well-known minimization:

$$\min \sum_{i=1}^{n} \left(\frac{C_{\exp,i}}{C_{\det,i}} - 1\right)^2 \rightarrow b \tag{2.5}$$

Equation (2.7) can be analytically developed to finally obtain:

$$b = \frac{\sum_{i=1}^{n} \left(\frac{C_{\exp,i}}{R\left\{f_{\exp,i}; a_{\exp,i}\right\}}\right)^2}{\sum_{i=1}^{n} \frac{C_{\exp,i}}{R\left\{f_{\exp,i}; a_{\exp,i}\right\}}} \tag{2.6}$$

The variance of δ is obtained as well, as:

$$\sigma_\delta^2 = Var\left[\frac{C_{\exp}}{C_{\det}}\right] \tag{2.7}$$

with its unbiased estimate being (after b has been fine-tuned):

$$s_\delta^2 = \frac{\sum_{i=1}^{n} \left(\frac{C_{\exp,i}}{b \cdot R\left\{f_{\exp,i}; a_{\exp,i}\right\}} - 1\right)^2}{n - 2} \tag{2.8}$$

By replacing Eq. (2.2) into Eq. (2.1), the following relation holds:

$$C\{f; a\} = C_{\det}\{f; a\} \frac{\delta\{X_i\}}{b} \tag{2.9}$$

Letting:

$$\bar{\delta}\{X_i\} = \frac{\delta\{X_i\}}{b} \tag{2.10}$$

$\overline{\delta}\{X_i\}$ represents the model error to be applied to the fine-tuned deterministic capacity model. It is characterized by mean $\mu_\delta = 1$ and variance:

$$\sigma_{\overline{\delta}}^2 = \frac{1}{b^2} Var\left[\frac{C_{\exp}}{C_{\det}}\right] \tag{2.11}$$

with its estimate being (after b has been fine-tuned):

$$s_{\overline{\delta}}^2 = \frac{\sum_{i=1}^{n}\left(\frac{C_{\exp,i}\left\{f_{\exp,i};a_{\exp,i}\right\}}{b\cdot R\left\{f_{\exp,i};a_{\exp,i}\right\}} - 1\right)^2}{b^2(n-2)} \tag{2.12}$$

Calibration of Partial Safety Factors Based on Testing

Once the random part has been "adjusted" by means of experiments, a common, though wrong, further step is to use only the deterministic part to predict the capacity, with the assumption that its characteristic and design values can be obtained by plugging in the argument, respectively, characteristic and design values. Worse usual mistakes regard an arbitrary reduction of b, to obtain a "safer" estimate. In the following, a rigorous and effective procedure is instead developed, where the "modeling" part is clearly distinguished from the "safety" part.

The probabilistic capacity model so developed has first-order-approximation mean and variance given by, respectively:

$$\mu_C = b \cdot R\left\{\mu_f; \mu_a\right\} \cdot \mu_{\overline{\delta}} = b \cdot R\left\{\mu_f; \mu_a\right\} \tag{2.13}$$

$$\begin{aligned} \sigma_C^2 &= C_{,f}^2\left\{\mu_f; \mu_a\right\} \cdot \sigma_f^2 + C_{,a}^2\left\{\mu_f; \mu_a\right\} \cdot \sigma_a^2 + C_{,\overline{\delta}}^2\left\{\mu_f; \mu_a\right\} \cdot \sigma_{\overline{\delta}}^2 \\ &= b^2 \cdot R_{,f}^2\left\{\mu_f; \mu_a\right\} \cdot \sigma_f^2 + b^2 \cdot R_{,a}^2\left\{\mu_f; \mu_a\right\} \cdot \sigma_a^2 + b^2 \cdot R^2\left\{\mu_f; \mu_a\right\} \cdot \sigma_{\overline{\delta}}^2 \end{aligned} \tag{2.14}$$

where $C_{,f} = \partial C/\partial f$, $C_{,a} = \partial C/\partial a$ and $C_{,\overline{\delta}} = \partial C/\partial\overline{\delta}$ are the partial derivatives of the function C with respect to f, a, and $\overline{\delta}$, respectively, and $R_{,f} = \partial R/\partial f$, $R_{,a} = \partial R/\partial a$ and $R_{,\overline{\delta}} = \partial R/\partial\overline{\delta}$ are the partial derivatives of the function R with respect to f, a, and $\overline{\delta}$, respectively. Also, note that all variables have been assumed as statistically independent, so that all covariance are zero.

In Eq. (2.14), the first term of the second member represents the intrinsic (material) uncertainty, the second term represents the parametric (geometry) variability, and the third term represents the epistemic (model) uncertainty.

Case of limited number of tests

When predicting a limited number of tests n, we can only obtain estimates of mean and variance of the capacity model, as follows:

$$C\{\bar{f};\bar{a}\} = b \cdot R\{\bar{f};\bar{a}\} \tag{2.15}$$

$$\begin{aligned} & s_C^2\{\bar{f};\bar{a}\} \\ & = b^2 \cdot R_{,f}^2\{\bar{f};\bar{a}\} \cdot s_f^2 + b^2 \cdot R_{,a}^2\{\bar{f};\bar{a}\} \cdot s_a^2 + b^2 \cdot R^2\{\bar{f};\bar{a}\} \cdot s_{\bar{\delta}}^2 \end{aligned} \tag{2.16}$$

where:

$$\bar{f} = \frac{\sum_{i=1}^{n} f_i}{n} \quad ; \quad \bar{a} = \frac{\sum_{i=1}^{n} a_i}{n} \tag{2.17}$$

$$s_f^2 = \frac{\sum_{i=1}^{n}\left(f_i - \bar{f}\right)^2}{n-1} \quad ; \quad s_a^2 = \frac{\sum_{i=1}^{n}(a_i - \bar{a})^2}{n-1} \tag{2.18}$$

and where $R_{,f} = \partial R/\partial f$, $R_{,a} = \partial R/\partial a$ and $R_{,\bar{\delta}} = \partial R/\partial\bar{\delta}$ are the partial derivatives of the function R with respect to f, a, and $\bar{\delta}$, respectively.

Under the hypothesis that C is normally distributed, when we have a limited number of tests, the characteristic value of the capacity model has a non-central *t*-Student distribution with n–1 degrees of freedom and with non-centrality parameter equal to $u_\alpha\sqrt{n}$ (notice that, when looking for the characteristic values in a normal distribution, which is the 5 % fractile, we have: $u_\alpha = 1.645$). An unbiased estimate (mean) of the characteristic value is (see e.g., Madsen et al. 1986):

$$C_k\{\bar{f};\bar{a}\} = C\{\bar{f};\bar{a}\} - k_{\alpha,n} \cdot s_C\{\bar{f};\bar{a}\} \tag{2.19}$$

where $k_{\alpha,n}$ is:

$$k_{\alpha,n} = u_\alpha \varepsilon_n = u_\alpha \sqrt{\frac{n-1}{2}} \frac{\Gamma\left(\frac{n-1}{2}\right)}{\Gamma\left(\frac{n}{2}\right)} \tag{2.20}$$

with Γ the Gamma function. An excellent approximation to the above equation is here proposed as:

$$k_{\alpha,n} = u_\alpha \varepsilon_n = u_\alpha \frac{4n-5}{4n-6} = u_\alpha \frac{n-1.25}{n-1.50} \tag{2.21}$$

For the purpose of the following developments, it is expedient to rewrite Eq. (2.19) as:

$$C_k\{\bar{f};\bar{a}\} = C\{\bar{f};\bar{a}\} - u_\alpha \varepsilon_n \cdot s_C\{\bar{f};\bar{a}\} \tag{2.22}$$

where ε_n becomes a sort of "scaling" coefficient of the capacity axis, easily determined for practical purposes as function of the number n of tests performed as:

$$\varepsilon_n = \frac{n - 1.25}{n - 1.50} \tag{2.23}$$

Having determined the characteristic value of the capacity model, the next step is the determination of its design value. This is given as:

$$C_d\{\bar{f};\bar{a}\} = \frac{C_k\{\bar{f};\bar{a}\}}{\gamma_C} = \frac{C\{\bar{f};\bar{a}\} - u_\alpha \varepsilon_n \cdot s_C\{\bar{f};\bar{a}\}}{\gamma_C} \tag{2.24}$$

where the safety factor γ_C is to be calibrated by considering that the design value is found as:

$$C_d\{\bar{f};\bar{a}\} = C\{\bar{f};\bar{a}\} - \beta_{LS}\alpha_C\varepsilon_n \cdot s_C\{\bar{f};\bar{a}\} \tag{2.25}$$

where it should be noted that the axis has been scaled by means of the same coefficient ε_n to account for the limited amount of tests; in the above equation, β_{LS} is the safety index associated to the acceptable exceeding probability of the considered Limit State in a given time period, and α_C is the FORM sensitivity coefficient associated to capacity variables.

Thus, the safety factor is found as:

$$\gamma_C = \frac{C_k\{\bar{f};\bar{a}\}}{C_d\{\bar{f};\bar{a}\}} = \frac{C\{\bar{f};\bar{a}\} - u_\alpha \varepsilon_n \cdot s_C\{\bar{f};\bar{a}\}}{C\{\bar{f};\bar{a}\} - \beta_{LS}\alpha_C\varepsilon_n \cdot s_C\{\bar{f};\bar{a}\}} \tag{2.26}$$

This is the factor that, once applied to the characteristic value $C_k\{\bar{f};\bar{a}\}$, gives the design value of the capacity model, $C_d\{\bar{f};\bar{a}\}$. Notice that there is an explicit dependence of the safety factor on the number of tests performed.

The above procedure needs now to be applied to the format adopted in Eurocode 0 (EN 1990) for all capacity equations, which reads as follows:

$$C_{d,EC0}\{f_d;\bar{a}\} = \frac{1}{\gamma_{Rd}} b \cdot R\{f_d;\bar{a}\} = \frac{1}{\gamma_{Rd}} b \cdot R\left\{\frac{f_k}{\gamma_m};\bar{a}\right\} \tag{2.27}$$

According to the Eurocode philosophy, safety factors are divided into “internal” ones, such as the different γ_m’s, applied to material properties, and an “external” one, such as γ_{Rd}, applied to the capacity model. The former are meant to cover the intrinsic uncertainties in the material properties, while the latter deals with epistemic uncertainties related to the model. This format allows calibrating γ_{Rd} separately from the γ_m’s, which may as well be taken as those already given in the code.

Thus, we should impose:

$$C_{d,EC0}\{f_d;\bar{a}\} = C_d\{\bar{f};\bar{a}\} \tag{2.28}$$

$$\frac{1}{\gamma_{Rd}} b \cdot R\left\{\frac{f_k}{\gamma_m}; \bar{a}\right\} = C\{\bar{f}; \bar{a}\} - \beta_{LS}\alpha_C \varepsilon_n \cdot s_C\{\bar{f}; \bar{a}\} \tag{2.29}$$

$$\begin{aligned} &\frac{1}{\gamma_{Rd}} R\left\{\frac{f_k}{\gamma_m}; \bar{a}\right\} \\ &= R\{\bar{f}; \bar{a}\} - \beta_{LS}\alpha_C \varepsilon_n \sqrt{R_{,f}^2\{\bar{f}; \bar{a}\} \cdot s_f^2 + R_{,a}^2\{\bar{f}; \bar{a}\} \cdot s_a^2 + R^2\{\bar{f}; \bar{a}\} \cdot s_{\bar{\delta}}^2} \end{aligned} \tag{2.30}$$

Finally, the sought general expression for the "external" safety factor accounting for the number of tests performed is found as:

$$\gamma_{Rd} = \frac{R\left\{\frac{f_k}{\gamma_m}; \bar{a}\right\}}{R\{\bar{f}; \bar{a}\} - \beta_{LS}\alpha_C \varepsilon_n \sqrt{R_{,f}^2\{\bar{f}; \bar{a}\} \cdot s_f^2 + R_{,a}^2\{\bar{f}; \bar{a}\} \cdot s_a^2 + R^2\{\bar{f}; \bar{a}\} \cdot s_{\bar{\delta}}^2}} \tag{2.31}$$

Notice that the safety factor depends on the number of tests performed through: ε_n, $\bar{f}$, $\bar{a}$, s_f^2, s_a^2, and $s_{\bar{\delta}}^2$, where the latter also contains the "ignorance" coefficient b, which represents a measure of the prediction capability of the capacity equation.

The above equation can be used to calibrate the "external" safety factor of any capacity equation, with a known functional form $R\{\cdot\}$, and with the "internal" safety factors already provided by the relevant code.

Also notice that, when dealing with quality-controlled materials, such as steel, one may replace the sample estimates $\bar{f}$ and s_f^2 with the corresponding population parameters μ_f and σ_f^2, so that the characteristic value of the material property may also be found as: $f_k = \mu_f - 1.645\sigma_f$. In this case, by knowing the coefficient of variation V_f, mean and variance can be easily found from the characteristic value as:

$$\mu_f = \frac{f_k}{1 - 1.645V_f} \tag{2.32}$$

$$\sigma_f^2 = \left(\frac{V_f f_k}{1 - 1.645V_f}\right)^2 \tag{2.33}$$

The safety factor then becomes:

$$\gamma_{Rd} = \frac{R\left\{\frac{f_k}{\gamma_m}; \bar{a}\right\}}{R\{\mu_f; \bar{a}\} - \beta_{LS}\alpha_C \varepsilon_n \sqrt{R_{,f}^2\{\mu_f; \bar{a}\} \cdot \sigma_f^2 + R_{,a}^2\{\mu_f; \bar{a}\} \cdot s_a^2 + R^2\{\mu_f; \bar{a}\} \cdot s_{\bar{\delta}}^2}} \tag{2.34}$$

Therefore, the design capacity is:

$$R_d\left\{\frac{f_k}{\gamma_m};\bar{a}\right\} = \frac{1}{\gamma_{Rd}} b \cdot R\left\{\frac{f_k}{\gamma_m};\bar{a}\right\} \quad (2.35)$$

Application 1—End Debonding

In this section, an application of the design by testing procedure is shown for the assessment of a design formulation to predict the end debonding load in Reinforced Concrete (RC) members strengthened with FRP Externally Bonded Reinforcement (EBR). Indeed, the high performances of FRP materials often cannot be properly exploited, since a typical failure is the debonding of the external reinforcement, namely the loss of bond at the concrete/FRP interface. This makes the bond strength at the interface a key issue in the strengthening design procedure. Usually debonding occurs within a thin layer of concrete and is related to its very low strength.

Several theoretical formulations have been proposed by researchers and international codes to predict the maximum stress in the FRP reinforcement when the end (Chen and Teng 2001; fib bulletin 2001; CNR-DT 200 2004; Smith and Teng 2002) or the intermediate debonding (Teng et al. 2003) occurs. Most of these formulations, characterized by similar structures, are calibrated by numerical factors based on experimental results.

Even though the assessment of models for bond strength has been widely dealt with by various researchers, the definition of safety factors to calculate design values is still an open item. Thus, detailed statistical analyses have been performed using a wide experimental database of bond tests in order to calibrate a bond strength model based on the fracture energy approach. The final proposed strength model is similar to other well-known models suggested in the literature and codes, but it is based on a detailed and consistent statistical analysis according to the 'design by testing' procedure suggested in the Eurocode 0 (Monti et al. 2009; Bilotta et al. 2011a; Monti and Petrone 2014). Different corrective factors allow both mean and characteristic values of debonding load to be predicted in order to follow a limit state design approach and associate a structural safety to the chosen model.

The approach to calculate the bond strength based on the fracture energy at the FRP-to-concrete interface has been summarized.

In order to develop statistical analyses, the experimental debonding loads of several bond tests have been collected and compared with three well-known relationships providing the end-debonding load in order to assess their reliability. Then, the same data have been used to assess a new relationship for the end-debonding-load according to the 'design by testing' procedure. In particular, numerical factors for both mean and percentiles provisions have been calibrated in order to furnish design provisions.

Moreover, the preformed and cured in situ EBR FRP systems have been distinguished to better exploit the performance of the latter ones.

Theoretical Formulations of Debonding Load

The maximum tensile force, $F_{\max}$, at debonding in an FRP external reinforcement characterized by an infinite bonded length can be calculated as:

$$F_{\max} = b_f \int_0^{\infty} \tau_b(x)dx \tag{2.36}$$

being $\tau_b(x)$ the bond shear stress distribution along the concrete-FRP interface and b_f the width of the FRP reinforcement.

Moreover, the fracture energy corresponding to a generic bond shear stress-slip law, $\tau_b(s)$, can be expressed as:

$$\Gamma_F = \int_0^{\infty} \tau_b(s)ds\,[\mathrm{F/L}] \tag{2.37}$$

This expression has the meaning of energy [FL] for unit surface [L^2].

Moreover, under the hypothesis that the concrete member has a stiffness much larger than the reinforcement, at the section in which the maximum stress, $\sigma_{f,max}$, is applied, the following relationship can be written:

$$\int_{A_f} \frac{1}{2} \cdot \sigma_f \cdot \varepsilon_f dA = b_f \cdot \int_0^{\infty} \tau_b(s)ds \tag{2.38}$$

This expression assumes the equality of the energy [F L] for unit length [L] associated to the tensile stress at the FRP section (area $A_f = b_f\, t_f$) with the fracture energy [F L] for unit length [L] developed at the FRP-concrete interface. Furthermore, assuming constant stresses along the FRP reinforcement section and a linear-elastic stress-strain relationship, Eq. (2.38) can be written as follows:

$$\frac{\sigma_f^2}{2 \cdot E_f} \cdot t_f \cdot b_f = b_f \cdot \Gamma_F \quad \frac{\left(\sigma_f \cdot t_f \cdot b_f\right)^2}{2 \cdot E_f} \cdot = b_f^2 \cdot t_f \cdot \Gamma_F \tag{2.39}$$

that gives the expression:

$$F_{\max} = b_f \cdot \sqrt{2 \cdot E_f \cdot t_f \cdot \Gamma_F} \tag{2.40}$$

where t_f, b_f, E_f are the thickness, the width, and the Young's modulus of the FRP reinforcement.

The fracture energy, Γ_F, depends on both the strength properties of adherents, concrete, and adhesive, and the characteristics of the concrete surface. If the FRP reinforcement is correctly applied, the debonding occurs in the concrete and the specific fracture energy of the interface law can be written in a form similar to that used for the shear fracture (Mode I). Therefore, the fracture energy can be expressed as a function of the concrete shear strength: $\Gamma_F(\tau_{b,max})$, where $\tau_{b,max}$ depends on both tensile and compressive concrete strength.

In most formulations, the fracture energy depends directly on the concrete tensile strength and on a shape factor that is function of the FRP-to-concrete width ratio (b_f / b_c). The formulations proposed by Neubauer and Rostasy (1997) and Lu et al. (2005), e.g., are:

$$G_f = 0.204 \cdot k_b^2 \cdot f_{ctm} \tag{2.41}$$

$$G_f = 0.308 \cdot \beta_w^2 \cdot \sqrt{f_{ctm}} \tag{2.42}$$

being f_{ctm} the mean tensile strength of concrete, k_b and β_w the shape factors defined as:

$$k_b = 1.06\sqrt{\frac{2 - b_f/b_c}{1 + b_f/400}}; \quad \beta_w = \sqrt{\frac{2 - b_f/b_c}{1 + b_f/b_c}} \tag{2.43}$$

Based on formulations of fracture energy similar to Eqs. (2.41) and (2.42) and on experimental results of bond tests, several theoretical formulations to evaluate the bond strength have been proposed in the past (Taljsten 1994; Neubauer and Rostasy 1997; Brosens and Van Gemert 1997; fib bulletin 2001; Chen and Teng 2001; Smith and Teng 2002; CNR-DT 200 2004). These expressions allow for predicting the end debonding load. In some cases, the same expressions are suitably modified by changing some factors in order to predict the intermediate crack debonding load in RC beams (Teng et al. 2003; Chen et al. 2006; CNR-DT 200 2004). The lay-out of these formulations is often similar, while the numerical coefficients calibrated on experimental results are different. Moreover, the safety factors, which are needed in order to calculate design provisions as part of the Limit State approach, are not always considered. This last point is an important issue, if a safety level (mean, characteristic or design) has to be associated to the provision.

The theoretical approaches suggested by fib bulletin 14 (2001), Chen and Teng (2001), CNR-DT 200 (2004) are considered. In particular, the bond strength expressed in terms of maximum tensile load in the FRP reinforcement, $N_{f,max}$, and the effective length, L_e, which is the minimum length required to full transfer the load, are defined as follows by the three approaches:

(1) fib bulletin 14 (2001):

$$N_{f,\max} = \alpha \cdot c_1 \cdot k_c \cdot k_b \cdot b_f \cdot \beta_L \cdot \sqrt{E_f \cdot t_f \cdot f_{ctm}}; \quad L_e = \sqrt{\frac{E_f \cdot t_f}{2 f_{ctm}}}$$
$$\beta_L = \frac{L_b}{L_e} \cdot \left(2 - \frac{L_b}{L_e}\right) \; if \; L_b \leq L_e, \quad \beta_L = 1 \; otherwise; \; \frac{b_f}{b} \geq 0.33 \tag{2.44}$$

where b_f, t_f, E_f, L_b are width, thickness, Young's modulus and bonded length of the FRP reinforcement, b_c is the width of the concrete element, f_{ctm} is the mean tensile strength of concrete, $c_1 = 0.64$ and $c_2 = 2$ are coefficients related to an experimental calibration of the fracture energy (Neubauer and Rostasy 1997), α = 0.9 is a reduction factor to account for the influence of inclined cracks on the bond strength, and k_c takes into account the state of compaction of concrete and usually is assumed equal to 1.00, or 0.67 for FRP bonded to concrete faces with low compaction. Finally, the shape factor k_b is given by Eq. (2.43).

(2) Chen and Teng (2001):

$$N_{f,\max} = \alpha \cdot \beta_w \cdot \beta_L \cdot b_f \cdot L_e \cdot \sqrt{f_c'}; \quad L_e = \sqrt{\frac{E_f \cdot t_f}{\sqrt{f_c'}}}$$
$$\beta_L = \sin\frac{\pi L_b}{2L_e} \; if \; L_b \leq L_e, \quad \beta_L = 1 \; otherwise \tag{2.45}$$

f'_c being the mean cylindrical compressive strength of concrete and α a coefficient equal to 0.427 or 0.315 to calculate a mean or a design provision, respectively. The shape factor β_w is given by Eq. (2.43). Note that the debonding strain values of Eq. (2.45) should be divided by an appropriate safety factor $\gamma_b = 1.25$ for design purpose, according to suggestion in Sect. 3.4 of Teng et al. (2001).

(3) CNR-DT 200 (2004):

$$N_{f,\max} = \frac{1}{\gamma_{f,d}\sqrt{\gamma_c}} \cdot \beta_L \cdot b_f \cdot \sqrt{k_G \cdot k_b} \cdot \sqrt{2 \cdot E_f \cdot t_f \cdot \sqrt{f_{ck} \cdot f_{ctm}}};$$
$$L_e = \sqrt{\frac{E_f \cdot t_f}{2 \cdot f_{ctm}}}$$
$$k_b = \sqrt{\frac{2 - b_f/b_c}{1 + b_f/400}} \geq 1 \; \beta_L = \frac{L_b}{L_e} \cdot \left(2 - \frac{L_b}{L_e}\right) \; if \; L_b \leq L_e, \; \beta_L = 1 \; otherwise \tag{2.46}$$

where f_{ck} is the characteristic value of the cylindrical compressive strength of concrete and k_G is an experimentally calibrated coefficient, which is 0.064 or 0.03 for mean or design provision, respectively. The shape factor k_b is the same given by Eq. (2.43), except for the coefficient 1.06.