Statistical Analysis of Existing Models for Flexural Strengthening of Concrete Bridge Beams using FRP Sheets

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Abstract

This paper performs a statistical analysis of previously proposed models for resisting the debonding of FRP sheets used in strengthening reinforced and prestressed concrete beams. End debonding and intermediate crack-induced debonding modes of failure are studied for beams in flexure. Two different databases are assembled from published experimental debonding tests on concrete beams of different span lengths. The first database contains the results of four point bending tests performed to study the behavior of the FRP-concrete bond at the end of the FRP sheet. The second database which includes four point bending tests, three point bending tests and one point loading tests, has been created to examine intermediate crack-induced debonding. These two databases are significantly larger than those used in developing any of the existing debonding strength models and provide a solid basis for assessing the performance of such models. A regression analysis reviews the relationship between the experimentally measured loads that caused debonding to the model predicted values as well as the bias and the variability in the prediction models. This regression analysis allows for drawing conclusions on the most appropriate and accurate models, from a statistical point of view, that may be used in a follow up reliability-based calibration of partial safety factors. The applicability of such information for the development of design specifications for strengthening of deteriorated concrete bridges is highlighted. This will be implemented in a forthcoming companion paper.

Keywords: fiber-reinforced plastic, flexural strengthening, model uncertainty

1. Introduction

Bonding of Fiber-Reinforced Polymer (FRP) to the tension face of concrete beams has become a frequent strengthening method over the last decade. The application of FRP to the tension face of a concrete beam or slab has multiple benefits, including: increased ultimate flexural strength capacity, increased post-cracking stiffness, as well as concrete crack control whereby the bonding of FRP sheets to concrete beams results in finer and more evenly distributed cracks when compared to the cracks that develop in unstrengthened beams. Because of these benefits, it is clear that flexural strengthening of reinforced concrete beams with FRP has a great potential for becoming a primary strengthening scheme for deteriorated infrastructure systems. However, because of the relative novelty of this technology, a more complete understanding of the behavior of FRP-strengthened beams and their failure mechanisms need to be gained before the wide spread adoption of this technology in engineering practice. As a minimum, the confidence levels in the safety margins of existing and proposed design criteria should be ascertained. This would require an objective estimation of the biases implied in existing and recently proposed design criteria and an analysis of the variability of the expected loads that cause failure from the predicted loads.

The fibers used in common FRP strengthening schemes are usually made of glass, carbon or aramid. To bond the fibers together and to the substrate surface on which they are being applied, a resin matrix is used. Two methods of application are frequently used. In situ installations require placing the fibers in an open mold, attaching the assembly to the surface and then saturating the assembly with resin. More commonly, the fibers may be first saturated with epoxy and then bonded to the prepared surface of concrete. In situ assemblies of the fibers are usually referred to as wet lay-up. Preimpregnated plates or sheets of fibers (prepreg) can also be installed by applying resin to bond the sheets to the concrete surface.

Hollaway and Teng [1] recognized six main failure mechanisms for reinforced concrete beams flexurally strengthened with FRP. These are represented in Figure 1 as: a) FRP rupture, b) crushing of concrete, c) intermediate crack-induced interfacial debonding, d) concrete cover separation, e) plate end interfacial debonding, f) concrete shear failure and g) critical diagonal cracking. Modes a), b) and f) are classical concrete beam failure mechanisms and have been analyzed to a great extent in the past [2,3,4]. The remaining failure mechanisms are specific to FRP strengthened beams and are characterized as premature failure modes because they take place before realizing the full potential of the strengthening scheme. Modes d), e) and g) are often indistinguishable and in fact, Smith and Teng [5,6] identified a mixed failure mode where the failure occurs at the same time by the concrete cover separation and the plate end interfacial debonding mode. In this paper, modes d), e) and the combined mode will be grouped under the label end debonding failure mode. Yao et al. [7] explain that intermediate crack induced debonding, identified as mode c), normally initiates in the high moment region due to a flexural or flexural-shear crack. On the other hand, end debonding is initiated by high interfacial stresses at the plate end as explained by Yao et al [7].

Failures of FRP strengthened beams due to concrete crushing are easily predicted and this mode of failure would allow for the most efficient use of the materials while preserving an acceptable level of ductility. FRP rupture is also a mode failure that can be easily determined. Although, the FRP rupture mode is permitted in most existing guidelines, it would be preferable to avoid it as it may lead to brittle failures.

When designing an FRP strengthening scheme, it is generally most difficult to predict and control the debonding modes of failure. In fact, a large number of experimental studies reported premature failures by debonding rather than concrete crushing or FRP rupture. For this reason, considerable research effort has been expended on understanding the underlying factors that cause FRP debonding, to develop models for predicting debonding, and to propose design guidelines to minimize the risk of its occurrence under design loading conditions [2,8,9,10].

Most analytical and experimental studies have focused on studying the end debonding mode of failure, while intermediate crack induced debonding has received much less attention and few

models have been developed for predicting its occurrence. Nevertheless, the debonding failure mechanisms are still found to be complicated processes that are not fully understood, making it difficult for engineers to estimate the actual capacity of FRP-strengthened concrete beams and thus making them reluctant to use this new technology on a wide scale.

Over the last decade, several guidelines for use of FRP in construction have been developed [11, 12, 13, 14, 15]. Many of these guidelines are under constant refinement as more information is gathered on the behavior of structural components strengthened using FRP. In particular, the American Concrete Institute (ACI) published the first version of its guidelines in 2002 and released a revised version in 2008. Most of these and other proposed design criteria have used equations that provide lower bounds for the beam's capacity based on limited sets of experimental data or else developed advanced fracture mechanics models without following modern methods for developing reliability-based design codes and specifications that would take into consideration the uncertainties associated with the input parameters as well as the modeling assumptions. Triantafillou in 1992 [16] was the first to propose reliability-based sets of design equations for FRP-strengthened beams but only looked at the concrete crushing and FRP rupture modes. A recent study by Atadero and Karbhari [17,18] developed reliability-based criteria for debonding using fracture mechanics models but the approach considered only randomness in the input parameters and did not account for the modeling uncertainties, also known as systemic uncertainties, which in many cases may be more significant than the parametric uncertainties. The implementation of reliability-based design specifications is extremely important for FRP-strengthened concrete beams due to the large level of variability observed in experimental tests results and the uncertainties in determining the material properties.

The objective of a study currently underway at the Technical University of Catalonia (UPC), in Barcelona is to propose appropriate reliability-based design equations with properly calibrated safety factors that can be used during the design of a strengthening scheme to enhance the flexural capacity of existing concrete bridge beams. In a first step of this study, a statistical analysis of available models is performed and presented in this paper. The goal of this analysis is to study the systemic uncertainties associated with existing models. To this end, experimental results on strengthened concrete beams that failed due to either end debonding or intermediate crack induced debonding are compared to several available proposed predictive and design models to determine the most appropriate ones for implementation in design codes. The experimental database was carefully constructed from an extensive survey of the published literature, and presented in Appendix A and Appendix B. The statistical analysis of the selected existing debonding models will provide the necessary information to calibrate appropriate safety factors that should be applied in conjunction with the selected models so that future design codes would provide the engineers with the tools necessary to use FRP-based concrete beam strengthening schemes that would lead to uniform and consistent reliability levels.

2. Experimental Database

Several models have been developed over the last two decades to predict the debonding of FRP-strengthened concrete beams or to propose design equations to avoid its occurrence. Many of these models were calibrated using a limited database generated by each model's own developers. To validate the accuracy of existing models, it is important to compare the predicted results against a very broad set of experimental laboratory and in-situ data that mimic as closely as possible the size and environmental conditions where FRP-strengthened beams will be applied. To perform a valid statistical analysis, each model should be compared to a wide data set that extends beyond the limited set which was used to calibrate the model. A rigorous statistical analysis that investigates the biases and variability of a model is the necessary first step toward performing a reliability

analysis of strengthened concrete beams and calibrating a set of appropriate safety factors for implementation in design specifications.

Over the last few years, several authors such as Smith & Teng [6], Colotti et al. [19] and Wu and Niu [3] have assembled from the existing literature exhaustive and useful experimental databases following clearly defined criteria. Due to the continuous ongoing effort in testing FRP strengthened beams, an updating of these previously published databases has been undertaken as part of this study to assemble a more comprehensive list that specifically includes recent test results and, to the extent possible, those conducted on large scale beams which may prove to be critical for the statistical analysis given the importance of scale effects. To that end, two large databases are assembled from the available literature, as reported in Appendix A and Appendix B. The databases include results for FRP strengthened concrete beams and small scale beam specimens. The first database provided in Appendix A includes 161 four-point bending tests that resulted in end debonding failures (see tables A.1 to A.6). The database was assembled from 34 testing programs carried out between 1991 and 2007. The test data is classified based on the material type and installation process as follows: 90 beams were reinforced by wet lay-up carbon sheets, 57 beams reinforced by prepeg carbon plates, 7 beams reinforced by wet lay-up glass sheets, and 7 beams reinforced by prepeg glass plates. This database includes beams that failed by end debonding including failures due to concrete cover separation, plate end interfacial debonding, mixed concrete cover separation, end interfacial debonding mode, and critical diagonal crack debonding. Table 1 gives a summary of the experimental programs including the reference from which the information was collected, the number of beams, the beam sizes, the FRP type and thickness as well as the shear load at failure. All the tests were performed for four-point bending labeled (f.p.b.t.) in Table 1. It is noted that all the tests were performed on laboratory scale models where the longest beam was about 4.5 m in length. The maximum observed failure load was 125 kN. These maximum length and load ranges are significantly smaller than would be expected in many structural applications particularly for bridges.

The second database provided in Appendix B includes strengthened beams that failed as a result of intermediate crack induced debonding (tables B.1 to B.6). The data set consists of 187 beam tests from 38 experimental programs carried out between 1996 and 2007, as summarized in Table 2. When separated by material type, the list includes 125 beams reinforced by carbon sheets, 4 beams reinforced by glass sheets, and 32 beams reinforced by aramid sheets. The loading configurations for these test specimens consisted of: a) four-point bending (f.p.b.t); b) three-point bending (3.p.b.t); and c) one-point bending of cantilevered beams (c.b.t). In this case, several researchers tested longer beams with spans up to 7.2 m and reported failure loads of up to 548.5 kN.

The experimental tests selected from the literature were those for which most material and geometric characteristics were clearly reported. To assemble a consistent database, the following criteria were used:

- all beams and slabs have rectangular cross-sections, are conventionally reinforced with steel rebars and strengthened with constant-thickness carbon, glass or aramid FRP sheets;
- failure of the beam was by end debonding or intermediate crack-induced debonding;
- the FRP sheet was neither prestressed nor anchored in any form at its ends;
- the beam did not experience prior cyclic loading after being repaired with FRP and before being tested statically to debonding failure;
- sufficient details about various geometric and material parameters were provided to enable the use of the results with confidence.

The above stated criteria mostly follow those previously set by Smith and Teng [6]. However the 4th criterion has been restricted to eliminating repaired beams that were subjected to cyclic

loading before the test. Therefore, the database assembled in this study will include beams that have been strengthened with FRP after being damaged from statically applied high loads. Because many of the reports did not provide all the data required as input for some models, a few assumptions are made to fill in the gaps. For that purpose, the recommendations of Smith and Teng [6] were followed. Specifically, when information is not provided, the concrete cover is assumed to be 10% of the total depth. This value was suggested for being the average concrete cover ratio used in the beams that provided cover information. Also, a modulus of elasticity of 200GPa was adopted for steel reinforcement when the modulus was not specified [6].

To calculate the concrete material characteristics, the ACI440 [11] models were used, as follows:

$$E_c = 4730\sqrt{f'_c} \quad f_{ct} = 0.53\sqrt{f'_c} \quad f'_c = 0.8f_{cu} \quad (1)$$

where E_c (in MPa) is the elastic modulus of concrete, f_{ct} (in MPa) is the splitting tensile strength of concrete, f'_c (in MPa) the concrete cylinder compressive strength, and f_{cu} (in MPa) the concrete cube compressive strength. These relationships are expected to produce on the average comparable results to experimental data [6].

In many instances, the adhesive layer thickness was not reported. In such cases, as proposed by Smith and Teng [6], the average thickness obtained from available test data was adopted. Specifically, the adhesive layer thickness of pultruded plates was assumed, where it is not specified, to be $t_a=2$ mm. For wet lay-up installations, the adhesive layer thickness is worked out using the following formulas for a single layer of FRP:

$$t_a = \frac{t_{frp,g} - t_{sheet}}{2} \quad (2)$$

$$t_{frp} = t_a + t_{sheet} \quad (3)$$

Where $t_{frp,g}$ is the gross thickness of all the FRP and adhesive layers assuming two equal layers of adhesive above and below the layer of fibers. t_{frp} represents the total effective thickness of each single layer of FRP in the strengthened beam. If more than one sheet were used, t_{frp} must be multiplied by the number of sheet layers. If the available information was still insufficient for using Eq. (2) and (3), t_a was assumed to be 0.42 mm based on measurements of samples formed from wet lay-up sheet by Smith and Teng [6]. Due to this additional uncertainty with wet lay-up plates, test data of beams with wet lay-up plates and those with pultruded plates are appropriately differentiated from each other during the data analysis process.

When the modulus of elasticity of the adhesive was not provided, a value of 8500 MPa was used, as proposed by Smith and Teng [6].

The databases listed in appendix A and appendix B, provide the RC beam details as follows: width of beam b , overall depth of beam h , distance from beam compression face to centroid of steel tension reinforcement d_s , distance from beam compression face to centroid of steel compression reinforcement d'_s . The properties of concrete are listed if available from the experimental investigation or using Eq. (1).

The properties of the steel reinforcement are provided for the moduli of elasticity of the steel tension reinforcement E_s , steel compression reinforcement E'_s and steel shear reinforcement (stirrups) E_{sv} . The yield strengths of the tension, compression and shearing reinforcements are denoted respectively as f_{ys} , f'_{ys} and f_{ys} . The corresponding cross-sectional areas are respectively denoted by A_s , A'_s , and A_{sv} .

The FRP material and installation procedure are provided under the column labeled “type” using the letters C, G or A in reference to Carbon, Glass or Aramid Fibers respectively. The letters W and P refer to wet lay-up procedure and pultruded fibers sheets respectively.

The FRP material properties are listed for the modulus of elasticity E_{frp} , and the tensile strength in the main fiber direction f_{frp} , sheet thickness t_{frp} , and sheet width b_{frp} as well as the distance from the support to the nearer end of the soffit plate a .

The adhesive properties provided are the modulus of elasticity E_a , and thickness of adhesive layer t_a . The width of resin layer b_a is assumed to be always equal to the FRP width b_{frp} .

The loading configuration and the failure load are described by the distance from the support to the nearest applied load, B , the span of the beam, L and the maximum shear force in the beam at debonding V_{exp} .

The statistical analysis of existing analytical and design models is verified by comparing the results predicted by the models to the experimental results assembled in the database. The analytical and design models that have been studied are briefly discussed next.

3. Review of models for debonding of FRP-strengthened RC beams

An exhaustive review of existing debonding models for end-debonding or intermediate crack (IC) debonding in FRP-strengthened RC beams was performed. A distinction between models developed from beam tests (type-A models) and models based on prism tests (type-B models) was done. A preliminary analysis of both type-A and type-B models was performed to investigate their reliability. The models were applied to two different databases assembled from the scientific literature; one database is for prism tests and the other is for beam tests. A better agreement, between analytical and experimental results was shown by the models which were developed based on beam tests [20]. Therefore, this paper will only analyze the models which seem to provide the best representation of the bending behavior of strengthened RC beams.

Fifteen models, which had been proposed between 1990 and 2007 and validated by experimental tests were selected for this analysis. From a theoretical point of view, the existing models for debonding behavior may be classified into three categories based on their approach, namely: models based on: a) materials strength, b) on fracture mechanics and c) on experimental data fitting. However, in practice, it is difficult to find models purely based on just only one of the mentioned approaches. In fact, many models are a combination of strength of materials and experimental fitting [21, 22,23]. Other models are based on fitting experimental data into empirical equations or used data fitting techniques to supplement or correct the strength of materials or fracture mechanics models, as done in the models by Zhang et al. [28], Wang and Ling (cited in [5]) and Raoof and Hassanen [25]. The Ziraba model [26] combines the strength model with fracture mechanics approaches. The group of models based on fracture mechanics look at the delamination of FRP sheets from the concrete substrate when a crack forms and propagates along the interface between the two materials. This group includes the Raoof and Zhang model [24], the Wang and Ling model (cited in [5]), Raoof and Hassanen model [25], Ziraba et al. model [26], Wu and Niu model [3], and Casas and Pascual model [27]. Several of these models had been previously studied by Smith and Teng [5] and are analyzed again in this paper using a consistent statistical method that is appropriate for implementation during the reliability-calibration of partial safety factors. The statistical analysis serves to provide a common basis for comparing the various models to the extended database assembled in this study.

Table 3 provides a summary of the principal methods used to predict the end-debonding and intermediate crack induced debonding along with the material parameters that these methods require as input. A rough classification of the models into the three groups a, b and c listed above is provided based on the primary principles used to develop them even though, as mentioned earlier,

most theoretical models utilized empirical corrections to modify the equations. It is noted that the classification in Table 3 is different than that used by Smith and Teng [6]. Smith and Teng [6] used a classification where the models are grouped into: shear capacity based models, concrete tooth models and interfacial stress based model. The first group includes the models based on the shear strength of the concrete with no or only partial contribution of the steel shear reinforcement [5,6,21,22]. The concrete tooth models take into account the concept of a cantilevered concrete “tooth” between two adjacent cracks. [5,24,25]. The last group assumes that end debonding occurs due to high interfacial stresses between the FRP and concrete cover [3,26,27]. Additional information on the models analyzed in this paper is provided in this section.

Oehlers’ model

One of the first models applied to study the debonding of FRP-strengthened RC beams was the model proposed by Oehlers [21] which was originally developed for RC beams strengthened with steel plates. The model is based on the shear force acting at the plate end while taking into account the effect of any coexistent moment. Using the classic ultimate strength analysis method, the flexural capacity of an externally plated reinforced concrete beam can be determined, considering the plate as part of the reinforcement. The moment applied to the end of the plate, which provoked the peeling, is given by the following equation:

$$M_{db,f} = \frac{(E_c I_{0,c}) f_{ct}}{0.901 E_{frp} t_{frp}} \quad (4)$$

where $M_{db,f}$ is the moment capacity of the strengthened section; $I_{0,c}$ is the cracked strengthened section’s moment of inertia transformed into equivalent concrete. The value 0.901 is a constant which is applied to obtain a safe characteristic design value.

The Oehlers’ model also uses the shear capacity of the concrete in the RC beam alone V_c , based on the Australian code equation, given as:

$$V_{db,s} = V_c = [1.4 - (d / 2000)] b d (\rho_s f'_c)^{1/3} \quad (5)$$

where $\rho_s = A_s / b d$ is the steel tension reinforcement ratio. The moment-shear interaction equation that should be used for checking the adequacy of an FRP-strengthened design is:

$$\frac{M_{db,end}}{M_{db,f}} + \frac{V_{db,end}}{V_{db,s}} \leq 1.17 \quad (6)$$

which considers the applied shear and moment at the end of the FRP plate, $M_{db,end}$ and $V_{db,end}$.

Smith and Teng’s model

For the cases when the ratio of the applied moment to the ultimate moment capacity $M_{db,end} / M_u \leq 0.67$, Smith and Teng [6] proposed a safe design equation that considers only the shear force:

$$V_{db,end} \leq 1.5V_c \quad (7)$$

where V_c is the concrete beam's shear capacity as specified by the design codes.

Teng and Yao's model

An empirical predictive model for flexural debonding of a plate end located in a pure bending region, was proposed by Teng and Yao [22,23]. Accordingly, the moment capacity is expressed as:

$$M_{db,f} = \frac{0.488M_{u,0}}{(\alpha_{flex} \alpha_{axial} \alpha_w)^{1/9}} \leq M_{u,0} \quad (8)$$

where α_{flex} , α_{axial} and α_w are three dimensionless parameters defined respectively by:

$$\alpha_{flex} = \frac{(EI)_{frp,c} - (EI)_{0,c}}{(EI)_{0,c}}, \quad \alpha_{axial} = \frac{E_{frp} t_{frp}}{E_c d}, \quad \alpha_w = \frac{b}{b_{frp}}, \quad \frac{b}{b_{frp}} \leq 3 \quad (9)$$

where $(EI)_{c,frp}$ and $(EI)_{c,0}$ are the flexural rigidities of the cracked section with and without a strengthening plate, respectively and $M_{u,0}$ is the theoretical ultimate moment of the unplated section which is also the upper bound of the flexural debonding moment $M_{db,f}$. The limitation imposed on the width ratio reflects the limitation of the test data used by Teng and Yao [22,23]. Eq. (8) is a best-fit expression of the results of eighteen tests.

The shear debonding strength is the sum of different values which represent the contributions of the concrete V_c , the strengthening plate V_{frp} , and the internal reinforcement V_s :

$$V_{db,s} = V_c + V_{frp} + \varepsilon_{v,e} \bar{V}_s \quad (10)$$

where $\bar{V}_s = A_{sv} E_{sv} d_e / s_v$. $\varepsilon_{v,e}$ is the strain in the steel shear reinforcement, referred to here as the effective strain, and this effective strain may be well below the yield strain of the steel shear reinforcement at debonding failure.

The best-fit expression for $\varepsilon_{v,e}$ is given by

$$\varepsilon_{v,e} = \frac{10}{(\alpha_{flex} \alpha_E \alpha_t \alpha_w)^{1/2}} \quad (11)$$

$$\text{where } \alpha_E = \frac{E_{frp}}{E_c}, \quad \alpha_t = \left(\frac{t_{frp}}{d} \right)^{1,3} \quad (12)$$

α_E represents the elastic moduli ratio and α_t the ratio between the plate thickness and the effective depth of the section.

As done by Oehlers [21], an interaction between plate end shear and bending has been proposed as follows:

$$\left(\frac{V_{db,end}}{0.85V_{db,s}}\right)^2 + \left(\frac{M_{db,end}}{0.85M_{db,f}}\right)^2 = 1.0 \quad (13)$$

Teng and Yao [22,23] explain that for design purposes, the shear capacity of concrete beams, V_c , to be used in Eq. (10) can be obtained from current code specified equations. The contributions of V_{frp} are usually small enough to be ignored. These simplifications yield conservative predictions of the debonding load when compared to most test results. In their study, three different codes were used to calculate the shear capacity of the concrete, V_c : the British code (BS8110), the Australian code (AS600) and the American code (ACI318).

Jansze's model

Another model originally developed for steel plated beams but has been proposed for application to RC beams was developed by Jansze (cited in [5]). The model considers that shear cracking starts in the RC beam, but does not account for the contribution of shear reinforcement. The critical shear force in the RC beam at the plate end that causes debonding $V_{db,end}$ is given as a function of the shearing stress, τ_{PES} , by:

$$V_{db,end} = \tau_{PES} bd = 0.18 \sqrt[3]{3 \frac{d}{B_{mod}}} \left(1 + \sqrt{\frac{200}{d}}\right) \sqrt[3]{100 \rho_s f'_c} bd \quad (14)$$

where B_{mod} is a modified shear span, equal to:

$$B_{mod} = \sqrt[4]{\frac{(1 - \sqrt{\rho_s})^2}{\rho_s}} da^3 \quad (15)$$

If B_{mod} is greater than the actual shear span, B , then the average value of both should be used $(B_{mod} + B)/2$. Jansze model (cited in [5]) appears to be invalid for plates that extend all the way to the support as B_{mod} becomes zero and Eq.(15) predicts that debonding will never take place.

Ahmed and van Germert's model

Ahmed and van Germert (cited in [5]) modified Jansze's model to make it more suitable for use with FRP-plated RC beams by introducing corrections to account for the differences in the FRP and steel material properties. The critical shear force is given as follows:

$$V_{db,end} = (\tau_{PES} + \Delta \tau_{mod}) bd \quad (16)$$

with

$$\Delta \tau_{mod} = \tau_{PES} bd \left(\frac{S_s}{I_{s,c} b_{frp}} - \frac{S_{frp}}{I_{frp,c} b_a} \right) + 6188.5 \left(\frac{\tau - 4.121}{bd} \right) \quad (17)$$

$$\tau = \left(0.15776 \sqrt{f'_c} + \frac{11.2366 \rho_s d}{B} \right) + 0.9 \frac{A_{sv} f_{sv}}{sb} \quad (18)$$

where τ_{PES} is the same shearing stress defined by Jansze, S_{frp} and S_s are the first moments of the FRP plate area, and that of an equivalent steel plate about the neutral axis of a cracked plated section transformed to concrete. The equivalent steel plate is one that has the same total tensile capacity and width as that of the FRP plate, but with an equivalent thickness determined assuming that the yield stress of steel is 550MPa, $I_{frp,c}$ and $I_{s,c}$ are the moments of inertia of a cracked plated section transformed to concrete with a FRP plate and an equivalent steel plate respectively. The increase in shear strength offered by the shear reinforcement is also included as seen in Eq.(14).

Colotti et al.'s model

Colotti et al. [19] proposed a model based on truss analogy. The actual load-carrying capacity of a plated beam is then determined as the minimum value obtained from four different shear strength expressions corresponding to: (a) plate-debonding failure; (b) shear failure; (c) tension/concrete crushing failure; and (d) plate rupture.

The ultimate load for the bond failure mode:

$$V_{db,end} = p_y d \left[\phi + \alpha - \sqrt{(\phi + \alpha)^2 - 2\phi\beta} \right], \quad p_y > 0 \quad (19)$$

where the terms α , β and ϕ are defined by $\alpha = a/d$, ratio of shear span to beam effective depth; $\beta = l_a/d$, ratio of plate length in shear to beam effective depth; and $\phi = U_y/p_y$, ratio of bond strength to stirrup strength. The limiting bond strength is given by, $U_y = b_m [2.77 + 0.06(f'_c - 20)]$ for $f'_c > 20$ MPa. The effective width of the plate-adhesive interface, b_m , is assumed to be the average of the beam and plate widths such that: $b_m = (b + b_{frp})/2$.

Shear failure mode in the concrete web or yielding of the steel stirrups are assumed to occur when:

$$V = \frac{bdf_c}{2} \left[\sqrt{1 + \alpha^2} - \alpha \right] + p_y d \alpha \quad \text{for } 0 \leq \frac{p_y}{bf_c} \leq \frac{\sqrt{1 + \alpha^2} - \alpha}{2\sqrt{1 + \alpha^2}} \quad (20a)$$

$$V = \sqrt{p_y b d^2 f_c \left(1 - \frac{p_y}{bf_c} \right)} \quad \text{for } \frac{\sqrt{1 + \alpha^2} - \alpha}{2\sqrt{1 + \alpha^2}} \leq \frac{p_y}{bf_c} \leq 0.5 \quad (20b)$$

$$V = \frac{bdf_c}{2} \quad \text{for } \frac{p_y}{bf_c} > 0.5 \quad (20c)$$

The failure of the concrete web or the failure of the longitudinal reinforcement occurs when the shearing force is:

$$V = p_y d \left[\sqrt{\frac{2T_y}{p_y d} + \alpha^2} - \alpha \right] \quad \text{for } \frac{p_y}{bf_c} > \frac{\sqrt{1 + \alpha^2} - \alpha}{2\sqrt{1 + \alpha^2}} \quad (21a)$$

$$V = \frac{bdf_c}{2} \left[\sqrt{\frac{4T_y}{bdf_c} \left(1 - \frac{T_y}{bdf_c} \right) + \alpha^2} - \alpha \right] \quad \text{for } \frac{p_y}{bf_c} \leq \frac{\sqrt{1 + \alpha^2} - \alpha}{2\sqrt{1 + \alpha^2}}, T_y \leq 0.5bdf_c \quad (21b)$$

$$V = \frac{bdf_c}{2} \left[\sqrt{1 + \alpha^2} - \alpha \right] \text{ for } \frac{p_y}{bf_c} \leq \frac{\sqrt{1 + \alpha^2} - \alpha}{2\sqrt{1 + \alpha^2}}, \quad T_y \leq 0.5bdf_c \quad (21c)$$

FRP tensile rupture, failure of the longitudinal steel reinforcement or concrete crushing in compression occurs when:

$$V = \frac{M_u}{a} \quad (22)$$

where the flexural capacity of the cross section is calculated as specified in ACI440 Guide [11], with $\varepsilon_p = \varepsilon_{cu}(d/c) - \varepsilon_{bi} \leq k_m \varepsilon_{pu}$.

The term k_m , defined to limit the strain in the FRP reinforcement to prevent debonding or delamination, is defined as follows

$$k_m = \begin{cases} 1 - \frac{nE_{frp}t_{frp1}}{428000} & \text{for } nE_{frp}t_{frp1} \leq 214000N/mm \\ \frac{107000}{nE_{frp}t_{frp1}} & \text{for } nE_{frp}t_{frp1} > 214000N/mm \end{cases} \quad (23)$$

where n is the number of plies of FRP reinforcement; t_{frp1} is the nominal thickness of one ply.

In the numerical investigation, the authors made the following assumptions: $d=0.9h$, $\nu=0.7$, $s_c=l_c/5$. Furthermore, the tensile strength of concrete f_t and the crack spacing size l_c , according to Eurocode2 [29] were assumed to be $f_{tc}=1.3\alpha_1\alpha_2f_c^{2/3}$ (MPa), $l_c=50+0.25k_1k_2\phi_l/\rho_r$, with $k_1=0.8$, $k_2=0.5$, $\rho_r=A_s/(2.5bd)$ and ϕ_l is the diameter of longitudinal bars.

Raoof and Zhang's model

Zhang et al. [28] and Raoof and Zhang [24] developed a strength model for simply-supported RC beams reinforced by steel plates subject to three or four point bending that also could be used for FRP sheets. They base their model on the shear strength of a single concrete "tooth". To prevent the debonding failure mode, the proposed limiting value for the stress should be compared to the stress in the plate directly under a point load. Elastic behavior and no-interaction between adjacent cracks were assumed. The critical point in the concrete "tooth" is assumed to be at the point of contact between the concrete and the longitudinal steel reinforcement. The tensile stress at this point is represented by:

$$\sigma = \frac{M}{I} \left(\frac{l_{min}}{2} \right) \quad (24)$$

where $M=\tau l_{min} b_{frp} d'$ and $I=b(l_{min})^3/12$. Here, l_{min} represents the minimum crack spacing, τ the shear stress at the interface between the concrete and the plate, I the moment of inertia of the tooth, and M the moment at the base of the tooth. Substituting M and I into Eq.(25) and assuming $\sigma=f_{ct}$, the shear stress at the interface between the concrete and the strengthening plate based on a minimum stabilized crack spacing can be determined as follows.

$$\tau_{\min} = \frac{f_{ct} l_{\min}}{6h'} \frac{b}{b_{frp}} \quad (25)$$

The authors assumed the simultaneous failure of all end anchorage teeth at debonding. The minimum width of a crack $l_{\min}$, termed the minimum stabilized crack spacing, is given by:

$$l_{\min} = \frac{A_c f_{ct}}{u(\Sigma O_{bars} + b_{frp})} \quad (26)$$

where A_c is the area of concrete in tension, u the strengthening plate -to-concrete average bond strength, ΣO_{bars} the total perimeter of the tension reinforcing bars. It is assumed that $u=0.28\sqrt{f_{cu}}$ (in MPa) and $f_{ct}=0.36\sqrt{f_{cu}}$ (in MPa). Considering a RC beam reinforced by a single plate, A_e is twice the distance from the centroid of the tension reinforcement to the base of the RC beam multiplied by the width of the RC beam.

The minimum stress in the strengthening plate $\sigma_{s,min}$ required to cause flexural cracking and failure of a tooth covering the minimum stabilized crack spacing can then be determined as follows:

$$\sigma_{s(min)} = 0.154 \frac{L_a h_l b^2 \sqrt{f_{cu}}}{h' b_{frp} t_{frp} (\Sigma O_{bars} + b_{frp})} \quad (27)$$

where L_a is an effective length of the plate for end anchorage, and h_l is the distance from the centroid of the tension reinforcement to the base of the RC beam.

In Zhang et al. [28], the effective length for end anchorage was taken as the length of the plate in the shear span. Raoof and Zhang [24] assumed that the anchorage length is equal to the smaller value between the length given in [28] and l_a , defined as follows:

$$l_a = l_{\min} (21 - 0.25l_{\min}), \quad l_{\min} \leq 72mm \quad (28)$$

$$l_a = 3l_{\min}, \quad l_{\min} > 72mm \quad (29)$$

This effective length model was calibrated to match test data of steel plated RC beams that failed by plate end debonding.

Once the stress in the plate is known, the moment that causes the separation of the plate, at the location where the stress is calculated, can be obtained using a conventional section analysis with the assumption that plane sections remain plane during bending and taking into account the tensile strength of the concrete. The higher bending moment is used assuming a minimum ($l_{\min}$) and a maximum ($l_{\max}=2l_{\min}$) stabilized crack spacing. If the end of the plate does not reach the shear span zone, the effective length should be obtained from Eq. (28).

Wang and Ling's model

Wang and Ling (cited in [5]) modified the previous model [28] to adapt it to match the results for FRP strengthened beams. They modified the definition of the minimum width of a crack, proposed by Raoof and Zang in Eq.(26) by the following expression:

$$l_{\min} = \frac{A_c f_{ct}}{u_s \Sigma O_{bars} + u_{frp} b_{frp}} \quad (30)$$

where $u_s = 0.313 \sqrt{f'_c}$ is the average bond strength between the steel tension reinforcement and the concrete, while u_{frp} is the average bond shear strength between the FRP and the concrete which was taken as 1.96 MPa.

Raoof and Hassanen's models

Raoof and Hassanen [25] tried to improve Zhang et al. model [28] proposing two expressions for the effective anchorage length of the FRP plate. These models will be called model I and model II.

In the first model [25], Eq. (30) is used to calculate the minimum stabilized crack spacing. The effective length of the FRP plate for end anchorage is taken to be the smaller of the plate length in the shear span and the following lengths which were calibrated based on the authors' experimental test data:

$$l_a = l_{\min} (24 - 0.5 l_{\min}), \quad l_{\min} \leq 40mm \quad (31)$$

$$l_a = 4l_{\min}, \quad l_{\min} > 40mm \quad (32)$$

In Raoof and Hassanen model II [25], u_{frp} is equal to 0.8MPa, as defined by Wang and Ling (cited in [5]), while u_s is equal to $0.28 \sqrt{f_{cu}}$, as defined by Zhang model [28]. The effective length of the FRP plate for end anchorage is calculated as done in the Raoof and Hassanen model I [25], using the new value of FRP to concrete bond strength:

$$L_{p2} = l_{\min} (11.6 - 0.17 l_{\min}), \quad l_{\min} \leq 56.5mm \quad (33)$$

$$L_{p2} = 2l_{\min}, \quad l_{\min} > 56.5mm \quad (34)$$

Once the stabilized crack spacing and the effective length of FRP plate for end anchorage are determined, the remainder of the analysis follows the same steps given by Zhang et al.

Ziraba et al. models

Two debonding strength models were proposed by Ziraba et al. [26] for steel plated RC beams. Model I is used to predict plate end interfacial debonding, while Model II is used for predicting concrete cover separation.

Model I proposes an expression for the shear capacity of an RC beam, at the plate end, to cause plate end interfacial debonding:

$$V_{bd,end} = \frac{f'_c}{C_{R1}} \left[\frac{C}{\alpha_1 f_{ct} (1 + \alpha_2 C_{R2} \tan \phi)} \right]^{4/5} \quad (35)$$

where

$$C_{R1} = \left[1 + \left(\frac{K_a}{E_{frp} b_{frp} t_{frp}} \right)^{1/2} \frac{M_0}{V_0} \right] \frac{b_{frp} t_{frp}}{I_{trc,frp} b_a} (d_{frp} - x_{trc,frp}), \quad (36a)$$

$$C_{R2} = t_{frp} \left(\frac{K_n}{4E_{frp} I_{frp}} \right)^{1/4} \quad (36b)$$

while α_1 and α_2 are empirical multipliers calibrated from numerical studies for RC beams retrofitted with steel plates. C is the coefficient of cohesion and φ the angle of internal friction. This is the first known model that takes into account the adhesive layer properties where the shear stiffness K_a and the normal stiffness K_n of the adhesive layer are defined as:

$$K_a = \frac{G_a b_a}{t_a}, \quad K_n = \frac{E_a b_a}{t_a} \quad (37)$$

with G_a being the shear modulus of the adhesive layer respectively. $I_{trc,frp}$ is the moment of inertia of the cracked plated section transformed into FRP, $x_{trc,frp}$ is the neutral axis depth of this transformed cracked section (distance from the compression face to the neutral axis), I_{frp} the second moment of area of the FRP plate alone, d_{frp} the distance from the compression face of the RC beam to the centroid of the FRP plate, and M_0 and V_0 the bending moment and shear force respectively at the plate end.

This relationship is subject to the constraint of $a/h < 3$. The following values for α_1 , α_2 and φ were specified in [26]: $\alpha_1=35$, $\alpha_2=1.1$ and $\varphi=28$. For the present study C has been taken as 7.15MPa which is an average of the values given in [26].

In Model II, Ziraba et al. [26] defined the shear beam capacity by modifying the ACI code equation; the contribution of steel reinforcement to the shear strength, V_s , is multiplied by a coefficient, k , which is empirically derived to account for the stirrup efficiency and is related to the peak interfacial normal stress at the plate end.

$$V_{db,end} = (V_c + kV_s) = 1/6 (\sqrt{f'_c} + 100\rho_s) b d + k(A_{sv} f_{yv} d) / s \quad (38)$$

where V_c and V_s are respectively the contributions of concrete and steel shear reinforcement to the shear capacity of an RC beam. The k value is based on a regression analysis of the test results of fourteen steel plated RC beams which failed by concrete cover separation:

$$k = 2.4e^n \quad \text{and} \quad n = -0.08C_{R1}C_{R2} \times 10^6 \quad (39)$$

Wu and Niu model

Wu and Niu [3] proposed a methodology for predicting the debonding failure caused by intermediate flexural cracks in flexural strengthening of RC beams with epoxy-bonded FRP sheets. The validity of the model was verified by comparing to an assembled database. The model based on fracture mechanics principles compares the maximum tensile force in the FRP to P_{max} which represents the maximum transferable load, derived from a pull-push or pull-pull shear tests. P_{max} can be expressed as follows:

$$P_{\max} = b_{frp} \sqrt{2G_f E_{frp} t_{frp}} \quad (40)$$

where G_f is the interfacial fracture energy consumed during debonding.

Taking into account a cracked FRP-strengthened beam, $P_{\max}$ represents the maximum stress value which the FRP can transfer at the beam without failing due to intermediate-crack (IC) debonding. This value is measured as difference between the FRP tension at maximum moment point and a L'_e spacing section.

The equivalent transfer length, L'_e is twice of the effective transfer length determined from the simple bond tests; it requires to develop the maximum transferable FRP tensile stress $P_{\max}$, and can be computed as:

$$L'_e = 2L_e = \frac{1,3\sqrt{E_{frp} t_{frp}}}{f_c^{0,095}} \quad (41)$$

Casas and Pascual's model

Casas and Pascual [27] proposed a new, easy to apply model for the end debonding and the intermediate crack induced debonding of FRP-strengthened concrete beams. Debonding occurs when the nominal average shear stress reaches the allowable stress value $\tau_{\max}$. Then, the tensile force per unit width in the FRP sheet, T_u , depends on the compressive strength of concrete, f'_c , and the effective bond length L_e .

$$T_u = \tau_{\max} L_e = 0.996\sqrt{f'_c} L_e \quad (42)$$

when the stiffness of the strengthened concrete element is much higher than the FRP stiffness, L_e is defined as:

$$L_e = \sqrt{\frac{k_{frp}}{g_b}} \quad (43)$$

where $k_{frp} = E_{frp} t_{frp}$ is the axial stiffness per unit length of the FRP. g_b is the shear joint stiffness of concrete plus adhesive resin, expressed as

$$g_b = \frac{g_a g_c}{g_a + g_c} \quad \text{where} \quad g_a = \frac{G_a}{t_a}, \quad g_c = \frac{G_c}{t_{ce}} \quad (44)$$

G_a and G_c are the shear modulus of the resin and concrete. Poisson ratios for concrete and resin are taken as 0.50 and 0.38 respectively for the case where the concrete is in the plastic range as explained in [30]. t_a is the resin thickness used in the application of the repair and t_{ce} is the concrete thickness that can be estimated as :

$$t_{ce} = b_{frp} + 50,8 \leq h/2 \quad (45)$$

where b_{frp} and h are expressed in mm.

The failure criteria in the case of beams with two or more consecutive cracks spaced at a distance of s ($s \cong d/2$) can be found using an adjusted ultimate capacity expression by applying a coefficient β . Then, the ultimate force per unit width in the FRP is:

$$T_u = \beta \tau_{\max} L_e \quad \text{where} \quad \beta = \frac{\xi_1}{1 - \xi_2 \xi_3} \quad (46)$$

where the coefficients ξ_1 and ξ_2 are defined by the following functions:

$$\xi_1 = \frac{e^{s/L_e} - e^{-s/L_e}}{e^{s/L_e} + e^{-s/L_e}}, \quad \xi_2 = \frac{2}{e^{s/L_e} + e^{-s/L_e}} \quad (47)$$

The simplified model is based on the moment-tension interaction M-T curve which relates the moments in the beam and the tension forces in the FRP characterized by the parameter ξ_3 defined as:

$$\xi_3 = 1 - \theta \frac{k}{k'} \left(1 - \frac{M_2}{M_1} \right) \quad (48)$$

Assuming a bilinear behavior, the M-T curve is characterized by the parameters k and k' which represent the slope of the linear elastic and post-yielding lines, respectively. The value of θ was demonstrated to be approximately equal to 2/3. The ratio between the bending moment M_2 at a distance $d/2$ from the section of interest and the bending moment at this section M_1 is obtained from a linear-elastic analysis.

ACI 440.2R-08

For sections away from the section where externally bonded FRP terminates, ACI 440.2R-08 [11] indicates that intermediate crack induced debonding can be prevented if the effective strain in the FRP remains below ϵ_{fd} , defined as:

$$\epsilon_{fd} = 0.41 \sqrt{\frac{f'_c}{n E_{frp} t_{frp}}} \leq 0.9 \epsilon_{fu} \quad (49)$$

Where ϵ_{fu} is the design rupture strain in the FRP.

4. Analysis of end dedonding models

In a first step of the analysis of the models, the shear force that led to the debonding of the FRP as obtained from the experimental data base V_{exp} , is compared to the shear value V_{an} , predicted by each of the models. To be consistent with the way the results of experimental investigations on end debonding are reported, V_{an} is calculated at the end of the beam. On the other hand, for crack induced debonding, V_{an} is calculated in the maximum moment region. The ratio of V_{exp}/V_{an} is obtained for each applicable test result and the average ratio for each model is defined as the model's bias. A model's Coefficient of Variation (COV) is defined as the standard deviation of the ratios divided by the bias. The results for all the models studied in this paper are summarized in Table 4. In addition to finding the mean bias and COV, the ratios for each model are plotted on

Normal and Lognormal probability scales to identify which of these two probability distributions can best describe the scatter around the bias. The regression coefficient R^2 for the regression analysis performed on the probability scale plot is used as a measure of the goodness of the probability distribution. Thus, when the R^2 approaches 1.0 on the Normal probability plot, the bias is considered to be well represented by the Normal distribution.

In Table 4, the comparison of the results of the end debonding considering only carbon FRP sheets is divided into two groups based on the installation type: (i) carbon wet lay-up (C-W) and (ii) carbon prepeg plates (C-P). The very low number of beams reinforced by glass or aramid FRP sheets does not allow for a realistic analysis of the models. Also, in each case, the number of valid tests that could be used in the comparison was limited for various reasons. In many cases, the models require as input specific detailed information about material properties that were not reported by the experimentalists. In other cases, the geometric and other conditions under which the models are applicable were not satisfied. These conditions and requirements have vastly limited the number of usable experimental test results. For instance, it was found that many tested specimens did not satisfy the flexural strength ratio range for which the Smith and Teng model [6] is applicable. In many cases, the Colotti et al. model [19] predicts a different failure mode than the one observed from test results. Casas and Pascual's model [27] as well as Ziraba's model [26] base their predictions on the resin's properties which have been reported by only a few experimental investigators. A large number of tested specimens have extended the FRP sheets all the way to the end supports which meant that these could not be used to verify Janze's model nor that of Ahmed and van Gemert. The number of valid test results for each model is provided in Table 4. For wet lay-up installations that failed by end debonding, the maximum number of usable tests is as high as 90 for the Teng and Yao model and as low as 19 for the Casas and Pascual or the Ziraba models. For prepeg sheet installations that failed by end debonding, the number of usable tests ranged from 59 down to 38 for the Casas and Pascual and the Ziraba models.

The bias values and COV's in Table 4 show large differences in the results between the models. Most models were developed to provide conservative envelopes to the experimental results for the specific objective of using the models for design purposes. These models show various levels of conservatism expressed in terms of bias ratios ranging from just above 1.0 (1.09) for the Colotti et al. model [19] applied to CP sheets up to 3.1 for the Raoof and Hassanen Model II [25] applied to the CP sheets. On the other hand, the models of Jansze and Ahmed and van Germert overpredict the experimental results for both the wet lay-up and prepeg installations by a considerable margin, yielding bias values varying between 0.64 and 0.82. This makes these two models less likely to be useful for application as predictive models or design models unless augmented by high safety factors.

The model by Colotti et al. seems to provide on the average results that are the closest to those from the tests with an average bias of 0.90 for wet layups and a bias of 1.09 for prepeg beams respectively. This gives an indication that Colotti is the closest to being used as a predictive model. It is noted however that in many instances, this model, which was developed to not only estimate the load at failure but also the failure mode, did not predict the correct failure mode even when the predicted load at failure was close to the reported experimental value. Applying the model to the database, it is observed that only in 56.7% of the cases was the analytically predicted failure mode consistent with the observed mode.

Several models, particularly, those of Ahmed, Raoof and Hassanen, Colotti and Ziraba I, show considerable differences exceeding 10% between the biases obtained for the wet lay-up specimens and the pre-peg specimens. Although differences in the variability in the results may be expected due to the difficulty of controlling the resin's thickness in the different installations procedures, none of the models distinguishes between installations methods. Therefore, good models should provide similar biases independent of the installation method, even though the standard deviations may be different to reflect the variability in the resin thickness associated with the different installation methods.

The standard deviation and the Coefficient of Variation (COV) provide a means to evaluate the consistency of the models. The models showing very high variability, with standard deviations on the ratios of V_{exp}/V_{an} exceeding 1.0, include those of Raoof and Zhang [24], Raoof and Hassanen [25], and Ziraba I [26]. A normalized measure of the variability in the model accuracy is expressed in terms of the Coefficient of Variation. Table 4 shows that most models give very high COV's emphasizing the difficulties associated with predicting the debonding strength of FRP-strengthened concrete beams. In particular, the models by Raoof and Hassanen, Raoof and Zhang, Ziraba, Jansze and Ahmed and van Germert are associated with COV's exceeding 40%.

The models showing the lowest COV values are those of Smith and Teng for both CP and wet lay-up installations and Casas and Pascual tested with the 19 data points that provided all the necessary input parameters as well as that of Colotti for wet lay-up.

The plots of V_{exp}/V_{an} on Normal and Lognormal scale help associate the variability of each model with a probability distribution type. As an example, Figure 2 shows the plot of the ratios obtained for the Casas and Pascual model for wet lay-up on Normal probability scale. When the data points lie on a straight line, they give an indication that the data follow a Normal probability distribution. The regression coefficient of determination R^2 of the linear fit on this normal probability scale provides a measure of how well the probability distribution will predict future outcomes. Also, the regression equation provides an approach to obtain the mean and standard deviation of the Normal distribution that best describe the variation in the data. Specifically, the mean of the Normal distribution is obtained by setting the ordinate "y" of the regression equation equal to zero and solving for x. In this example of the Casas and Pascual model, the mean ratio or the bias is obtained as 2.2 which is the same value obtained from the point estimation procedure of the bias. The standard deviation is obtained by setting the ordinate "y=1" and solving for x which in this case is the sum of the mean plus the standard deviation. In this case, the standard deviation is found to be 0.52 which is higher than the 0.46 value obtained from the point estimates as shown in Table 4, but still reasonably close given the relatively large variability in the data. Thus, the Normal probability plot with a coefficient of determination $R^2=0.96$ indicates that 96% of the variation in the data can be explained by describing the ratio of V_{exp}/V_{an} as a random variable that follows a Normal probability distribution with a mean = 2.21 and a standard deviation = 0.52 or a COV=24%. The plot on the Lognormal scale (Figure 3) shows a lower $R^2=0.94$ value indicating that the Normal distribution gives a better description of the variability of the data.

Because the number of valid tests for the Casas and Pascual model was relatively small with only 19 valid tests, more tests were fitted to this model by assuming that the properties of the resin are known and equal to the average properties from the 19 tests for which such data is available. This approach is similar to the one used by Smith and Teng [6]. In this case, the thickness of the resin is assumed to be $t_a=2$ mm for C-P and 0.42 mm for C-W. The modulus of elasticity for the resin is $E_a=8500$ MPa. This increased the number of valid tests to 90 for C-W tests and to 59 for prepeg test beams. Applying the model to all 90 C-W data points, the mean of the bias is slightly reduced to 1.89 from the original 2.21, while the standard deviation increases from the original 0.46 to 0.57. The range of the tests however is widely increased to range from 186 kN to 6.70 kN. The same coefficient of determination R^2 is reported. For the C-P beams, using 59 data points instead of the original 38 data for which all the information is provided the bias changes from 2.27 to 2.13 while the COV remains practically the same at 24%.

Table 4 also lists which of the Normal or Lognormal probability distributions can best describe the variability in each of the models based on the R^2 values which are also provided in the table. For example, the table shows that the variability in the results of the model of Teng and Yao [22] applied using the ACI equations for the shear capacity of concrete beams can very well be described using a Normal probability distribution whereas the application of the Colotti model [19] for prepeg installations shows the least good fit on either the Normal or Lognormal plots. It should be noted that during the plotting on the probability scale when occasionally a single data point

would clearly shift the linear regression, this point was designated as an outlier and removed from the linear regression analysis.

In summary, the results of this first set of statistical analyses can be used to relate the actual load that is expected to lead to the debonding of an FRP strengthened concrete beam to the analytically predicted load using an equation of the form:

$$V_{actual} = \lambda V_{analytical} \quad (50)$$

where the analytically predicted load that causes failure V_{an} is calculated using as input the basic geometric and material parameters relevant to the model being used, λ is a random variable with a mean equal to the bias and a COV as given in Table 4 for each model and installation type and λ would follow the appropriate probability distribution for the model being used. It should be noted that the variability in λ as expressed in the COV values of Table 4 does not account for the variability in the input material and geometric parameters in actual installations where these parameters are not carefully controlled.

The use of Eq. (50) to represent the bias and the variability in the analytical model imply that the same bias is applicable for the whole range of loads and beam sizes and that the tests were performed on beams having geometries that adequately represent the geometries on which the models will apply. Unfortunately, as observed in the data presented in Appendix A and B, the range of beam sizes for the test specimens is relatively small and is not necessarily representative of the dimensions expected in engineering applications such as for bridges. Furthermore, most of the tests were performed on very small scale beams and few of the tests are performed on medium or full scale beams. The value of λ that is obtained from the average of the beams tested will be mostly influenced by the large number of small scale tests as compared to the fewer medium or full scale tests. A good representation of λ , must be obtained when the tested beam sizes are adequately distributed over the applicable dimension range. An approach is used next in an attempt to account for the fewer number of medium scale tests. The approach consists of applying a regression analysis rather than the point estimation method described above to study the relationship between experimental results and predicted analytical results.

A second set of statistical analyses performed as part of this study consists of plotting the experimental results expressed as V_{exp} versus the model prediction V_{an} and performing a linear regression fit of the data. The best linear expression that describes the relationship between V_{an} and V_{exp} is thus obtained. The regression is executed such that the line goes through the origin (0,0). The analysis of the regression residuals consists of a plot of the residuals (differences between V_{exp} and V_{an}) to verify their distribution around zero and the calculation of the residuals' standard deviation. In addition, the standard error of the regression's coefficient is also calculated. Figures 4, through 9 provide examples of the regression analysis performed for a number of the models. Specifically, the figures illustrate the regression results for the wet lay-up and pre-peg installations of beams that were compared to the Smith and Teng [6], Colotti [19] and Casas and Pascual model [27] for the end debonding of FRP. These results are selected for illustration because they showed relatively good relationship between the experimental and analytical as expressed in regression coefficients of determination R^2 which are higher than 0.90 for both types of installation. The results of the analyses for the plotted data and all the other models are also summarized in Table 4.

The regression analysis shows how well the relation between the experimental results and the analytical results can be represented by an equation of the form:

$$V_{exp} = a V_{an} + \varepsilon \quad (51)$$

where the regression error ε follows a Normal probability distribution with an average value of zero and a standard deviation σ_ε . The coefficient a or the slope of the linear relationship can also be considered to be a random variable with a standard deviation, σ_a . Table 4 gives a listing of the regression coefficient, a , the regression coefficient standard error, σ_a , the regression error, σ_ε and regression coefficient of determination R^2 . These data can be used to obtain the 95% confidence intervals on the regression equation as well as the 95% prediction intervals which are respectively shown in red and blue dashed lines in Figures 4 through 9 for the cases with relatively high R^2 values.

The results summarized in Table 4 indicate that all the slopes of the best fit line, “a”, have very narrow bounds with standard errors of less than 0.10. On the other hand, large variations in the overall standard error expressed as σ_ε are observed where σ_ε varies between a low of $\sigma_\varepsilon=5.91$ for the Casas and Pascual model [27] for wet lay-up installations and $\sigma_\varepsilon > 35$ for the Wang and Ling (cited in [5]), Raoof and Hassanen [25] and Zhang models [28]. These same models have also very low regression coefficients R^2 .

In most cases, the value of the regression slope of Eq. (51) is quite similar to the bias obtained from Eq. (50) highlighting the robustness of the statistical analysis process for the given range of beam sizes and beam capacities. The cases where the difference between a and the bias λ is significant are those for which the regression analysis produced low R^2 values indicating that the variability in V_{exp} is not well described by the expression of Eq. (50) or for those in which the regression equation of Eq. (51) was driven by a few points at high values of V_{an} and where the residuals are not evenly distributed around zero. This problem may be explained by the scaling effect since most of the available test results have been performed on small scale laboratory specimens. More tests need to be executed on full scale beams to verify the consistency of the current models for all pertinent scale levels.

The plots shown in Figures 4 to 9 are for the models that showed the highest R^2 values. These models also happen to have among the lowest regression standard errors. The figures show the plots of the V_{exp} versus V_{an} as well as the regression line. Also, shown in the figure are the upper and lower 95% confidence intervals of the regression line as well as the upper and lower 95% prediction intervals. The plots are shown at the same scale varying between 0 kN to 200 kN to highlight the limitations in the applicable ranges for each of the models. For example while the Smith and Teng [6] and the Colotti [19] models show a wider range for the upper and lower 95% intervals than the Casas and Pascual model [27] for wet lay-up tests analyzed with the 19 data for which all the information is provided. The limited range for which the applicability of the Casas model can be tested is readily noticed. Increasing the number of valid tests for the Casas and Pascual model from 19 to 90 by using average values for the resin material increased the range of valid test results to 186 kN. This reduced the slope of the regression line from the original 2.18 to 1.65. The standard error however increases from the original 5.91 to 20.92. It should however be noted that some of the increase in the error may reflect the deviation of the true resin properties from those assumed by taking average values.

For the prepreg tests, the range of applicability of all models is approximately the same with a high shear force of 80.10 kN. The data plots in Figure 9 show the regression results of the Casas and Pascual model. If the number of valid tests is increased from 38 to 59 by using the average properties of the resin, a smaller drop in the slope from the original 2.14 to 1.99 for the Casas and Pascual model is observed for the C-P samples as compared to the C-W samples. These results serve to further emphasize the importance of performing large scale tests for obtaining higher confidence levels in our ability to predict the strength of FRP strengthened beams and have more confidence in the design of such strengthening schemes.

5. Analysis of IC induced debonding strength models

The results shown in Table 4 for the intermediate crack-induced debonding case compare the analytical results from the Casas and Pascual Model [27], the Wu and Niu model [3] and the ACI model [11] for different subgroups of the experimental database. Different numbers of specimens are used in each comparison because the different models require different input that may not have been made available in the published test results. For example, the Casas and Pascual model requires the properties of the adhesive layer as input, whereas they are not required for the other models. Only 24 tests in Appendix B out of which 20 are reinforced concrete beams strengthened with CFRP, reported the values of the resin's thickness. The second comparison made for the Casas and Pascual model adds two large scale experiments of two beams that had been previously damaged under static loading and then strengthened using FRP sheets as reported by Casas et al. [27,31]. The third comparison of the Casas and Pascual model includes the whole set of experimental data including the tests for which no resin properties were available. In these cases, as done for the end debonding when necessary, a resin thickness t_a is assumed to be equal to 2 mm for pultruded installations and according to equations (2) and (3) for wet lay-up installations. The modulus of elasticity, E_a , is set equal to the average of the available values (7389 MPa). A parametric analysis has shown that the final results are not very sensitive to changes in this assumed value as demonstrated by Ceci [20]. For example, if E_a is changed to 3000MPa the values of V_{an} increase by a maximum of +7.0%. When E_a is set at 12000MPa, the values of V_{an} decrease by about 2.2%. Due to the low number of tests performed on beams with wet lay-up installations, the analysis of intermediate crack induced debonding does not distinguish between the method of installation and the data includes pre-peg as well as wet lay-up installations.

Wu and Niu model is based on a fracture mechanics approach to study if the crack would progress along the interface between the FRP and the concrete. The model does not require input on the resin characteristics. This model considers the possibility that the beam fails due to either concrete crushing or FRP rupture. In 5 of the 20 tests considered, the model predicted the wrong failure mode (3 FRP rupture and 2 concrete crushing). For this reason, only 15 of the 20 beams for which all the properties are provided are compared for the Wu and Niu model in the IC group of Table 4. Subsequently, all the experimental data for which the Wu and Niu predicted the correct failure mode were included in the analysis.

Similarly, the ACI method does not require the resin's properties and it only requires as input the FRP properties and the concrete strength. IC debonding occurs before concrete crushing which for design purposes, and according to ACI, is assumed to take place when the maximum concrete strain ϵ_{cu} reaches 0.003. However, it is well known that in actual situations concrete may exceed this nominal design strain value. Therefore, three cases are considered for the evaluation of the ACI model. Case 1 compares the ACI equations to experimental results where the maximum strain in the concrete remains below 0.003 and for which all the resin properties are known. Thirteen specimens satisfied these requirements. This Case 1 is checked to compare the ACI results to those of the Casas and Pascual model even though the ACI equations do not require the resin properties as input. Case 2 performs the statistical analysis for all the 68 specimens where the concrete strain, according to ACI equations, remains below 0.003. Case 3 performs the statistical analysis for 183 specimens for which the concrete strain may have exceeded the 0.003 design limit, yet the actual test failure was reported to be caused by IC debonding rather than concrete crushing.

The results of the statistical analysis summarized in Table 4 show that the Wu and Niu model [3] vastly over-predicts the failure load. This is reflected by a point estimation of the bias λ and a regression slope, a , significantly less than 1.0. The regression analysis results for the Wu and Niu model [3] are shown in Figure 10. The results of the Casas and Pascual model [27] for the original

20 beam tests are shown in Figure 11. The results of the Casas and Pascual model [27] show a point estimated bias value of 1.88 and a slope for the regression line of 1.76 which although slightly different are nevertheless within a close range. However, both figures 10 and 11 highlight the low range of values for which the Wu and Niu [3] as well as the Casas and Pascual models [27] were tested when only the beams with known resin properties were analyzed. In order to increase the range, two additional tests obtained from [27,31] have been added to the database. The inclusion of these two additional tests into the results of the Intermediate debonding database labeled Casas IC + Casas D in Table 4, show how the inclusion of a few large scale tests will alter the results of the regression analysis. As a matter of fact as noted in Table 4, the point estimation of the bias changes very little from 1.88 to 1.80 when the two large scale tests are added. These values are similar to the regression coefficient of 1.76 obtained for the original data set. On the other hand, the lack of sufficient test data along the whole range of application has meant that the point estimation of the bias becomes inconsistent with the regression fit of Eq. (50) when the additional tests are included. In this case, the regression line slope of 1.08 is significantly different from the point estimated bias. The reasons for this discrepancy are illustrated in Figure 12 that shows how the large number of data points in the lower load ranges controls the point estimation statistics, while the regression analysis provides what seems to be a more balanced evaluation of the relationship between V_{exp} and V_{an} in the whole range of the tests. It should be noted that the addition of the two large scale tests have led to a higher regression standard error σ_e of 8.34 as compared to the original standard error of 2.78. More large scale tests are needed in order to further verify the relationship between the predicted values and experimental test results. Significant differences are observed in the bias and the regression coefficients as well as in the COV's and regression errors when the data for which no resin properties are provided are included with the assumed resin properties. These differences are due to both the increase in the database as well as the assumptions on the resin properties.

The inclusion of the additional experimental data for the analysis of the Wu and Niu model gives a lower bias value which changes from 0.60 to 0.50 increasing the gap between the model results and the experimental results. This difference is only due to the increase in the database and is not related to the resin properties since the Wu and Niu model does not require the resin properties as input.

The results of the ACI440.2R model demonstrate the importance of having a large database that extends to the whole range of practical beam dimensions. The results show that when applied as intended for only the cases where concrete strain remains below 0.003 (Case 2), the ACI model shows a bias of 1.22 and a COV 0.45. While a bias above 1.0 indicates that on the average the ACI equation is conservative, the high COV of 45% would indicate that many beams will fail at lower loads than anticipated by the ACI equations. This is further confirmed when looking at the regression coefficient for case 2 which falls to 0.91 with a regression error of 36.73. The high COV and regression error are practically similar to those of the Casas and Pascual model although in the latter case, the required resin properties have to be assumed while no such properties are needed for the ACI model. When all the data are considered including those for which the concrete strain exceeds the nominal limit of 0.003 (Case 3) the value of the ACI bias drops back to 0.87 and the regression coefficient is 0.77 which are both significantly lower than 1.0.

The large differences between the regression coefficient and the bias between cases 2 and 3, reflect the fact that the regression analysis puts some additional weighting on the few large scale tests as compared to the bias which is an average value that may be more influenced by the many tests on small scale specimens. This phenomenon may be visualized in the plot of Figure 13.

One reason for the inconsistencies between the predicted load and the experimental failure load may be due to the fact that the Wu and Niu and the ACI models were calibrated based on tests performed on prisms rather than on beam tests. The use of the prism tests means that these two

models are applicable for type-1 IC debonding, where there are no other cracks between the crack where the delamination process initiates and the end of the FRP sheet. In prism tests, the tension at the end of the FRP sheet is zero, which is not the case when there are other cracks in between the initiating crack and the end of the FRP sheet. Also, the effect of the curvature in the beam is ignored in the prism tests. While the Casas and Pascual model takes these factors into account, the Wu and Niu fracture mechanics-based model considers only type-1 IC debonding. Similarly, the ACI model is based on the bond-strength model proposed by Chen and Teng, which was calibrated with a large data base of shear tests, and verified with a limited number of tests of RC beams in flexure. However, a significant number of additional tests have been used in the present analysis. This may explain why the ACI and Wu and Niu models over-predict the resistance of beams in IC debonding where a combination of type-1 and type-2 IC debonding is observed.

As expected for all IC models, the inclusion of the additional data set increases the COV associated with the point estimation of the bias by a significant amount. For the Casas and Pascual model one reason for the increase in the COV is due to the possible differences between the assumed resin properties and the real values which are not known. The COV of the Casas model increases from 18% to a very high 65%, while the Wu and Niu COV increases from 32% to 46%. The COV associated with the ACI bias also increases from 29% with the limited data set to 55% with the full data set. Figure 13 shows the plot for the ACI comparison with the full data set. The larger range of the loads that cause failure is reflected by the larger scale used. The increase in the COV with the larger database may be due to the scale effects when the models are developed based on small scale specimens as well as the preponderance of type-2 debonding in the additional beam tests included.

Similar observations to those noted above for the bias can be made for the regression coefficients. The trends in the regression standard error are also similar to those observed for the COV.

6. Conclusions

In this paper, two experimental databases were assembled to study the debonding failure mechanism of concrete beams reinforced in flexure using FRP sheets. One database is for beams that failed due to end debonding and the second database is for beams that failed due to intermediate-crack induced debonding. The database for beams that failed by end debonding includes 34 testing programs, consisting of 161 tests performed on beams of various dimensions. The 184 beams that failed in intermediate crack induced debonding were collected from 37 different experimental investigations. The databases assembled in this study build up on the data bases previously assembled by Smith and Teng [6] and Colotti [19].

The two databases were used to evaluate the suitability of existing analytical models for designing FRP strengthening schemes that resist debonding or for predicting the debonding failure mode. The goal of the analysis is to assess the applicability of existing models for calibrating design criteria and proposing reliability-based safety factors which would produce uniform levels of safety during the design of FRP strengthening schemes for concrete beams in bending. Twelve models were investigated for end debonding and three models for intermediate crack induced debonding. Based on the comparative study, the following conclusions can be drawn:

1. Many models were verified or calibrated to match the results of specific databases and may not produce similar results when the test conditions or scales are different than those of the

tests that generated the models. For example, the Wu and Niu model [3] was found to over-predict the test results reported by other investigators.

2. The objective of many model authors was not necessarily to predict correctly the load of failure, but to provide a model that will lead to safe lower bounds on the failure loads for design purposes. For example, this is the approach taken by Smith and Teng [6] and Teng and Yao [22] whose models show large values for the bias of the model which gives the average ratio of the experimental results to the analytical results. Also, the ACI model for IC debonding was developed to prevent this mode of failure and not to predict it. However, even when the average bias for the design equation may exceed 1.0 by a significant margin, the high variability in the test results may show that in many cases, the proposed equations may overpredict the load carrying capacity of FRP strengthened beams. Therefore, it is important to calibrate design equations for FRP debonding using structural reliability criteria that ensure that the design equations produce acceptable levels of safety that are consistent with those of other structural failure modes.
3. Colotti et al. proposed a model [19] based on strength of materials criteria to not only predict the failure load but also the mode of failure of strengthened RC beams and came closest to providing a predictive model rather than a safe design model.
4. The design models which gave the lowest coefficients of variation (COV) while showing consistent results for end debonding for both pre-peg and wet lay-up installations are the Smith and Teng [6] and the Casas and Pascual model [27]. The Casas and Pascual model takes into consideration the interaction between FRP, the resin and the concrete which all other models except for Ziraba [26] ignored.
5. One clear advantage of the Casas and Pascual model [27] is that it can be used for end debonding and also for intermediate crack-induced debonding.
6. The results show that many models give good predictions for wet lay-up installations but few models give the same level of accuracy for prepeg plates. Very few models take into account the difference in behavior due to the installation process.
7. Most models ignored the effect of the FRP-concrete interface and did not take the resin properties into account although recent studies have shown that the debonding process does initiate at the interface.
8. As previously reported in [27], it is noted that the scale effect is very important for the debonding mode of failure and that more tests need to be executed on full scale beams to verify the consistency of the current models for all pertinent scale levels.

The conclusions presented in this paper provide a statistical analysis of the response of FRP flexurally strengthened beams and, therefore, they are the first step on the way to developing a set of reliability-based partial safety factors to use during the design of FRP strengthening schemes for bridge decks and beams. This will be the subject of a companion paper.

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Reference	n. of tests	L [mm]	h/b [-]	Type of FRP	t _{frp} [mm]	V _{exp} [kN]	Type of test
Ahmed et al. (1999- 2000)	7	1500	2.00	C-W	0.33 – 0.67	41.5 – 70.1	f.p.b.t.
Arduini et al. (1997)	4	-	1.00 – 1.30	C-W	0.20 – 1.30	114.0 – 45.0	f.p.b.t.
Berber et al. (1999)	6	2350	2.08	C-P	0.22	50.3 – 68.5	f.p.b.t.
Benjeddou et al. (2007)	6	1800	1.25	C-P	1.20	15.1 – 20.1	f.p.b.t.
Breña et al. (2004)	11	812	1.00	C-W / C-P	0.16 – 1.19	13.9 – 26.20	f.p.b.t.
David et al. (1999)	4	2800	2.00	C-P	1.20 – 2-40	68.0 – 79.5	f.p.b.t.
Esfahani et al. (2007)	2	1600	1.30	C-W	0.18	35.5 – 37.2	f.p.b.t.
Fanning et al. (2000)	6	2800	1.56	C-P	1.20	31.0 – 51.5	f.p.b.t.
Garden et al. (1997)	17	2200	1.00 – 1.77	C-W / C-P	0.50 – 1.30	15.4 – 30.0	f.p.b.t.
Grace et al. (2005)	4	2540	1.67	C-W / C-P	0.40 -1.20	66.7 – 68.3	f.p.b.t.
Hau et al. (1999)	5	1500	1.30 – 2.60	G-W	1.66	53.0 – 79.4	f.p.b.t.
Juvandes et al. (1998)	2	1500	2.00	C-W / C-P	1.20	6.7 – 12.5	f.p.b.t.
Matthys (2000)	6	-	2.25	C-W	1.20 – 0.20	95.8 – 186.0	f.p.b.t.
Nguyen et al. (2001)	4	1330	1.25	C-P	1.20	28.1 – 65.1	f.p.b.t.
Pham et al. (2006)	11	2300	1.86	C-W	1.06 – 1.58	25.7 – 37.3	f.p.b.t.
Quantrill et al. (1996)	4	900	1.00	G-P / C-P	1.20 – 1.60	20.4 – 12.3	f.p.b.t.
Rahimi et al. (2001)	8	2742	1.33	C-W	0.80 – 1.20	35.3 – 29.7	f.p.b.t.
Ritchie et al. (1991)	5	2438	2.00	G-P	1.27 – 4.76	50.6 – 72.1	f.p.b.t.
Ross et al. (1999)	11	2742	1.00	C-P	0.50	35.6 – 84.5	f.p.b.t.
Saadatmanesh et al. (1991)	1	4575	2.22	G-P	6.00	125.0	f.p.b.t.
Sharif et al. (1994)	2	1180	1.00	G-W	2.00 – 3.00	33.0 – 34.0	f.p.b.t.
Spadea et al. (1998)	2	-	1.00	C-W	1.20 – 1.20	37.4 – 43.4	f.p.b.t.
Tan et al. (1999)	7	-	1.50	C-W	0.20	19.8 – 27.5	f.p.b.t.
Täljsten et al. (1997)	6	3600	1.50	C-W / C-P	1.40 – 2.40	71.4 – 80.1	f.p.b.t.
Teng and Yao (2007)	5	1500	1.66	C-W / C-P	1.20 – 2.63	76.0 – 99.4	f.p.b.t.
Triantafillou et al. (1992-1998)	8	1220 - 2130	1.67 – 2.00	C-W	0.49 – 1.90	12.8 – 98.2	f.p.b.t.

Table 1: Experimental tests collected in end debonding database. f.p.b.t. =four point bending test. C= carbon, G = glass, W= wet lay-up, P = pultruded.

Reference	n. of tests	L [mm]	h/b [-]	Type of FRP	t _{frp} [mm]	V _{exp} [kN]	Type of test
Beber et al. (1999)	6	2349	2.08	C	0.44 – 1.10	50.3 – 68.5	f.p.b.t.
Benjamin (2005)	8	4537	1.67	C	1.4	37.7 – 51.8	3.p.b.t.
Bonacci and Maalej (2000)	1	3650	1.48	C	0.3	250.0	f.p.b.t.
Chan et al. (2001)	4	4600	1.88	C	1.2	129.0 - 220.	f.p.b.t.
Delaney (2006)	4	1800	1.50	C	1.4	44.4 – 49.5	f.p.b.t.
Esfahani et al. (2007)	1	1600	1.33	C	0.18	54.5	f.p.b.t.
Gao et al. (2004)	3	1500	1.33	C	0.2	39.3 – 43.9	f.p.b.t.
Kishi et al. (1998-2003)	11	2600	1.67 – 2.66	A / C	0.1 – 0.6	37.0 – 80.0	f.p.b.t.
Kotynia (2005)	6	3000 – 4200	2.00	C	0.4 – 1.4	90.0 – 46.0	f.p.b.t. 3.p.b.t.
Kurihashi et al. (1999 – 2000)	8	1800 – 3400	1.66	A / C	0.2 – 0.6	34.9 – 77.5	f.p.b.t.
Leung (2004)	16	7200 – 1800	2.66	C	0.2 – 1.8	32.1 – 548.5	f.p.b.t.
Maleej and Leung (2005)	10	4800 - 1500	1.27	C	0.2 – 0.7	37.7 – 334.7	f.p.b.t.
Maeda et al. (2001)	2	1800	1.00	C	0.2 – 0.3	39.2 – 54.5	f.p.b.t.
M'Bazza et al. (1996)	1	3000	1.5	C	0.9	49.9	f.p.b.t.
Mikami et al. (1999)	1	3000	1.66	C	0.3	40.2	3.p.b.t.
Niu et al. (2006)	12	4200	0.21	C	1.2 – 2.6	54.2 – 133.8	f.p.b.t. 3.p.b.t.
Seim et al. (2001)	6	2030	0.25	G / C	1.1 -1.2	40.8 – 80.8	3.p.b.t.
Spadea et al. (1998)	2	4800	2.14	G / C	1.2	37.4 – 43.4	f.p.b.t.
Takahashi and Sato (2003)	5	1600	1.5	C	0.2 – 0.5	113.5 – 155.5	f.p.b.t.
Takeo et al. (1999)	6	2000	1.62	C	0.2	33.8 – 85.6	f.p.b.t. 3.p.b.t.
Teng and Yao (2007)	2	1500	1.67	G / C	1.67 – 2.01	71.3 – 82.0	f.p.b.t.
Wu et al. (1999-2000)	5	1800	1.33	C	0.1 – 0.3	65.0 – 78.4	f.p.b.t.
Yao et al. (2005)	10	1100	0.5	C	0.2 – 1.2	7.2 – 21.4	c.b.t. 3.p.b.t.
Zarniç et al. (1999)	2	2900	0.15 - 1.5	C	1.2	31.5 – 58.4	f.p.b.t.
Zhang et al. (2005)	20	1800 - 3500	1.66 - 2.66	A / C	0.2 – 0.6	34.0 – 78.1	f.p.b.t.

Table 2: Experimental tests collected in I-C induced debonding database. f.p.b.t. =four point bending test; 3.p.b.t. = three point bending test; c.b.t = cantilever beam test.

Model	Mechanism criteria	Parameter addressed					Limitations
		R.C.	S _l	S _t	FRP	Rs	
Oehlers	S.M.- E.F.	✓	✓		✓		$1.4-(d/200) \geq 1.1$ $M_{db,end}/M_u \leq 0.61$ $b/b_{frp} \leq 3$ $B_{mod} > 0$
Smith and Teng	S.M.	✓	✓		✓		
Teng and Yao	S.M.- E.F.	✓	✓	✓	✓		
Jansze et al.	S.M.	✓	✓				
Ahmed and van Germert	S.M.	✓	✓	✓	✓		
Colotti et al.	S.M.	✓	✓	✓	✓		
Raoof and Zhang	F.M.- E.F.	✓	✓		✓		
Wang and Ling	F.M.- E.F.	✓	✓		✓		
Raoof and Hassanen I	F.M.- E.F.	✓	✓		✓		
Raoof and Hassanen II	F.M.- E.F.	✓	✓		✓		
Ziraba I	S.M.- F.M.	✓	✓		✓	✓	$a/h < 3$ $a/h < 3$
Ziraba II	S.M.- F.M.	✓	✓		✓	✓	
Wu and Niu	F.M.	✓	✓		✓		
Casas and Pascual	F.M.	✓	✓		✓	✓	

Table 3: Summary of principal methods used to predict the end-debonding and intermediate crack induced debonding. S.M. = strength materials, F.M. = fracture mechanics. E.F. = experimental data fitting. R.C. = reinforced concrete. S_l = longitudinal reinforcement steel. S_t = Stirrups. FRP = fiber reinforced polymers. Rs = resin.

Model	Regression coefficient	Regression coefficient standard error	Regression standard error	R2	Number of observations	Bias	Standard deviation	COV	Probability distribution	R2	V _{max}	V _{min}
CW	END DEBONDING											
Smith and Teng	1.64	0.05	16.32	0.94	69	1.66	0.45	0.27	Lognormal	0.97	186.00	6.70
Teng and Yao ACI	1.34	0.06	22.74	0.87	90	1.45	0.50	0.34	Normal	0.99	186.00	6.70
Teng and Yao AS600	1.83	0.07	21.67	0.88	90	1.96	0.72	0.37	Lognormal	0.99	186.00	6.70
Teng and Yao BS	1.55	0.06	22.63	0.87	90	1.57	0.61	0.39	Lognormal	0.99	186.00	6.70
Jansze	0.55	0.03	24.41	0.85	83	0.65	0.32	0.49	Lognormal	0.97	186.00	6.70
Ahmed	0.53	0.02	23.60	0.86	83	0.65	0.30	0.46	Lognormal	0.95	186.00	6.70
Raof and Zhang	0.90	0.09	41.87	0.55	89	1.42	0.86	0.61	Lognormal	0.95	186.00	6.70
Wang	0.90	0.08	40.89	0.57	89	1.45	0.87	0.61	Lognormal	0.94	186.00	6.70
Raof and Hassanen I	1.05	0.08	36.72	0.66	89	1.57	1.03	0.66	Lognormal	0.97	186.00	6.70
Raof and Hassanen II	1.43	0.11	37.67	0.64	89	2.29	1.91	0.84	Lognormal	0.97	186.00	6.70
Colotti	0.99	0.02	12.01	0.97	55	0.90	0.21	0.24	Normal	0.96	186.00	6.70
Ziraba I	1.44	0.08	7.45	0.95	19	1.63	0.52	0.32	Normal	0.96	74.40	13.95
Ziraba II	0.89	0.06	8.46	0.93	19	1.17	0.21	0.18	Normal	0.95	74.40	13.95
Casas and Pascual	2.18	0.10	5.91	0.97	19	2.21	0.46	0.21	Normal	0.96	74.40	13.95
Casas and Pascual (with additional data)	1.65	0.06	20.92	0.89	90	1.89	0.57	0.37	Normal	0.96	186.00	6.70
CP	END DEBONDING											
Smith and Teng	1.47	0.06	13.57	0.93	39	1.62	0.44	0.27	Lognormal	0.91	80.10	12.50
Teng and Yao ACI	1.15	0.05	16.09	0.89	59	1.41	0.54	0.38	Normal	0.97	80.10	12.50
Teng and Yao AS600	1.60	0.07	15.30	0.90	59	1.84	0.65	0.35	Lognormal	0.98	80.10	12.50
Teng and Yao BS	1.31	0.06	15.57	0.90	59	1.44	0.52	0.36	Lognormal	0.98	80.10	12.50
Jansze	0.66	0.03	16.02	0.89	50	0.64	0.28	0.44	Lognormal	0.95	80.10	12.50
Ahmed	0.67	0.04	18.89	0.87	50	0.82	0.56	0.69	Lognormal	0.95	80.10	12.50
Raof and Zhang	1.31	0.09	22.96	0.78	59	1.68	1.19	0.71	Lognormal	0.96	80.10	12.50
Wang and Ling	1.29	0.08	21.79	0.80	59	1.61	1.17	0.76	Lognormal	0.95	80.10	12.50
Raof and Hassanen I	1.36	0.09	22.28	0.79	59	2.22	1.96	0.88	Lognormal	0.92	80.10	12.50
Raof and Hassanen II	1.79	0.12	22.67	0.78	59	3.14	3.27	1.04	Lognormal	0.92	80.10	12.50
Colotti	1.02	0.06	13.65	0.90	34	1.09	0.44	0.40	Normal	0.91	79.50	12.50
Ziraba I	1.06	0.10	23.51	0.77	38	2.09	1.32	0.63	Normal	0.95	80.10	12.50

Ziraba II	0.77	0.05	17.94	0.86	38	1.26	0.52	0.41	Normal	0.91	80.10	12.50
Casas and Pascual	2.14	0.07	8.94	0.97	38	2.27	0.54	0.24	Normal	0.97	80.10	12.50
Casas and Pascual (with additional data)	1.99	0.06	10.94	0.95	59	2.13	0.54	0.24	Lognormal	0.95	80.10	12.50
CW + CP	INTERMEDIATE DEBONDING											
Casas IC	1.76	0.05	2.78	0.98	20	1.88	0.35	0.18	Lognormal	0.96	34.90	8.51
Casas IC + CasasD	1.08	0.05	8.34	0.96	22	1.80	0.42	0.23	Normal	0.97	135.45	8.51
Casas IC + CasasD (all data)	1.48	0.03	31.95	0.92	188	2.23	1.45	0.65	Lognormal	0.96	548.5	6.90
Wu and Niu	0.57	0.03	3.82	0.96	15	0.60	0.19	0.32	Normal	0.93	34.90	10.00
Wu and Niu (all data)	0.37	0.02	39.59	0.86	78	0.50	0.23	0.46	Lognormal	0.98	548.5	10.00
ACI440.2R Case 1	0.70	0.03	3.63	0.97	13	0.73	0.22	0.29	Lognormal	0.88	34.90	10
ACI440.2R Case 2	0.91	0.03	36.73	0.94	68	1.22	0.56	0.45	Normal	0.94	548.5	10
ACI440.2R (all data, case 3)	0.77	0.02	34.08	0.90	183	0.87	0.48	0.55	Lognormal	0.97	548.5	6.9

Table 4: Statistical results of existing models compared to experimental database

Appendix A: Database of beams failed by end debonding

	Beam	b	h	d	d'	a	B	L	f _c '	E _s	f _{ys}	A _s	Reo	E' _s	f _y	A' _s
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[GPa]	[MPa]	[mm ²]	[tension]	[GPa]	[MPa]	[mm ²]
Ahmed et al. (1999)	AF3	125	225	193	32	100	500	1500	46.0	185	568	101	2-8	195	553	57
	CF2-1	125	225	193	32	100	500	1500	46.0	185	568	129	2-8,1-6	195	553	57
	CF3-1	125	225	193	32	100	500	1500	46.0	185	568	151	3-8	195	553	57
	CF4-1	125	225	193	32	100	500	1500	46.0	183	586	207	2-10,1-8	195	553	57
	DF.2	125	225	193	32	50	500	1500	46.0	185	568	151	3-8	195	553	57
	DF.3	125	225	193	32	50	500	1500	46.0	185	568	151	3-8	195	553	57
	DF.4	125	225	193	32	50	500	1500	46.0	185	568	151	3-8	195	553	57
Ahmed et al. (2000)	AF.2	125	225	196	25	200	500	-	41.0	200	568	101	-8	200	568	57
	AF.2-1	125	225	196	25	150	500	-	41.0	200	568	101	-8	200	568	57
	AF.4	125	225	196	25	50	500	-	41.0	200	568	101	-8	200	568	57
	DF.1	125	225	196	25	50	500	-	42.0	200	568	151	-8	200	568	57
	BF.2-1	125	225	196	25	50	500	-	41.0	200	568	101	-8	200	568	57
	BF.3-1	125	225	196	25	50	500	-	41.0	200	568	101	-8	200	568	57
	EF.1-1	125	225	196	25	50	500	-	46.0	200	568	151	-8	200	568	57
	EF.3-1	125	225	196	25	50	500	-	38.0	200	568	151	-8	200	568	57
	EF.4-1	125	225	196	25	50	500	-	33.0	200	568	151	-8	200	568	57
FF.2-3	125	225	196	25	70	700	-	39.5	200	568	151	-8	200	568	57	
Arduini et al. (1997)	A4	200	200	163	30	150	700	-	33.0	200	540	308	-14	200	540	308
	A5	200	200	163	30	150	700	-	33.0	200	540	308	-14	200	540	308
	B2	300	400	349.5	44	100	1100	-	30.0	200	340	398	-13	200	340	266
	B3	300	400	349.5	44	100	1100	-	30.0	200	340	398	-13	200	340	266
Beber et al. (1999)	VR5	120	250	214	34	75	783	2350	33.6	200	565	157	2-10	200	738	57
	VR6	120	250	214	34	75	783	2350	33.6	200	565	157	2-10	200	738	57
	VR7	120	250	214	34	75	783	2350	33.6	200	565	157	2-10	200	738	57
	VR8	120	250	214	34	75	783	2350	33.6	200	565	157	2-10	200	738	57
	VR9	120	250	214	34	75	783	2350	33.6	200	565	157	2-10	200	738	57
	VR10	120	250	214	34	75	783	2350	33.6	200	565	157	2-10	200	738	57
Benjeddou et al. (2007)	RB1	120	150	120	30	50	600	1800	21.0	200	400	157	2-10	200	400	157
	RB2	120	150	120	30	50	600	1800	21.0	200	400	157	2-10	200	400	157
	RB3	120	150	120	30	50	600	1800	21.0	200	400	157	2-10	200	400	157
	RB4	120	150	120	30	50	600	1800	21.0	200	400	157	2-10	200	400	157
	RB5	120	150	120	30	50	600	1800	21.0	200	400	157	2-10	200	400	157
	RB6	120	150	120	30	50	600	1800	38.0	200	400	157	2-10	200	400	157

Table A. 1: Database of beams failed by end debonding. Ahmed et al. (cited in [6]), Ahmed et al. (cited in [19]), Arduini et al. (cited in [19]), Beber et al. (cited in [6]), Benjeddou et al. [32].

	Beam	E_{yv} [GPa]	f_{yv} [MPa]	A_{sv} [mm ²]	s [mm]	E_a [MPa]	t_a [mm]	Type	E_{fip} [GPa]	f_{fip} [MPa]	t_{fip} [mm]	b_{fip} [mm]	V_{exp} [kN]
Ahmed et al. (1999)	AF3	195	553	57	71	7200	-	C-W	240.00	3500	0.334	75	48.30
	CF2-1	195	553	57	71	7200	-	C-W	240.00	3500	0.334	75	52.40
	CF3-1	195	553	57	71	7200	-	C-W	240.00	3500	0.334	75	59.10
	CF4-1	195	553	57	71	7200	-	C-W	240.00	3500	0.334	75	70.10
	DF.2	195	553	57	100	7200	-	C-W	240.00	3500	0.334	75	60.30
	DF.3	195	553	57	100	7200	-	C-W	240.00	3500	0.501	75	60.00
	DF.4	195	553	57	100	7200	-	C-W	240.00	3500	0.668	75	62.80
Ahmed et al. (2000)	AF.2	200	553	57	71	-	-	C-W	240.00	3500	0.3	75	41.50
	AF.2-1	200	553	57	71	-	-	C-W	240.00	3500	0.3	75	42.90
	AF.4	200	553	57	71	-	-	C-W	240.00	3500	0.3	75	55.50
	DF.1	200	553	57	100	-	-	C-W	240.00	3500	0.2	75	59.00
	BF.2-1	200	553	57	167	-	-	C-W	240.00	3500	0.3	75	45.00
	BF.3-1	200	553	57	100	-	-	C-W	240.00	3500	0.3	75	52.00
	EF.1-1	200	553	57	100	-	-	C-W	240.00	3500	0.3	75	65.90
	EF.3-1	200	553	57	100	-	-	C-W	240.00	3500	0.3	75	59.50
	EF.4-1	200	553	57	100	-	-	C-W	240.00	3500	0.3	75	60.30
	FF.2-3	200	553	57	100	-	-	C-W	240.00	3500	0.5	75	53.00
Arduini et al. (1997)	A4	200	540	57	150	-	-	C-W	167.00	2906	1.3	150	55.00
	A5	200	540	57	150	-	-	C-W	167.00	2906	2.6	150	45.00
	B2	200	340	101	100	-	-	C-W	400.00	3000	0.2	300	85.00
	B3	200	340	101	100	-	-	C-W	400.00	3000	0.5	300	114.00
Beber et al. (1999)	VR5	200	738	57	110	8500	-	C-P	230.00	3400	0.22	120	51.10
	VR6	200	738	57	110	8500	-	C-P	230.00	3400	0.22	120	50.30
	VR7	200	738	57	110	8500	-	C-P	230.00	3400	0.22	120	62.10
	VR8	200	738	57	110	8500	-	C-P	230.00	3400	0.22	120	62.00
	VR9	200	738	57	110	8500	-	C-P	230.00	3400	0.22	120	64.80
	VR10	200	738	57	110	8500	-	C-P	230.00	3400	0.22	120	68.50
Benjeddou et al. (2007)	RB1	200	235	28	10	12800	-	C-P	165.00	2800	1.2	100	20.06
	RB2	200	235	28	10	12800	-	C-P	165.00	2800	1.2	100	18.83
	RB3	200	235	28	10	12800	-	C-P	165.00	2800	1.2	100	16.05
	RB4	200	235	28	10	12800	-	C-P	165.00	2800	1.2	100	15.38
	RB5	200	235	28	10	12800	-	C-P	165.00	2800	1.2	50	15.05
	RB6	200	235	28	10	12800	-	C-P	165.00	2800	1.2	100	18.69

Table A. 1 (cont): Database of beams failed by end debonding. Ahmed et al. (cited in [6]), Ahmed et al. (cited in [19]), Arduini et al. (cited in [19]), Beber et al. (cited in [6]), Benjeddou et al. (cited in [32]).

	Beam	b	h	d	d'	a	B	L	f _c '	E _s	f _{ys}	A _s	Reo	E' _s	f _y	A' _s
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[GPa]	[MPa]	[mm ²]	[tension]	[GPa]	[MPa]	[mm ²]
Breña et al. (2004)	A1-I	102	102	89	13	25	330	812	42.2	200	435	71	1-9,5	200	435	71
	A1-II	102	102	89	13	25	330	812	42.2	200	435	142	2-9,5	200	435	142
	A2-I	102	102	89	13	25	330	812	42.2	200	435	71	1-9,5	200	435	71
	A2-II	102	102	89	13	25	330	812	42.2	200	435	142	2-9,5	200	435	142
	A3-I	102	102	89	13	25	330	812	53.3	200	435	71	1-9,5	200	435	71
	A3-II	102	102	89	13	25	330	812	53.3	200	435	142	2-9,5	200	435	142
	A4-I	102	102	89	13	25	330	812	53.3	200	435	71	1-9,5	200	435	71
	A4-II	102	102	89	13	25	330	812	53.3	200	435	142	2-9,5	200	435	142
	A5-I	102	102	89	13	25	330	812	53.3	200	435	71	1-9,5	200	435	71
	A5-II	102	102	89	13	25	330	812	53.3	200	435	142	2-9,5	200	435	142
	A6-I	102	102	89	13	25	330	812	47.7	200	435	71	1-9,5	200	435	71
David et al. (1999)	P ₂	150	300	257	-	200	933	2800	40.0	200	500	308	2-14	-	-	-
	P ₃	150	300	257	-	200	933	2800	40.0	200	500	308	2-14	-	-	-
	P ₄	150	300	257	-	200	933	2800	40.0	200	500	308	2-14	-	-	-
	P ₅	150	300	257	-	200	933	2800	40.0	200	500	308	2-14	-	-	-
Esfahani et al. (2007)	B3-12D-2L15	150	200	166	25	100	600	1600	25.2	200	400	226	2-12	200	365	157
	B3-12D-3L15	150	200	166	25	100	600	1600	25.2	200	400	226	2-12	200	365	157
Fanning et al. (2000)	F5	155	240	209	31	385	1100	2800	66.4	204	460	339	3-12	204	460	226
	F6	155	240	209	31	385	1100	2800	66.4	204	460	339	3-12	204	460	226
	F7	155	240	209	31	462	1100	2800	66.4	204	460	339	3-12	204	460	226
	F8	155	240	209	31	462	1100	2800	66.4	204	460	339	3-12	204	460	226
	F9	155	240	209	31	550	1100	2800	66.4	204	460	339	3-12	204	460	226
	F10	155	240	209	31	550	1100	2800	66.4	204	460	339	3-12	204	460	226
Garden et al. (1997)	1A _u	100	100	84	16	20	300	900	47.3	215	350	85	3-6	215	350	57
	2A _u	100	100	84	16	20	340	900	47.3	215	350	85	3-6	215	350	57
	3A _u	100	100	84	16	20	400	900	47.3	215	350	85	3-6	215	350	57
	1B _u	100	100	84	16	20	300	900	47.3	215	350	85	3-6	215	350	57
	2B _u	100	100	84	16	20	340	900	47.3	215	350	85	3-6	215	350	57
	3B _u	100	100	84	16	20	400	900	47.3	215	350	85	3-6	215	350	57
	1B _{2u}	100	100	84	16	20	300	900	47.3	215	350	85	3-6	215	350	57

Table A. 2: Database of beams failed by end debonding. Breña et al. [33], David et al. (cited in [6]), Esfahani et al. [34], Fanning et al. [35], Garden et al. (cited in [6]).

	Beam	E_{yv} [GPa]	f_{yv} [MPa]	A_{sv} [mm ²]	s [mm]	E_a [MPa]	t_a [mm]	Type	E_{fip} [GPa]	f_{fip} [MPa]	t_{fip} [mm]	b_{fip} [mm]	V_{exp} [kN]
Breña et al. (2004)	A1-I	200	420	28	51	3035	0.4	C-W	230.00	3790	0.165	51	13.95
	A1-II	200	420	28	51	3035	0.4	C-W	230.00	3790	0.165	51	20.10
	A2-I	200	420	28	51	3035	0.8	C-W	230.00	3790	0.33	51	15.70
	A2-II	200	420	28	51	3035	0.8	C-W	230.00	3790	0.33	51	22.25
	A3-I	200	420	28	51	3035	1.2	C-W	230.00	3790	0.495	51	19.50
	A3-II	200	420	28	51	3035	1.2	C-W	230.00	3790	0.495	51	24.00
	A4-I	200	420	28	51	3035	0.8	C-W	230.00	3790	0.33	76	18.40
	A4-II	200	420	28	51	3035	0.8	C-W	230.00	3790	0.33	76	26.25
	A5-I	200	420	28	51	3035	0.4	C-W	230.00	3790	0.165	102	17.60
	A5-II	200	420	28	51	3035	0.4	C-W	230.00	3790	0.165	102	24.45
	A6-I	200	420	28	51	4480	1.6	C-P	155.00	2400	1.19	51	17.40
David et al. (1999)	P ₂	200	500	57	140	8500	1.0	C-P	150.00	2400	1.2	100	68.00
	P ₃	200	500	57	140	8500	1.0	C-P	150.00	2400	1.2	100	71.10
	P ₄	200	500	57	140	8500	1.0	C-P	150.00	2400	2.4	100	78.00
	P ₅	200	500	57	140	8500	1.0	C-P	150.00	2400	2.4	100	79.50
Esfahani et al. (2007)	B3-12D-2L15	200	350	50	80	-	-	C-W	237.00	2845	0.176	150	35.47
	B3-12D-3L15	200	350	50	80	-	-	C-W	237.00	2845	0.176	150	37.22
Fanning et al. (2000)	F5	198	250	28	125	-	-	C-P	155.00	2400	1.2	120	50.00
	F6	198	250	28	125	-	-	C-P	155.00	2400	1.2	120	51.50
	F7	198	250	28	125	-	-	C-P	155.00	2400	1.2	120	48.75
	F8	198	250	28	125	-	-	C-P	155.00	2400	1.2	120	32.00
	F9	198	250	28	125	-	-	C-P	155.00	2400	1.2	120	31.00
	F10	198	250	28	125	-	-	C-P	155.00	2400	1.2	120	41.00
Garden et al. (1997)	1A _u	215	350	14	51	11560	2.0	C-P	111.00	1273	0.5	90	19.80
	2A _u	215	350	14	51	11560	2.0	C-P	111.00	1273	0.5	90	19.30
	3A _u	215	350	14	51	11560	2.0	C-P	111.00	1273	0.5	90	19.50
	1B _u	215	350	14	51	11560	2.0	C-P	111.00	1273	0.7	65	18.30
	2B _u	215	350	14	51	11560	2.0	C-P	111.00	1273	0.7	65	17.00
	3B _u	215	350	14	51	11560	2.0	C-P	111.00	1273	0.7	65	17.30
	1B _{2u}	215	350	14	51	11560	2.0	C-P	111.00	1273	0.7	65	18.20

Table A. 2 (cont): Database of beams failed by end debonding. Breña et al. [33], David et al. (cited in [6]), Esfahani et al. [34], Fanning et al. [35], Garden et al. (cited in [6]).

	Beam	b	h	d	d'	a	B	L	f _c '	E _s	f _{ys}	A _s	Reo	E' _s	f' _y	A' _s
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[GPa]	[MPa]	[mm ²]	[tension]	[GPa]	[MPa]	[mm ²]
Garden et al. (1997)	1C _u	100	100	84	16	20	300	900	47.3	215	350	85	3-6	215	350	57
	2C _u	100	100	84	16	20	340	900	47.3	215	350	85	3-6	215	350	57
	3C _u	100	100	84	16	20	400	900	47.3	215	350	85	3-6	215	350	57
Garden et al. (1998)	B1u,1.0	100	100	84	16	20	300	900	43.2	215	350	85	3-6	215	350	57
	B2u,1.0	100	100	84	16	20	300	900	43.2	215	350	85	3-6	215	350	57
	B1u,2.3	130	230	206	25	20	844	2200	37.6	220	556	236	3-10	220	556	101
	B3U1.0	100	100	87	10	20	340	-	44.8	215	350	85	-6	215	350	57
	B4U1.0	100	100	87	10	20	400	-	44.8	215	350	85	-6	215	350	57
	B5U1.0	100	100	87	10	20	400	-	44.8	215	350	85	-6	215	350	57
	B1U4.5	145	230	214	15	40	1525	-	39.0	220	556	236	-12	220	556	111
Grace et al. (2005)	Bb1	152	254	228.6	25.4	152.5	864	2540	31.0	200	414	397	3-15,9	200	414	142
	Bb2	152	254	228.6	25.4	152.5	864	2540	31.0	200	414	397	3-15,9	200	414	142
	Bb3	152	254	228.6	25.4	152.5	864	2540	31.0	200	414	397	3-15,9	200	414	142
	Bb4	152	254	228.6	25.4	152.5	864	2540	31.0	200	414	397	3-15,9	200	414	142
Hau et al. (1999)	2	150	250	205	45	350	500	1500	35.4	231	537	157	2-10	231	537	157
	4	150	250	205	45	200	500	1500	36.2	231	537	157	2-10	231	537	157
	5	150	250	205	45	50	500	1500	40.6	231	537	157	2-10	231	537	157
	6	150	250	205	45	200	500	1500	39.9	231	537	157	2-10	231	537	157
	7	150	250	205	45	350	500	1500	37.6	231	537	157	2-10	231	537	157
Juvandes et al. (1998)	B7	75	150	131	22	10	650	1500	37.0	200	190	14	2-3	200	470	151
	B.11	75	150	128.5	20	200	650	-	36.0	200	190	14	-3	200	190	151
Matthys (2000)	BF2	200	450	409	-	70	1250	-	36.5	200	590	804	-16	200	590	-
	BF3	200	450	409	-	70	1250	-	34.9	200	590	804	-16	200	590	-
	BF4	200	450	409	-	70	1250	-	30.8	200	590	804	-16	200	590	-
	BF5	200	450	409	-	70	1250	-	37.4	200	590	804	-16	200	590	-
	BF8	200	450	409	-	71	1251	-	39.4	200	590	402	-16	200	590	-
	BF9	200	450	409	-	71	1251	-	33.7	200	590	402	-16	200	590	-
Nguyen et al. (2001)	A950	120	150	120	34	190	440	1330	25.7	200	384	236	3-10	200	400	57
	A1100	120	150	120	34	115	440	1330	25.7	200	384	236	3-10	200	400	57
	A1150	120	150	120	34	90	440	1330	25.7	200	384	236	3-10	200	400	57
	B2	120	150	120	34	115	440	1330	25.7	200	384	236	3-10	200	400	57
Pham et al. (2006)	E1a	140	260	220	40	150	700	2300	53.7	205	551	339	3-12	205	551	226

Table A. 3: Database of beams failed by end debonding. Garden et al. (cited in [6]), Garden et al. [36], Grace et al. [37], Hau et al. (cited in [6]), Juvandes et al. (cited in [22]), Matthys [38], Nguyen et al. [39], Pham et al. [40].

	Beam	E_{yv} [GPa]	f_{yv} [MPa]	A_{sv} [mm ²]	s [mm]	E_a [MPa]	t_a [mm]	Type	E_{fip} [GPa]	f_{fip} [MPa]	t_{fip} [mm]	b_{fip} [mm]	V_{exp} [kN]
Garden et al. (1997)	1C _u	215	350	14	51	11560	2.0	C-P	111.00	1273	1	45	16.00
	2C _u	215	350	14	51	11560	2.0	C-P	111.00	1273	1	45	17.80
	3C _u	215	350	14	51	11560	2.0	C-P	111.00	1273	1.0	45	15.40
Garden et al. (1998)	B1u,1.0	215	350	14	51	8600	2.0	C-P	111.00	1414	0.82	67	18.30
	B2u,1.0	215	350	14	51	8600	2.0	C-P	111.00	1414	0.82	67	16.00
	B1u,2.3	215	350	57	150	8600	2.0	C-P	115.00	1284	1.28	90	50.20
	B3U1.0	215	350	14	50	-	-	C-W	111.00	1414	0.8	67	17.00
	B4U1.0	215	350	14	50	-	-	C-W	111.00	1414	0.8	67	17.30
	B5U1.0	215	350	14	50	-	-	C-W	111.00	1414	0.8	67	17.30
	B1U4.5	220	350	57	150	-	-	C-W	115.00	1284	1.3	90	30.00
Grace et al. (2005)	Bb1	200	414	142	102	2140	-	C-W	138.00	2070	1.2	152	68.30
	Bb2	200	414	142	102	2140	-	C-W	138.00	2070	1.2	152	68.30
	Bb3	200	414	142	102	2140	-	C-P	227.00	2758	0.4	152	66.75
	Bb4	200	414	142	102	2140	-	C-P	227.00	2758	0.4	152	66.75
Hau et al. (1999)	2	231	537	157	100	3260	0.4	G-W	19.72	259	1.32	150	53.00
	4	231	537	157	100	3260	0.4	G-W	19.72	259	1.3	150	65.40
	5	231	537	157	100	3260	0.4	G-W	19.72	259	2.6	150	79.40
	6	231	537	157	100	3260	0.4	G-W	19.72	259	1.32	150	63.10
	7	231	537	157	100	3260	0.4	G-W	19.72	259	1.32	150	53.90
Juvandes et al. (1998)	B7	200	190	14	60	10250	2.5	C-P	150.00	2400	1.2	50	12.50
	B.11	200	190	14	60	-	-	C-W	150.00	2400	1.2	50	6.70
Matthys (2000)	BF2	200	560	101	100	-	-	C-W	159.00	3200	1.2	100	185.00
	BF3	200	560	101	100	-	-	C-W	159.00	3200	1.2	100	186.00
	BF4	200	560	101	100	-	-	C-W	159.00	3200	1.2	100	184.20
	BF5	200	560	101	100	-	-	C-W	159.00	3200	1.2	100	177.00
	BF8	200	560	101	100	-	-	C-W	159.00	3200	1.2	100	111.30
	BF9	200	560	101	100	-	-	C-W	159.00	3200	0.2	100	95.80
Nguyen et al. (2001)	A950	200	400	57	50	12800	1.5	C-P	181.00	3140	1.2	80	28.10
	A1100	200	400	57	50	12800	1.5	C-P	181.00	3140	1.2	80	28.70
	A1150	200	400	57	50	12800	1.5	C-P	181.00	3140	1.2	80	29.50
	B2	200	400	57	50	12800	1.5	C-P	181.00	3140	1.2	80	65.10
Pham et al. (2006)	E1a	204	334	79	125	3500	-	C-W	209.00	3900	1.056	100	35.35

Table A.3 (cont): Database of beams failed by end debonding. Garden et al. (cited in [6]), Garden et al. [36], Grace et al. [37], Hau et al. (cited in [6]), Juvandes et al. (cited in [22]), Matthys [38], Nguyen et al. [39], Pham et al. [40].

	Beam	b	h	d	d'	a	B	L	f _c '	E _s	f _{ys}	A _s	Reo	E' _s	f' _y	A' _s
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[GPa]	[MPa]	[mm ²]	[tension]	[GPa]	[MPa]	[mm ²]
Pham et al. (2006)	E1b	140	260	220	40	150	700	2300	53.7	205	551	339	3-12	205	551	226
	E2a	140	260	220	40	350	700	2300	53.7	205	551	339	3-12	205	551	226
	E2b	140	260	220	40	350	700	2300	53.7	205	551	339	3-12	205	551	226
	E3a	140	260	220	40	150	700	2300	53.7	205	551	339	3-12	205	551	226
	E3b	140	260	220	40	150	700	2300	53.7	205	551	339	3-12	205	551	226
	E4a	140	260	220	40	150	700	2300	53.7	205	551	339	3-12	205	551	226
	E4b	140	260	220	40	150	700	2300	53.7	205	551	339	3-12	205	551	226
	E5a	140	260	220	40	150	700	2300	53.7	205	551	339	3-12	205	551	226
	E5b	140	260	220	40	150	700	2300	53.7	205	551	339	3-12	205	551	226
	E3b2	140	260	220	40	150	700	2300	53.7	205	551	226	2-12	205	551	226
Quantrill et al. (1996)	B2	100	100	84	16	20	300	900	42.4	215	350	85	3-6	215	350	57
	B3	100	100	84	16	20	300	900	42.4	215	350	85	3-6	210	350	57
	B4	100	100	84	16	20	300	900	42.4	215	350	85	3-6	215	350	57
	B6	100	100	84	16	20	300	900	42.4	215	350	85	3-6	215	350	57
Rahimi et al. (2001)	A4	200	150	120	30	85	750	2100	41.5	210	575	157	2-10	210	575	101
	A5	200	150	120	30	85	750	2100	41.5	210	575	157	2-10	210	575	101
	A6	200	150	120	30	85	750	2100	41.5	210	575	157	2-10	210	575	101
	A7	200	150	120	30	85	750	2100	41.5	210	575	157	2-10	210	575	101
	A8	200	150	120	30	85	750	2100	41.5	210	575	157	2-10	210	575	101
	A9	200	150	120	30	85	750	2100	41.5	210	575	157	2-10	210	575	101
	A10	200	150	120	30	85	750	2100	41.5	210	575	157	2-10	210	575	101
	A11	200	150	120	30	85	750	2100	41.5	210	575	157	2-10	210	575	101
Ritchie et al (1991)	C	152	305	251	-	203	914	2438	39.8	200	414	253	2-12.7	-	-	-
	D	152	305	251	-	203	914	2438	39.8	200	414	253	2-12.7	-	-	-
	G	152	305	251	-	0	914	2438	43.0	200	414	253	2-12.7	-	-	-
	I	152	305	251	-	203	914	2438	39.8	200	414	253	2-12.7	-	-	-
	M	152	305	251	-	0	914	2438	43.0	200	414	253	2-12.7	-	-	-
Ross et al. (1999)	1B	200	200	152	48	1	914	2742	54.8	200	410	143	2-9,5	200	410	143
	1C	200	200	152	48	1	914	2742	54.8	200	410	143	2-9,5	200	410	143
	2B	200	200	152	48	1	914	2742	54.8	200	410	253	2-12,7	200	410	143
	2C	200	200	152	48	1	914	2742	54.8	200	410	253	2-12,7	200	410	143
	2D	200	200	152	48	1	914	2742	54.8	200	410	253	2-12,7	200	410	143

Table A. 4: Database of beams failed by end debonding. Pham et al. [40], Quantrill et al. (cited in [6]), Rahimi et al. [10], Ritchie et al. [41], Ross et al. [9].

	Beam	E_{yv} [GPa]	f_{yv} [MPa]	A_{sv} [mm ²]	s [mm]	E_a [MPa]	t_a [mm]	Type [-]	E_{fip} [GPa]	f_{fip} [MPa]	t_{fip} [mm]	b_{fip} [mm]	V_{exp} [kN]
Pham et al. (2006)	E1b	204	334	79	125	3500	-	C-W	209.00	3900	1.056	100	37.30
	E2a	204	334	79	125	3500	-	C-W	209.00	3900	1.056	100	25.70
	E2b	204	334	79	125	3500	-	C-W	209.00	3900	1.056	100	26.70
	E3a	204	334	79	125	3500	-	C-W	209.00	3900	1.056	100	33.00
	E3b	204	334	79	125	3500	-	C-W	209.00	3900	1.056	100	32.60
	E4a	204	334	79	125	3500	-	C-W	209.00	3900	1.056	100	39.50
	E4b	204	334	79	125	3500	-	C-W	209.00	3900	1.056	100	30.60
	E5a	204	334	79	125	3500	-	C-W	209.00	3900	1.584	100	31.65
	E5b	204	334	79	125	3500	-	C-W	209.00	3900	1.584	100	31.60
	E3b2	204	334	79	125	3500	-	C-W	209.00	3900	1.056	100	30.00
Quantrill et al. (1996)	B2	215	350	14	50	11560	2.0	G-P	49.00	1078	1.2	80	17.00
	B3	215	350	14	50	11560	2.0	G-P	49.00	1078	1.2	30	12.30
	B4	215	350	14	50	11560	2.0	G-P	49.00	1078	1.6	60	17.50
	B6	215	350	14	50	11560	2.0	C-P	118.50	987	1.2	80	20.40
Rahimi et al. (2001)	A4	210	575	28	150	7000	2.0	C-W	127.00	1532	0.8	150	30.95
	A5	210	575	28	150	7000	2.0	C-W	127.00	1532	0.8	150	31.60
	A6	210	575	28	150	7000	2.0	C-W	127.00	1532	1.2	150	29.70
	A7	210	575	28	150	7000	2.0	C-W	127.00	1532	1.2	150	35.30
	A8	210	575	28	150	7000	2.0	C-W	127.00	1532	0.8	150	32.60
	A9	210	575	28	150	7000	2.0	C-W	127.00	1532	0.8	150	31.95
	A10	210	575	28	150	7000	2.0	C-W	127.00	1532	0.8	150	33.75
	A11	210	575	28	150	7000	2.0	C-W	127.00	1532	0.8	150	34.70
Ritchie et al (1991)	C	200	414	99	102	8500	2.0	G-P	11.72	161	4.76	152	55.40
	D	200	414	99	102	8500	2.0	G-P	11.72	161	4.76	151	59.60
	G	200	414	99	102	8500	2.0	G-P	10.34	184	4.19	152	62.90
	I	200	414	99	102	8500	2.0	C/G-P	27.58	319	4.06	150	50.60
	M	200	414	99	102	8500	2.0	C-P	117.91	1489	1.27	152	72.10
Ross et al. (1999)	1B	200	410	143	102	8500	2.0	C-P	138.00	2206	0.5	200	40.10
	1C	200	410	143	102	8500	2.0	C-P	138.00	2206	0.5	200	35.60
	2B	200	410	143	102	8500	2.0	C-P	138.00	2206	0.5	200	49.00
	2C	200	410	143	102	8500	2.0	C-P	138.00	2206	0.5	200	35.60
	2D	200	410	143	102	8500	2.0	C-P	138.00	2206	0.5	200	40.10

Table A. 4 (cont): Database of beams failed by end debonding. Pham et al. [40], Quantrill et al. (cited in [6]), Rahimi et al. [10], Ritchie et al. [41], Ross et al. [9].

	Beam	b	h	d	d'	a	B	L	f _c '	E _s	f _{ys}	A _s	Reo	E' _s	f' _y	A' _s
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[GPa]	[MPa]	[mm ²]	[tension]	[GPa]	[MPa]	[mm ²]
Ross et al. (1999)	3B	200	200	152	48	1	914	2742	54.8	200	410	396	2-15,9	200	410	143
	4B	200	200	150.5	40	0	914	-	54.8	200	410	567	-19	200	410	142
	5B	200	200	149	40	0	914	-	54.8	200	410	774	-22	200	410	142
	6B	200	200	147	40	0	914	-	54.8	200	410	1019	-26	200	410	142
	3C	200	200	152	48	1	914	2742	54.8	200	410	396	2-15,9	200	410	143
	3D	200	200	152	48	1	914	2742	54.8	200	410	396	2-15,9	200	410	143
Saadatmanesh et al. (1991)	B	205	455	400	55	155	1983	4575	35.0	200	456	1013	2-25.4	200	456	253
Sharif et al. (1994)	P2	150	150	114	36	75	393	1180	37.7	200	450	157	2-10	200	450	57
	P3	150	150	114	36	75	393	1180	37.7	200	450	157	2-10	200	450	57
Spadea et al. (1998)	A1.1	140	300	262	30	50	1800	-	24.9	200	435	402	-16	200	435	402
	A3.1	140	300	262	30	50	1800	-	24.9	200	435	402	-16	200	435	402
Tan et al. (1999)	A00	100	150	124	21	-	600	-	28.6	200	500	157	-10	200	500	57
	A15	100	150	124	22	-	600	-	31.4	200	500	157	-11	200	500	57
	A25	100	150	124	23	-	600	-	29.7	200	500	157	-12	200	500	57
	A40	100	150	124	24	-	600	-	31.4	200	500	157	-13	200	500	57
	A60	100	150	124	25	-	600	-	28.6	200	500	157	-14	200	500	57
	A75	100	150	124	26	-	600	-	28.5	200	500	157	-15	200	500	57
	A90	100	150	124	27	-	600	-	30.1	200	500	157	-16	200	500	57
Täljsten et al. (1997)	SB1	200	300	252	48	150	1300	3600	51.2	200	527	402	2-16	200	527	402
	SB2	200	300	252	48	200	1300	3600	52.0	200	527	402	2-16	200	527	402
	SB3	200	300	252	48	300	1300	3600	52.0	200	527	402	2-16	200	527	402
	MB1	200	300	252	48	150	1300	3600	56.0	200	527	402	2-16	200	527	402
	HB1	200	300	252	48	150	1300	3600	56.0	200	527	402	2-16	200	527	402
	FB1	200	300	252	48	150	1300	3600	51.2	200	527	402	2-16	200	527	402
Teng and Yao (2007)	CS	150.2	252.7	221.7	31	50	500	1500	25.5	199	536	157	2-10	199	536	157
	CS-L3	151.1	253	222	31	50	500	1500	27.3	199	536	157	2-10	199	536	157
	CS-W100	150.6	254	218.5	35.5	50	500	1500	31.4	199	536	157	2-10	199	536	157
	CP	151.1	252.8	222.8	30	50	500	1500	30.7	199	536	157	2-10	199	536	157
	CS-C10	151.1	252.7	240.7	12	50	500	1500	22.7	199	536	157	2-10	199	536	157
Triantafillou et al. (1992)	4	76	127	111	16	75	305	1220	44.7	200	517	33	2-4.6	-	-	-
	5	76	127	111	16	75	305	1220	44.7	200	517	33	2-4.6	-	-	-
	6	76	127	111	16	75	305	1220	44.7	200	517	33	2-4.6	-	-	-

Table A. 5: Database of beams failed by end debonding. Ross et al. [9], Saadamanesh et al. [42], Sharif et al. [43], Spadea et al. [44], Tan et al. (cited in [22]), Täljsten et al. [45], Teng and Yao [23], Triantafillou et al. [16].

	Beam	E_{yv} [GPa]	f_{yv} [MPa]	A_{sv} [mm ²]	s [mm]	E_a [MPa]	t_a [mm]	Type	E_{fip} [GPa]	f_{fip} [MPa]	t_{fip} [mm]	b_{fip} [mm]	V_{exp} [kN]
Ross et al. (1999)	3B	200	410	143	102	8500	2.0	C-P	138.00	2206	0.5	200	54.50
	4B	200	410	142	102	-	-	C-W	138.00	2206	0.5	200	53.80
	5B	200	410	142	102	-	-	C-W	138.00	2206	0.5	200	73.40
	6B	200	410	142	102	-	-	C-W	138.00	2206	0.5	200	84.50
	3C	200	410	143	102	8500	2.0	C-P	138.00	2206	0.5	200	54.10
	3D	200	410	143	102	8500	2.0	C-P	138.00	2206	0.5	200	54.30
Saadatmanesh et al. (1991)	B	200	456	253	150	8500	1.5	G-P	37.23	400	6.0	152	125.00
Sharif et al. (1994)	P2	200	450	28	60	-	1.0	G-W	14.90	170	2	100	34.00
	P3	200	450	28	60	-	1.0	G-W	14.90	170	3	100	33.00
Spadea et al. (1998)	A1.1	200	435	57	150	-	-	C-W	152.00	2400	1.2	80	43.40
	A3.1	200	435	25	150	-	-	C-W	152.00	2400	1.2	80	37.40
Tan et al. (1999)	A00	200	500	57	75	-	-	C-W	230.00	3400	0.2	100	27.50
	A15	200	500	57	75	-	-	C-W	230.00	3400	0.2	100	24.70
	A25	200	500	57	75	-	-	C-W	230.00	3400	0.2	100	24.30
	A40	200	500	57	75	-	-	C-W	230.00	3400	0.2	100	24.70
	A60	200	500	57	75	-	-	C-W	230.00	3400	0.2	100	26.00
	A75	200	500	57	75	-	-	C-W	230.00	3400	0.2	100	21.90
	A90	200	500	57	75	-	-	C-W	230.00	3400	0.2	100	19.80
Täljsten et al. (1997)	SB1	200	527	157	75	8500	2.1	C-P	155.00	2400	1.4	120	71.40
	SB2	200	527	157	75	8500	2.4	C-P	155.00	2400	1.4	120	75.50
	SB3	200	527	157	75	8500	3.0	C-P	155.00	2400	1.4	120	73.90
	MB1	200	527	157	75	8500	2.4	C-P	210.00	2000	1.4	120	79.60
	HB1	200	527	157	75	8500	2.1	C-P	300.00	1400	1.4	100	80.10
	FB1	200	527	157	75	8500	0.4	C-W	95.00	1800	2.4	150	74.40
Teng and Yao (2007)	CS	199	536	157	100	-	2.0	C-W	256.00	4114	1.74	148	81.50
	CS-L3	199	536	157	100	-	2.0	C-W	256.00	4114	2.63	148	78.50
	CS-W100	199	536	157	100	-	2.0	C-W	256.00	4114	1.95	100	80.80
	CP	199	536	157	100	-	2.0	C-P	165.00	2800	1.2	148	76.00
	CS-C10	199	536	157	100	-	2.0	C-W	256.00	4114	1.86	148	99.40
Triantafillou et al. (1992)	4	200	517	17	40	-	-	C-W	186.00	1450	0.7	63	14.78
	5	200	517	17	40	-	-	C-W	186.00	1450	0.7	63	15.25
	6	200	517	17	40	-	-	C-W	186.00	1450	0.9	63	13.95

Table A. 5 (cont): Database of beams failed by end debonding. Ross et al. [9], Saadamanesh et al. [42], Sharif et al. [43], Spadea et al. [44], Tan et al. (cited in [22]), Täljsten et al. [45], Teng and Yao [23], Triantafillou et al. [16].

	Beam	b	h	d	d'	a	B	L	f _c '	E _s	f _{ys}	A _s	Reo	E' _s	f _y	A' _s
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[GPa]	[MPa]	[mm ²]	[tension]	[GPa]	[MPa]	[mm ²]
Triantafillou et al. (1992)	7	76	127	111	16	75	305	1220	44.7	200	517	33	2-4.6	-	-	-
	8	76	127	111	16	75	305	1220	44.7	200	517	33	2-4.6	-	-	-
Tumialan et al. (1999)	A3	150	300	250	-	-	1065	2130	51.7	207	427	792	4-15.9	-	-	-
	A8	150	300	250	-	-	1065	2130	51.7	207	427	792	4-15.9	-	-	-
	C2	150	300	250	-	-	1065	2130	51.7	207	427	792	4-15.9	-	-	-

	Beam	E _{yv}	f _{yv}	A _{sv}	s	E _a	t _a	Type	E _{fip}	f _{fip}	t _{fip}	b _{fip}	V _{exp}
		[GPa]	[MPa]	[mm ²]	[mm]	[MPa]	[mm]	[-]	[GPa]	[MPa]	[mm]	[mm]	[kN]
Triantafillou et al. (1992)	7	200	517	17	40	-	-	C-W	186.00	1450	0.9	63	12.80
	8	200	517	17	40	-	-	C-W	186.00	1450	1.9	64	18.67
Tumialan et al. (1999)	A3	207	427	143	125	2000	-	C-W	230.00	3400	0.495	150	86.10
	A8	207	427	143	125	2000	-	C-W	230.00	3400	0.99	75	98.20
	C2	207	427	143	250	2000	-	C-W	230.00	3400	0.495	150	79.30

Table A. 6: Database of beams failed by end debonding. Triantafillou et al. [16], Tumialan et al. (cited in [6]).

Appendix B: Database of beams failed by intermediate crack induced debonding

		b	h	a	B	L	d	d'	f _c '	E _s	f _{ys}	A _s	E _s '	f _y	A _s '	t _a	Type	E _{frp}	f _{frp}	t _{frp}	b _{frp}	V _{exp}	type
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[GPa]	[MPa]	[mm ²]	[GPa]	[MPa]	[mm ²]	[mm]	[-]	[GPa]	[MPa]	[mm]	[mm]	[kN]	of test
Bonacci (2000)	B2	270	400	101	1300	3650	341	54	22.6	201	484	900	199	507	142		C-W	230.00	3400	0.165	250	148	f.p.b.t
Garden et al. (1998)	C4u, 1.0	100	100	632	587	587	84	16	51.2	215	350	85	215	350	57	2.0	C-P	111.00	1414	0.82	67	15.43	t.p.b.t.
	C5u, 1.0	100	100	817	772	772	84	16	51.2	215	350	85	215	350	57	2.0	C-P	111.00	1414	0.82	67	11.33	t.p.b.t.
	A1	150	300	0	1065	2130	250	-	51.7	207	427	792	207	-	-		C-W	230.00	3400	0.165	150	72.8	t.p.b.t.
	A2	150	300	0	1065	2130	250	-	51.7	207	427	792	207	-	-		C-W	230.00	3400	0.33	150	84.9	t.p.b.t.
	A7	150	300	0	1065	2130	250	-	51.7	207	427	792	207	-	-		C-W	230.00	3400	0.33	75	86.1	t.p.b.t.
	C1	150	300	0	1065	2130	250	-	51.7	207	427	792	207	-	-		C-W	230.00	3400	0.165	150	77.2	t.p.b.t.
	B3u, 1.0	100	100	20	340	900	84	16	43.2	215	350	85	215	350	57	2.0	C-P	111.00	1414	0.82	67	17	f.p.b.t.
	B4u, 1.0	100	100	20	400	900	84	16	43.2	215	350	85	215	350	57	2.0	C-P	111.00	1414	0.82	67	17.25	f.p.b.t.
Rahimi et al. (2001)	B5u, 1.0	100	100	20	400	900	84	16	43.2	215	350	85	215	350	57	2.0	C-P	111.00	1414	0.82	67	17.3	f.p.b.t.
	B1u, 4.5	145	230	40	1525	4400	205	25	37.6	220	556	226	220	556	101	2.0	C-P	115.00	1284	1.28	90	30	f.p.b.t.
	B3	200	150	85	750	2100	120	30	49.2	210	460	157	210	460	157	2.0	C-P	127.00	1532	0.4	150	27.6	f.p.b.t.
	B4	200	150	85	750	2100	120	30	49.2	210	460	157	210	460	157	2.0	C-P	127.00	1532	0.4	150	26.3	f.p.b.t.
	B5	200	150	85	750	2100	120	30	49.2	210	460	157	210	460	157	2.0	C-P	127.00	1532	1.2	150	34.9	f.p.b.t.
	B6	200	150	85	750	2100	120	30	49.2	210	460	157	210	460	157	2.0	C-P	127.00	1532	1.2	150	34.8	f.p.b.t.
	B7	200	150	85	750	2100	120	30	49.2	210	460	157	210	460	157	2.0	G-P	36.00	1074	1.8	150	29.6	f.p.b.t.
	B8	200	150	85	750	2100	120	30	49.2	210	460	157	210	460	157	2.0	G-P	36.00	1074	1.8	150	30.8	f.p.b.t.
Saadatmanesh (1991)	E	205	455	155	1983	4575	400	55	35.0	200			200	456	253	1.5	G-P	37.23	400	6	152	32.5	f.p.b.t.
Triantafillou (1992)	4	76	127	75	458	1220	111	-	44.7	200	517	33	200	-	-	2.0	C-P	186.00	1450	0.65	63.2	14.8	f.p.b.t.
	5	76	127	75	458	1220	111	-	44.7	200	517	33	200	-	-	2.0	C-P	186.00	1450	0.65	63.2	15.3	f.p.b.t.
	6	76	127	75	458	1220	111	-	44.7	200	517	33	200	-	-	2.0	C-P	186.00	1450	0.9	63.3	14	f.p.b.t.
	7	76	127	75	458	1220	111	-	44.7	200	517	33	200	-	-	2.0	C-P	186.00	1450	0.9	63.3	12.8	f.p.b.t.
	8	76	127	75	458	1220	111	-	44.7	200	517	33	200	-	-	2.0	C-P	186.00	1450	1.9	63.9	18.7	f.p.b.t.
Yao et al. (2002)	CP1	301.5	150.5	100	1000	1100	117.4	-	27.0	208	343	314	208	-	-	1.0	C-P	165.00	2800	1.2	50	19.95	c.b.t.
	CP2	303.6	151.9	100	1000	1100	111.3	-	37.7	208	343	314	208	-	-	1.0	C-P	165.00	2800	1.2	50	17.58	c.b.t.
	CP3	302.7	150	100	1000	1100	108.2	-	12.6	208	343	157	208	-	-	1.0	C-P	165.00	2800	1.2	50	13.31	c.b.t.
	CP5	304	149	100	1000	1100	117.4	-	25.6	210	355	157	210	-	-	1.0	C-P	165.00	2800	1.2	50	10	c.b.t.
	CS1	303	150.8	100	1000	1100	115.3	-	21.4	208	343	157	208	-	-	0.5	C-W	271.00	3720	0.165	50	8.51	c.b.t.
	GS1	302	151.2	100	1000	1100	117.9	-	22.6	208	343	157	208	-	-	0.3	G-W	20.50	269	1.27	89.7	10	c.b.t.

Table B. 1: Database of beams failed by intermediate crack induced debonding. Type of test: f.p.b.t.=four point bending test; c.b.t.=cantilever beam test; 3.p.b.t.=three point bending test. Bonacci [46], Garden et al. [36], Rahimi et al. [10], Saadatmesh [42], Triantafillou [16], Yao et al. [47].

		b	h	a	B	L	d	d'	f _c '	E _s	f _{ys}	A _s	E' _s	f _y	A' _s	Type	E _{frp}	f _{frp}	t _{frp}	b _{frp}	V _{exp}	type
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[GPa]	[MPa]	[mm ²]	[GPa]	[MPa]	[mm ²]	[-]	[GPa]	[MPa]	[mm]	[mm]	[kN]	of test
Beber et al. (1999)	VR5	120	250	75	783	2349	214	34	33.6	200	565	157	200	565	57	C-	230.0	3400	0.44	120	51.1	f.p.b.t.
	VR6	120	250	75	783	2349	214	34	33.6	200	565	157	200	565	57	C-	230.0	3400	0.44	120	50.3	f.p.b.t.
	VR7	120	250	75	783	2349	214	34	33.6	200	565	157	200	565	57	C-	230.0	3400	0.77	120	62.1	f.p.b.t.
	VR8	120	250	75	783	2349	214	34	33.6	200	565	157	200	565	57	C-	230.0	3400	0.77	120	62	f.p.b.t.
	VR9	120	250	75	783	2349	214	34	33.6	200	565	157	200	565	57	C-	230.0	3400	1.1	120	64.8	f.p.b.t.
	VR10	120	250	75	783	2349	214	34	33.6	200	565	157	200	565	57	C-	230.0	3400	1.1	120	68.5	f.p.b.t.
Benjamin (2005)	L1	152	254	-	2269	4537	229	25	23.3	200	429	398	200	429	157	C-	155.1	2792	1.4	25	39.9	3.p.b.t.
	H1	152	254	-	2269	4537	229	25	23.3	200	429	398	200	429	157	C-	155.1	2792	1.4	25	37.7	3.p.b.t.
	L2	152	254	-	2269	4537	229	25	23.3	200	429	398	200	429	157	C-	155.1	2792	1.4	51	44.3	3.p.b.t.
	L2x1	152	254	-	2269	4537	229	25	23.3	200	429	398	200	429	157	C-	155.1	2792	1.4	51	45.5	3.p.b.t.
	H2	152	254	-	2269	4537	229	25	23.3	200	429	398	200	429	157	C-	155.1	2792	1.4	51	43.5	3.p.b.t.
	H2x1	152	254	-	2269	4537	229	25	23.3	200	429	398	200	429	157	C-	155.1	2792	1.4	51	45.1	3.p.b.t.
	L4	152	254	-	2269	4537	229	25	23.3	200	429	398	200	429	157	C-	155.1	2792	1.4	102	51.8	3.p.b.t.
	H4	152	254	-	2269	4537	229	25	23.3	200	429	398	200	429	157	C-	155.1	2792	1.4	102	49.2	3.p.b.t.
Bonacci & Maalej (2000)	B2	270	400	101	1300	3650	350.2	44.8	22.6	201	485	900	201	485	143	C-	230.0	3400	0.3	250	148	f.p.b.t.
Chan et al. (2001)	B2	250	470	50	1600	4600	430	40	42.4	200	505	628	200	505	402	C-	181.0	3180	1.2	150	142.5	f.p.b.t.
	B3	250	470	50	1600	4600	430	40	42.4	200	505	943	200	505	402	C-	181.0	3180	1.2	150	176	f.p.b.t.
	B6	250	470	50	1600	4600	430	40	42.4	200	505	628	200	505	402	C-	181.0	3180	1.2	150	129	f.p.b.t.
	B8	250	470	50	1600	4600	430	40	42.4	200	505	1257	200	505	402	C-	181.0	3180	1.2	150	220	f.p.b.t.
Chan and Li (2000)	S6-50-0	330	100	-	850	2500	76	-	53.4	200	677	113	200	-	-	C-	165.0	2940.0	1.2	50	14.9	f.p.b.t.
	S8-50-0	330	100	-	850	2500	76	-	53.4	200	653	201	200	-	-	C-	165.0	2940.0	1.2	50	17.9	f.p.b.t.
	S8-50-0F	330	100	-	850	2500	76	-	53.4	200	653	201	200	-	-	C-	165.0	2940.0	1.2	50	16.45	f.p.b.t.
Delaney (2006)	R_UC_C1	150	200	50	820	1800	160	40	49.1	190	477	266	190	477	266	C-	50.5	817	1.4	150	44.4	f.p.b.t.
	R_UC_C2	150	200	50	820	1800	160	40	50.1	190	477	266	190	477	266	C-	50.5	817	1.4	150	49.5	f.p.b.t.
	R_UC_C3	150	200	50	820	1800	160	40	50.2	190	477	266	190	477	266	C-	50.5	817	1.4	150	45.3	f.p.b.t.
	R_UC_C4	150	200	50	820	1800	160	40	50.4	190	477	266	190	477	266	C-	50.5	817	1.4	150	48.5	f.p.b.t.
Esfahani et al. (2006)	11-20D-1L	150	200	-	600	1600	162	25	24.1	200	350	628	200	365	157	C-W	237.0	2845	0.176	150	54.455	f.p.b.t.
Gao et al. (2004)	A0	150	200	150	500	1500	162	27	35.7	200	531	157	200	531	101	C-	235.0	4200	0.2	75	40.35	f.p.b.t.
	A10	150	200	150	500	1500	162	27	35.7	200	531	157	200	531	101	C-	235.0	4200	0.2	75	39.35	f.p.b.t.
	A20	150	200	150	500	1500	162	27	35.7	200	531	157	200	531	101	C-	235.0	4200	0.2	75	43.95	f.p.b.t.
Kishi et al. (1998)	A200-1	150	250	50	1050	2600	210	40	24.8	206	378	402	206	378	402	A-	126.5	2480	0.1	130	37	f.p.b.t.
	A200-2	150	250	50	1050	2600	210	40	24.8	206	378	402	206	378	402	A-	126.5	2480	0.1	130	38	f.p.b.t.
	A415-1	150	250	50	1050	2600	210	40	24.8	206	378	402	206	378	402	A-	126.5	2480	0.3	130	41.7	f.p.b.t.
	A623-1	150	250	50	1050	2600	210	40	24.8	206	378	402	206	378	402	A-	126.5	2480	0.4	130	39.5	f.p.b.t.

Table B. 2: Database of beams failed by intermediate crack induced debonding. Type of test: f.p.b.t.=four point bending test; c.b.t.=cantilever beam test; 3.p.b.t.=three point bending test. Beber et al. (as cite in [3]), Benjamin (as cite in [3]), Bonacci and Maalej [46], Chan et al. (as cite in [3]), Chan and Li (as cite in [3]), Delaney (as cite in [3]), Esfahani et al. [34], Gao et al. [48], Kishi et al. (as cite in [3]).

		b	h	a	B	L	d	d'	f _c '	E _s	f _{ys}	A _s	E' _s	f _y	A' _s	Type	E _{frp}	f _{frp}	t _{frp}	b _{frp}	V _{exp}	type
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[GPa]	[MPa]	[mm ²]	[GPa]	[MPa]	[mm ²]	[-]	[GPa]	[MPa]	[mm]	[mm]	[kN]	of test
Kishi et al. (1998)	A623-2	150	250	50	1050	2600	210	40	24.8	206	378	402	206	378	402	A-	126.5	2480	0.4	130	40.25	f.p.b.t.
	C300-1	150	250	50	1050	2600	210	40	24.8	206	378	402	206	378	402	C-	230.5	4070	0.2	130	39.6	f.p.b.t.
	C300-2	150	250	50	1050	2600	210	40	24.8	206	378	402	206	378	402	C-	230.5	4070	0.2	130	37.5	f.p.b.t.
	C445-1	150	250	50	1050	2600	210	40	24.8	206	378	402	206	378	402	C-	230.5	4070	0.2	130	42	f.p.b.t.
	C445-2	150	250	50	1050	2600	210	40	24.8	206	378	402	206	378	402	C-	230.5	4070	0.2	130	41.4	f.p.b.t.
Kishi et al. (2003)	A-250-1	150	250	100	1050	2600	210	40	29.6	210	406	402	210	406	402	A-	118.0	2060	0.3	130	42.1	f.p.b.t.
	A-400-2	150	400	100	1050	2600	360	40	29.6	210	406	402	210	406	402	A-	118.0	2060	0.6	130	80	f.p.b.t.
Kotynia (2005)	B-08/S1	150	300	150	800	3000	270	30	33.8	195	490	339	195	490	157	C-	172.0	2915	1.2	80	90	f.p.b.t.
	BF-04/0.5S	150	300	150	1500	3000	270	30	33.0	199	421	157	199	421	157	C-	172.0	2915	1.2	40	48	3.p.b.t.
	BF-06/S	150	300	150	1500	3000	270	30	32.5	195	490	226	195	490	157	C-	172.0	2915	1.2	80	86	3.p.b.t.
	B-08/M	150	300	75	1400	4200	270	30	37.3	220	436	339	220	436	157	C-	220.0	2742	1.4	120	70	f.p.b.t.
	B-08/S2	150	300	75	1400	4200	270	30	32.3	220	436	339	220	436	157	C-	172.0	2915	1.2	50	47	f.p.b.t.
	B-083m	150	300	75	1400	4200	270	30	34.4	220	436	339	220	436	157	C-	170.0	2915	0.4	150	46	f.p.b.t.
Kurihashi et al. (1999)	B0-A	150	250	100	1100	2200	200	50	23.9	206	364	266	206	364	266	A-	126.5	2480	0.3	80	56.1	3.p.b.t.
	B40-A	150	250	100	1100	2600	200	50	23.9	206	364	266	206	364	266	A-	126.5	2480	0.3	80	26.15	f.p.b.t.
	B0-C	150	250	100	1100	2200	200	50	23.9	206	364	266	206	364	266	C-	126.5	4070	0.2	80	55.1	3.p.b.t.
Kurihashi et al. (2000)	R7-2	150	250	100	1450	3400	210	40	28.2	206	378	402	206	378	402	A-	126.5	2480	0.6	130	34.95	f.p.b.t.
	R6-2	150	250	100	1260	3020	210	40	28.2	206	378	402	206	378	402	A-	126.5	2480	0.6	130	41.3	f.p.b.t.
	R5-2	150	250	100	1050	2600	210	40	28.2	206	378	402	206	378	402	A-	126.5	2480	0.6	130	46.5	f.p.b.t.
	R4-2	150	250	100	840	2180	210	40	28.2	206	378	402	206	378	402	A-	126.5	2480	0.6	130	58.6	f.p.b.t.
	R3-2	150	250	100	650	1800	210	40	28.2	206	378	402	206	378	402	A-	126.5	2480	0.6	130	77.55	f.p.b.t.
Leung (2004)	B11	300	800	800	2400	7200	740	60	41.5	200	526	2513	200	526	1030	C-	235.0	4200	0.9	300	508.8	f.p.b.t.
	B12	300	800	800	2400	7200	740	60	41.5	200	526	2513	200	526	1030	C-	235.0	4200	0.9	300	516.5	f.p.b.t.
	B21	150	400	400	1200	3600	370	30	41.5	200	535	603	200	535	226	C-	235.0	4200	0.4	150	137.2	f.p.b.t.
	B22	150	400	400	1200	3600	370	30	41.5	200	535	603	200	535	226	C-	235.0	4200	0.4	150	136.25	f.p.b.t.
	B31	75	200	200	600	1800	170	30	41.5	200	599	157	200	599	101	C-	235.0	4200	0.2	75	32.1	f.p.b.t.
	B32	75	200	200	600	1800	170	30	41.5	200	599	157	200	599	101	C-	235.0	4200	0.2	75	32.15	f.p.b.t.
	B41	75	200	200	600	1800	185	15	41.5	200	599	157	200	599	101	C-	235.0	4200	0.2	75	34.8	f.p.b.t.
	B42	75	200	200	600	1800	185	15	41.5	200	599	157	200	599	101	C-	235.0	4200	0.2	75	37.85	f.p.b.t.
	NB1-8	300	800	60	2400	7200	740	60	29.0	200	519	2513	200	519	1030	C-	235.0	4200	0.9	300	512	f.p.b.t.
	NB1-16	300	800	60	2400	7200	740	60	29.0	200	519	2513	200	519	1030	C-	235.0	4200	1.8	300	548.5	f.p.b.t.
	NB2-2	150	400	30	1200	3600	370	30	29.0	200	521	603	200	521	226	C-	235.0	4200	0.2	150	108.1	f.p.b.t.
	NB2-4	150	400	30	1200	3600	370	30	29.0	200	521	603	200	521	226	C-	235.0	4200	0.4	150	119.55	f.p.b.t.

Table B. 3: Database of beams failed by intermediate crack induced debonding. Type of test: f.p.b.t.=four point bending test; c.b.t.=cantilever beam test; 3.p.b.t.=three point bending test. Kishi et al. (as cite in [3]), Kishi et al. (as cite in [3]), Kotynia (as cite in [3]), Kurihashi et al. (as cite in [3]), Kurihashi et al. (as cite in [3]), Leung (as cite in [3]).

		b	h	a	B	L	d	d'	f _c '	E _s	f _{ys}	A _s	E' _s	f _y	A' _s	Type	E _{frp}	f _{frp}	t _{frp}	b _{frp}	V _{exp}	type
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[GPa]	[MPa]	[mm ²]	[GPa]	[MPa]	[mm ²]	[-]	[GPa]	[MPa]	[mm]	[mm]	[kN]	of test
Leung (2004)	NB2-6	150	400	30	1200	3600	370	30	29.0	200	521	603	200	521	226	C-	235.0	4200	0.7	150	127.6	f.p.b.t.
	NB2-8	150	400	30	1200	3600	370	30	29.0	200	521	603	200	521	226	C-	235.0	4200	0.9	150	137.95	f.p.b.t.
	NB3-2	75	200	15	600	1800	185	15	29.0	200	599	157	200	599	101	C-	235.0	4200	0.2	75	34.45	f.p.b.t.
	NB3-4	75	200	15	600	1800	185	15	29.0	200	599	157	200	599	101	C-	235.0	4200	0.4	75	37.2	f.p.b.t.
Maalej and Leong (2005)	A3	115	146	25	500	1500	120	26	42.8	180	547	236	180	547	157	C-	235.0	3550	0.2	108	38.75	f.p.b.t.
	A4	115	146	25	500	1500	120	26	42.8	180	547	236	180	547	157	C-	235.0	3550	0.2	108	37.75	f.p.b.t.
	A5	115	146	25	500	1500	120	26	42.8	180	547	236	180	547	157	C-	235.0	3550	0.3	108	43.7	f.p.b.t.
	A6	115	146	25	500	1500	120	26	42.8	180	547	236	180	547	157	C-	235.0	3550	0.3	108	42.9	f.p.b.t.
	B3	230	292	50	1000	3000	240	52	42.8	183	544	943	183	544	628	C-	235.0	3550	0.3	216	131.75	f.p.b.t.
	B4	230	292	50	1000	3000	240	52	42.8	183	544	943	183	544	628	C-	235.0	3550	0.3	216	130.15	f.p.b.t.
	B5	230	292	50	1000	3000	240	52	42.8	183	544	943	183	544	628	C-	235.0	3550	0.7	216	147.35	f.p.b.t.
	B6	230	292	50	1000	3000	240	52	42.8	183	544	943	183	544	628	C-	235.0	3550	0.7	216	142.15	f.p.b.t.
	C3	368	467.2	80	1600	4800	384	83.2	42.4	181	552	2413	181	552	1609	C-	235.0	3550	0.5	368	326.45	f.p.b.t.
	C4	368	467.2	80	1600	4800	384	83.2	42.4	181	552	2413	181	552	1609	C-	235.0	3550	0.5	368	334.65	f.p.b.t.
Maeda et al. (2001)	SP-C	200	200	30	750	1800	165	35	35.0	200	360	266	200	360	266	C-	236.0	4120	0.2	200	39.15	f.p.b.t.
	SP-C2	200	200	30	750	1800	165	35	35.0	200	360	266	200	360	266	C-	236.0	4120	0.3	200	54.5	f.p.b.t.
M'Bazaa et al. (1996)	P111	200	300	50	1000	3000	240	-	44.3	200	439	157	200	-	-	C-	82.0	-	0.9	167	49.9	f.p.b.t.
Mikami et al. (1999)	A-140	150	250	100	1500	3000	200	50	23.9	200	364	266	200	364	266	C-	126.5	2480	0.3	80	40.2	3.p.b.t.
Niu et al. (2006)	A1	960	203	475	2100	4200	168	-	31.6	192	452	861	192	-	-	C-	184.0	2446	1.3	200	127.8	3.p.b.t.
	A2	960	203	475	2100	4200	168	-	33.4	192	452	861	192	-	-	C-	195.0	2384	1.2	200	130.4	3.p.b.t.
	A3	960	203	475	2100	4200	168	-	35.2	192	452	861	192	-	-	C-	80.4	724	1.4	300	102.7	3.p.b.t.
	A4	960	203	475	2100	4200	168	-	34.4	192	452	861	192	-	-	C-	108.5	859	2.6	300	133.7	3.p.b.t.
	A5	960	203	475	2100	4200	168	-	35.9	192	452	861	192	-	-	C-	108.5	859	2.6	200	107.4	3.p.b.t.
	A6	960	203	475	2100	4200	168	-	35.1	192	452	861	192	-	-	C-	80.4	724	1.4	200	93.7	3.p.b.t.
	B1	960	203	475	1600	4200	168	-	35.2	192	452	861	192	-	-	C-	184.0	2446	1.3	200	71.85	f.p.b.t.
	B2	960	203	475	1600	4200	168	-	34.5	192	452	861	192	-	-	C-	80.4	724	1.4	300	56.7	f.p.b.t.
	B3	960	203	475	1600	4200	168	-	34.7	192	452	861	192	-	-	C-	80.4	724	1.4	200	54.15	f.p.b.t.
	C2	960	203	475	2100	4200	168	-	33.3	196	446	896	196	-	-	C-	184.0	2446	1.3	200	133.8	3.p.b.t.
	C3	960	203	475	2100	4200	168	-	34.1	196	446	896	196	-	-	C-	80.4	724	1.4	300	107.2	3.p.b.t.

Table B. 4: Database of beams failed by intermediate crack induced debonding. Type of test: f.p.b.t.=four point bending test; c.b.t.=cantilever beam test; 3.p.b.t.=three point bending test. Leung (as cite in [3]), Maalej and Leong (as cite in [3]), Maeda et al. (as cite in [3]), M'Bazaa et al. (as cite in [3]), Mikami et al. (as cite in [3]), Niu et al [49].

		b	h	a	B	L	d	d'	f _c '	E _s	f _{ys}	A _s	E' _s	f _y	A' _s	Type	E _{frp}	f _{frp}	t _{frp}	b _{frp}	V _{exp}	type
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[GPa]	[MPa]	[mm ²]	[GPa]	[MPa]	[mm ²]	[-]	[GPa]	[MPa]	[mm]	[mm]	[kN]	of test
Niu et al. (2006)	C4	960	203	475	2100	4200	168	-	34.5	196	446	896	196	-	-	C-	80.4	724	1.4	200	90.5	3.p.b.t.
Seim et al. (2001)	S11	480	102	100	1015	2030	81	-	33.2	205	462	214	205	-	-	C-	198.0	2270	1.2	100	40.8	3.p.b.t.
	S12	480	102	100	1015	2030	81	-	33.2	205	462	214	205	-	-	C-	198.0	2270	1.2	100	42.5	3.p.b.t.
	S5	480	102	100	1015	2030	81	-	33.2	205	462	214	205	-	-	C-	198.0	2270	1.2	100	43.2	3.p.b.t.
	S1m	480	102	285	1015	2030	81	-	33.2	205	462	214	205	-	-	C-	198.0	2270	1.2	100	41.9	3.p.b.t.
	C12	480	102	100	1015	2030	81	-	33.2	205	462	214	205	-	-	G-	63.8	675	1.1	480	80.8	3.p.b.t.
	C21	480	102	100	1015	2030	81	-	33.2	205	462	214	205	-	-	G-	91.5	724	1.2	480	71.3	3.p.b.t.
Spadea et al. (1998)	A1.1	140	300	50	1800	4800	261	39	27.4	200	435	402	200	435	402	G-	152.0	2300	1.2	80	43.4	f.p.b.t.
	A3.1	140	300	50	1800	4800	263	37	28.5	200	435	402	200	435	402	C-	152.0	2300	1.2	80	37.4	f.p.b.t.
Takahashi and Sato (2003)	F1	200	300	50	700	1600	250	50	35.8	200	371	573	200	371	63	C-	230.0	3480	0.2	200	113.5	f.p.b.t.
	F2	200	300	50	700	1600	250	50	40.2	200	371	573	200	371	63	C-	230.0	3480	0.3	200	122	f.p.b.t.
	F3	200	300	50	700	1600	250	50	39.0	200	371	573	200	371	63	C-	230.0	3480	0.5	200	135	f.p.b.t.
	F5	200	300	50	700	1600	250	50	50.3	200	371	573	200	371	63	C-	230.0	3480	0.3	200	139	f.p.b.t.
	F6	200	300	50	700	1600	250	50	49.5	200	371	573	200	371	63	C-	230.0	3480	0.5	200	155.5	f.p.b.t.
Takeo et al. (1999)	No.2	160	260	70	1000	2000	225	35	31.3	200	356	266	200	356	266	C-	230.0	3480	0.2	140	33.85	3.p.b.t.
	No.3	160	260	70	800	2000	225	35	31.3	200	356	266	200	356	266	C-	230.0	3480	0.2	140	38.35	f.p.b.t.
	No.4	160	260	70	700	2000	225	35	31.3	200	356	266	200	356	266	C-	230.0	3480	0.2	140	43.5	f.p.b.t.
	No.5	160	260	70	550	2000	225	35	39.0	200	356	266	200	356	266	C-	230.0	3480	0.2	140	66	f.p.b.t.
	No.6	160	260	70	1000	2000	225	35	39.0	200	356	266	200	356	266	C-	230.0	3480	0.2	140	78.6	3.p.b.t.
	No.7	160	260	70	1000	2000	225	35	39.0	200	356	266	200	356	266	C-	230.0	3480	0.2	140	85.6	3.p.b.t.
Teng and Yao (2007)	CS-W50	150.6	255.3	50	500	1500	222.8	32.5	32.8	199	536	157.08	199	536	157.08	C-W	256.0	4114	2.01	50	71.3	f.p.b.t.
	GS	151.1	252.2	50	500	1500	217.7	34.5	32.3	199	536	157.08	199	536	157.08	G-W	22.5	351	1.67	148	82	f.p.b.t.
Wu et al. (1999)	H-s CFRP	150	200	50	600	1800	150	50	33.8	210	390	402	210	390	266	C-	230.0	4200	0.3	150	68.25	f.p.b.t.
	H-m CFRP	150	200	50	600	1800	150	50	33.8	210	390	402	210	390	266	C-	290.0	4000	0.3	150	78.45	f.p.b.t.
Wu et al. (2000)	RC-1	150	200	50	900	1800	160	40	30.2	210	360	402	210	360	266	C-	230.0	3200	0.1	140	65	3.p.b.t.
	RC-2	150	200	50	900	1800	160	40	30.2	210	360	402	210	360	266	C-	230.0	3200	0.1	140	68.9	3.p.b.t.
	RCS-1	150	200	50	900	1800	160	40	34.6	210	360	402	210	360	266	C-	230.0	3200	0.222	140	73.5	3.p.b.t.
Yao et al. (2005)	CP4	304.5	150.3	100	1000	1100	120.2	-	46.2	208	343	157	208	-	-	C-	165.0	2800	1.2	50	13.5	c.b.t.
	II-1	303	153	100	1000	1100	117.5	-	25.6	208	349	157	208	-	-	C-	257.0	4519	0.2	30	7.2	c.b.t.
	I-2	305	149	100	1000	1100	116.5	-	25.3	208	332	157	208	-	-	C-	257.0	4519	0.2	50	8.4	c.b.t.
	II-3	305	150	100	1000	1100	117.5	-	30.2	208	332	157	208	-	-	C-	257.0	4519	0.2	70	8.9	c.b.t.
	II-4	302	150	100	1000	1100	118.7	-	21.9	208	332	157	208	-	-	C-	257.0	4519	0.2	90	10.2	c.b.t.
	II-8	203.5	152	100	1000	1100	114.5	-	23.8	208	364	157	208	-	-	C-	257.0	4519	0.2	50	8.4	c.b.t.

Table B. 5: Database of beams failed by intermediate crack induced debonding. Type of test: f.p.b.t.=four point bending test; c.b.t.=cantilever beam test; 3.p.b.t.=three point bending test. Niu et al. [49], Seim et al. [50], Spadea et al. [44], Takahashi and Sato (as cited in [3]), Takeo et al. (as cited in [3]), Teng and Yao [23], Wu et al. 1999,2000 (as cited in [3]), Yao et al. [47].

		b	h	a	B	L	d	d'	f _c '	E _s	f _{ys}	A _s	E' _s	f _y	A' _s	Type	E _{frp}	f _{frp}	t _{frp}	b _{frp}	V _{exp}	type
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[MPa]	[GPa]	[MPa]	[mm ²]	[GPa]	[MPa]	[mm ²]	[-]	[GPa]	[MPa]	[mm]	[mm]	[kN]	of test
Yao et al. (2005)	II-9	320	151	100	1000	1100	117	-	22.2	208	332	157	208	-	-	C-	257.0	4519	0.2	30	6.9	c.b.t.
	III-1	203	155	100	1000	2000	121	-	22.5	208	346	157	208	-	-	C-	257.0	4519	0.2	50	15	3.p.b.t.
	III-2	199	156.5	100	1000	2000	122.5	-	21.2	210	373	157	210	-	-	C-	257.0	4519	0.2	100	21.4	3.p.b.t.
	III-4	150.1	153.5	100	1000	2000	122	-	22.4	206	351	157	206	-	-	C-	257.0	4519	0.2	50	18.4	3.p.b.t.
Zarniç et al. (1999)	1	200	300	100	960	2900	270	30	25.0	205	450	339	205	450	226	C-	150.0	2400	1.2	50	58.4	f.p.b.t.
	2	800	120	100	960	2900	105	15	25.0	205	450	339	205	450	156	C-	150.0	2400	1.2	100	31.5	f.p.b.t.
Zhang et al. (2005)	A-1	150	340	100	1200	3000	300	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	63.35	f.p.b.t.
	A-2	150	340	100	1200	3000	300	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	63.5	f.p.b.t.
	A-3	150	340	100	1200	3000	300	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	63.1	f.p.b.t.
	A-4	150	340	100	1200	3000	300	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	65.8	f.p.b.t.
	A-5	150	340	100	1200	3000	300	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	62.15	f.p.b.t.
	A-6	150	340	100	1200	3000	300	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	62.1	f.p.b.t.
	B-2	150	250	100	1050	2600	210	40	31.5	210	407	402	210	407	402	C-	230.0	3400	0.2	130	40.45	f.p.b.t.
	B-3	150	250	100	1050	2600	210	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.3	130	42.1	f.p.b.t.
	B-4	150	250	100	1050	2600	210	40	31.5	210	407	402	210	407	402	A-	78.5	2350	0.4	130	41.05	f.p.b.t.
	B-6	150	400	100	1050	2600	360	40	31.5	210	407	402	210	407	402	C-	230.0	3400	0.3	130	78.15	f.p.b.t.
	B-7	150	400	100	1050	2600	360	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	79.6	f.p.b.t.
	B-8	150	400	100	1050	2600	360	40	31.5	210	407	402	210	407	402	A-	78.5	2350	0.8	130	78.1	f.p.b.t.
	C-1	150	235	100	650	1800	210	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	74.95	f.p.b.t.
	C-2	150	250	100	650	1800	210	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	75.95	f.p.b.t.
	C-4	150	235	100	1050	2600	210	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	45.25	f.p.b.t.
	C-5	150	250	100	1050	2600	210	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	47.2	f.p.b.t.
	C-6	150	270	100	1050	2600	205	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	48.5	f.p.b.t.
	C-7	150	235	100	1450	3500	210	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	34.4	f.p.b.t.
	C-8	150	250	100	1450	3500	210	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	34	f.p.b.t.
	C-9	150	270	100	1450	3500	205	40	31.5	210	407	402	210	407	402	A-	118.0	2060	0.6	130	35.4	f.p.b.t.

Table B. 6: Database of beams failed by intermediate crack induced debonding. Type of test: f.p.b.t.=four point bending test; c.b.t.=cantilever beam test; 3.p.b.t.=three point bending test. Yao et al. [47], Zaniç et al. (as cited in [3]), Zhang et al. (as cited in [3]).

Figure 1: Failure modes of FRP-strengthened beams. a. FRP rupture; b. Crushing of compression concrete; c. Intermediate crack induced interfacial debonding; d. Concrete cover separation; e. Plate end interfacial debonding; f. Shear failure; g. Critical diagonal crack.

Figure 2: Plot of $V_{\text{exp}}/V_{\text{an}}$ of wet lay-up installations for the Casas and Pascual model on Normal probability scale (19 experimental tests).

Figure 3: Plot of $V_{\text{exp}}/V_{\text{an}}$ of wet lay-up installations for the Casas and Pascual model on Lognormal probability scale (19 experimental tests).

Figure 4: Regression analysis of Teng and Smith model for end debonding for wet lay-up (69 experimental tests).

Figure 5: Regression analysis of Colotti model for end debonding for wet lay-up (55 experimental tests).

Figure 6 Regression analysis of Casas and Pascual model for end debonding for wet lay-up (19 experimental tests).

Figure 7 : Regression analysis of Teng and Smith model for end debonding for prepeg (39 experimental tests).

Figure 8 : Regression analysis of Colotti model for end debonding for prepeg (34 experimental tests).

Figure 9 : Regression analysis of Casas and Pascual model for end debonding for prepeg (38 experimental tests).

Figure 10 : Regression analysis of Wu and Niu model for intermediate debonding (15 tests).

Figure 11 : Regression analysis of Casas and Pascual model for intermediate debonding (20 tests).

Figure 12 : Regression analysis of Casas and Pascual model for intermediate debonding including full scale beam tests (22 tests).

Figure 13 : Regression analysis of ACI model for intermediate debonding including the complete data set (183 experimental tests).