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Bond Strength of FRP Laminates to Concrete: State-of-the-Art Review

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ABSTRACT: Rehabilitation of existing infrastructure has become a priority in recent years as an alternative to the daunting costs of rebuilding structures. Traditional repair methods have drawbacks, many of which can be overcome through the use of fibre reinforced polymer FRP laminates. However, the behaviour of FRP rehabilitated structures has yet to be conveniently and accurately modelled in many situations. For example, better understanding of their failure modes will allow for more precise designs that will balance safety and cost. To strengthen an RC beam or slab for flexure, FRP laminates are usually bonded externally on the structural element. A common failure mode encountered in initial tests was the laminate debonding from the surface. Here, the bond strength and modes of debonding between the FRP laminates and reinforced concrete members strengthened in flexure are reviewed. Current models for predicting the bond strength between the laminates and concrete are also scrutinized.

1 INTRODUCTION

Although not yet fully understood, the application of FRPs has progressed beyond the experimental stage and has been implemented in a number of construction projects worldwide. The performance of the materials in these projects is being monitored closely as skepticism regarding long-term performance remains high. However, with each successful application, composite laminates demonstrate that they are a viable solution to many construction issues. For example, the flexural strength of a reinforced concrete beam/slab can generally be increased by bonding FRP laminates, with or without end anchors, to the soffit.

Fibre reinforced polymer technology is not limited to use in rehabilitation - it can also be used for the construction of new structures. FRP prestressing tendons, new FRP section profiles, FRP reinforced bars, FRP bridge decks and FRP cable stays have either been employed worldwide or are currently under investigation (Christoffersen et al. 1999; Braestrup 1999; Seible et al. 1999; Karbhari et al. 2000; Seible 2001; Uomoto and Mutsuyoshi 2002; Reising et al. 2004, Cheng et al. 2005; Cheng and Karbhari 2006; Guan et al. 2006). Due to the materials' durability, incorporating them in original planning may stave off repair costs longer than conventional designs.

2 BOND BETWEEN FRP AND CONCRETE

Much of the success of externally reinforcing members lies with the integrity of the bond between the FRP and the original material. Primary considerations include surface preparation, epoxy quality and laminate application: a successful bond depends heavily on the quality of the workmanship and less on the reliability of the material.

With few exceptions, it has been argued that bond is significantly affected by surface preparation and general concrete quality (Bizindavyi and Neale 1999; Chajes et al. 1996). The bonding concrete surface should also be free of weak layers and/or loose particles (Triantafillou et al. 1992). No preferred method of surface preparation has been stated and variations occurred with changes in concrete strength and test specimen geometry. Mechanical grinding, sandblasting and gritblasting, combined possibly with power washing or vacuuming to remove the debris are common means of surface preparation. Weak acid has even been applied to the surface of composite plates, neutralized prior to adhering the plates to the concrete (Saadatmanesh and Ehsani 1991). The goal of surface preparation is to roughen the surface and expose small to medium size pieces of aggregate. Due to the resulting unevenness, it is seldom possible to obtain a uniform epoxy thickness as recommended by the manufacturer (maximum recommended thickness is typically 3 mm). Numerous techniques have been developed to achieve constant epoxy thickness (Fanning and Kelly 2001; Swamy and Mukhopadhyaya 1999; Rahimi and Hutchinson 2001), but the most common method is the use of a hand roller. Although it may not be precise in creating a "uniform thickness", complete coverage of the laminates can be ensured by forcing excess epoxy out at the sides of the joint.

2.1 *Composite Action*

For reinforced concrete, perfect bond is assumed between the concrete and the steel reinforcement. The resulting strain compatibility is at the heart of many design and analysis methods. The degree to which strain can be transferred to an FRP, or conversely how much slip occurs in the adhesive, will determine the forces in each material and the overall resistance of the section. Swamy and Mukhopadhyaya (1995) stated that maintaining composite behaviour at all stages up to failure is one of the most important aspects of externally strengthened concrete beams. Nguyen et al. (2001) reported that the extent of the composite action and its effect on failure modes is not yet fully understood. The degree of composite behaviour may be related to the brittle failure modes that are one of the pitfalls of strengthening with FRP. Bond behaviour between the FRP laminates and the concrete surface is thus central to the issue of strain compatibility. Essentially the question is "how well does the adhesive bond the external reinforcement to the concrete surface?" In some studies, strain compatibility through the depth of the section appeared to occur (Spadea et al. 1998; Meier 1995; Lee et al. 1999; Triantafillou and Plevris 1992), whereas other investigators (Riad 1998; Sayed-Ahmed et al. 2004; Breña et al. 2003; Esfahani et al. 2007) have reported that strain compatibility does not occur, particularly close to failure. This leads to the question of what influences the degree of composite behaviour.

The degree and type of external anchorage was found to be important in maintaining the composite behaviour [Spadea et al. 1998]. Bakay (2003) showed that for a reinforced concrete beam strengthened with externally bonded CFRP strips with no additional anchorage, composite action halted at about 85% of the ultimate load of the beam. For another beam with additional anchorage, composite behaviour was maintained up to almost 99% of the ultimate load. The failure mode was observed to change, possibly because of the increased composite behaviour. The first beam failed from explosive debonding of the CFRP plate. Debonding was also caused failure of the second beam but was confined to local areas and was less destructive.

The properties and characteristics of the adhesive have been identified as significant factors for developing composite action (Triantafillou and Plevris 1992). Buyukozturk and Hearing (1998) regarded adherent stiffness as crucial for effective stress transfer. The ability of the adhesive to transfer stress depends on its bond with the concrete and the laminates, the interfacial shear stresses, and its material properties such as stiffness, flexibility and viscosity (Swamy and Mukhopadhyaya 1995). Low creep has also been identified as a desirable characteristic (Triantafillou et al. 1992). The result of any deficiencies in any of these properties can be detrimental to composite behaviour.

Based on experimental investigations performed by Chen and Teng (2001), Udea et al. (2003) and Yuan et al. (2004), Lue et al. (2005) argued that the major factors affecting bond-slip (and thus composite action) between the concrete and the FRP are:

- concrete compressive strength $f_c^{/}$
- bond length L up to a certain effective bond length L_e
- FRP laminate axial stiffness $E_p t_p$
- FRP-to-concrete width ratio b_p/b_c
- adhesive axial stiffness $E_a t_a$, and
- adhesive compressive strength f_a.

2.2 *Interface Stresses*

The interface stresses influence bond behaviour and thus the mode of failure. These stresses have been investigated primarily in relation to "ripping" failure in externally strengthened beams. Peeling forces at the plate ends combined with interface stresses are thought to be responsible for plate separation in many tests (Bizindavyi and Neale 1999). Large forces in the tension region cause high shear stresses in the concrete resulting in high interface stresses and peeling forces leading to premature failure (Swamy and Mukhopadhyaya 1995). These stresses are associated with concrete tensile capacity, flexural rigidity of the cracked plated section, surface preparation, adhesive strength and thickness, and the width to thickness ratio of the laminate. Additional studies found the interface stresses to vary with plate thickness and elastic modulus, the number of laminates and the shear span to depth ratio.

Some of the results were obtained from analytical investigations using the finite element method and others from laboratory experiments. For studies that investigated the same parameters there is good agreement in the findings. The results show:

- Increasing concrete compressive strength will result in slightly higher interface shear stresses at failure (Mukhopadhyaya and Swamy 2001).
- Increasing the elastic modulus of the adhesive results in higher interfacial stresses but has no effect on the location of the peak stresses (Teng et al. 2002).
- Reducing the adhesive thickness will increase interfacial shear and normal stresses and will affect the location of peak stresses (Teng et al. 2002).
- Increases in plate thickness will increase interfacial stresses but not influence the location of the maximum value (Teng et al. 2002; Mukhopadhyaya and Swamy 2001; Rahimi and Hutchinson 2001). Similarly increasing the number of laminates will increase the stress (Shahawy et al. 1996).
- Increasing the plate elastic modulus increases the interfacial stresses but has no impact on the location of peak value (Mukhopadhyaya and Swamy 2001; Teng *et al.* 2002).
- An increase in the shear span to depth ratio will reduce interface stresses even for plates of high elastic modulus (Mukhopadhyaya and Swamy 2001).

2.3 *Stress and Strain Distribution in the Bonded FRP Plate*

FRP strain and stress distributions have been investigated in both flexural experiments and bond tests. Hutchinson and Rahimi (1993) showed the strain distribution in the laminate to have no apparent connection to the overall behaviour of the beam. Conversely, Fanning and Kelly (2001) tested different beams with anchored plates and stated that they all failed as a result of plate peel-off when the strain gradients in the laminates reached approximately the same values. Maeda et al. (1997) also concluded that the strain gradient at failure could be considered the same for different plate stiffness and bond lengths, supporting Fanning and Kelly's results.

From bond strength "pull" tests (Figure 1), the initial stress distribution was found to be quadratic with peak values occurring near the loaded end. However with increasing load, the maximum stress location shifted towards the unloaded end. Failure load increased with bonded length up to a critical length beyond which the load remained constant (Brosens and van Gemert 1997). This critical bond length was related to specimen geometry and surface preparation (Bizindavyi and Neale 1999). It was argued by Lu et al. (2005) and Teng et al. (2002) that unlike internal reinforcement, the bond strength between externally bonded FRP laminates and the concrete surface can not be increased with increasing the bond length beyond a certain value, which they defined as the effective bond length Le. Unlike results obtained in direct bond tests, the stress distribution in flexural members strengthened with FRP will be affected by normal stresses perpendicular to the bond area resulting from the bending.

In flexural specimen tests Nguyen et al. (2001) determined that strain development in CFRP laminates can be separated into three distinct zones. Zone 1 is a de-stress region at the plate end, Zone 2 is a development region where strains increase linearly and Zone 3 is a composite region where composite behaviour is achieved. From these findings they expressed the development length ldev required to obtain composite behaviour as:

$$l_{dev} = c_c + \frac{d_p}{2} + \frac{4.61}{\lambda}$$
$$\lambda^2 = \frac{1}{E_p t_p} \frac{G_a G_c}{G_c t_a + G_a c_c} \tag{1}$$

where cc is the concrete cover thickness, dp is the depth to the bonded plate, tp is the thickness of the bonded plate. E and G are the modulus of elasticity and the shear modulus, respectively with the subscripts c, p and a referring to the concrete, the FRP and the adhesive, respectively.

2.4 *Bond Strength Models*

Many models (Table 1) have been proposed for the bond strength between FRP laminates and concrete. Some models were based on empirical relations calibrated with experimental data (Hiroyuki and Wu 1997; Tanaka 1996; Maeda et al 1997). Others were based on fracture mechanics theories, again with many parameters calibrated with experimental data (Holzenkämpfer 1994; Niedermeier 1996; Blaschko et al. 1998; Täljsten 1994; Yuan and Wu 1999; Yuan et al. 2001; Neubauer and Rostásy 1997). Design models were also proposed by adopting simple assumptions; then verified against test data (van Gemert 1980; Challal et al. 1998; Khalifa et al. 1998; Izumo et al. 1999; Dai et al. 2005; Sato et al. 2001; Sato et al.1997 and JCI 2003; Chen and Teng 2001). In all models, the stress state simulates a "pull" test on a specimen with bonded FRP plate (Figure 1).

Table 1: Bond strength models

Model Name	Model
Hiroyuki and Wu Model (Hiroyuki and Wu 1997)	$\tau_u = 0.27 \cdot L^{-0.669}$ $P_u = \tau_u \cdot L \cdot b_p$
Tanaka Model (Tanaka 1996; Sato et al. 1996)	$\tau_u = 6.13 - \ln(L)$ $P_u = \tau_u \cdot L \cdot b_p$

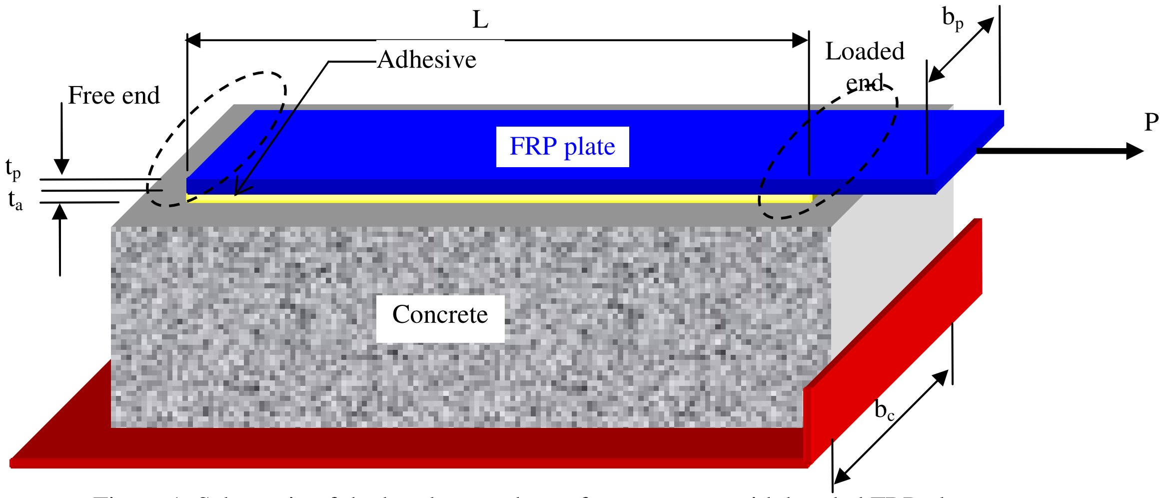

Figure 1. Schematic of the bond strength test for a concrete with bonded FRP plate.

Maeda Model (Maeda *et al.* 1997)	$\tau_u = (110.2\ x\ 10^{-6})E_p t$ $P_u = \tau_u L_e b_p \qquad L_e = e^{2.1235-0.580\cdot\ln(E_p t_p)}$
Khalifa *et al.* Model (Khalifa *et al.* 1998)	$\tau_u = (110.2\ x\ 10^{-6})(f_c'/42)\cdot E_p t_p$ $P_u = \tau_u L_e b_p \qquad L_e = e^{2.1235-0.580\cdot\ln(E_p t_p)}$
Sato Model (Sato *et al.* 2001; Sato *et al.*1997; JCI 2003)	$\tau_u = 2.68x10^{-5}\,(f_c')^{0.2}E_p t_p$ $P_u = \tau_u L_e(b_p + 7.4)$ $L_e = 1.89(E_p t_p)^{0.4} \quad if\ L > L_e : L_e = L$

Täljsten Model (Täljsten 1994)	$P_u = b_b\sqrt{\dfrac{2G_f E_p t_p}{1+(E_p t_p / E_c t_c)}}$
Yuan and Wu Model (Yuan and Wu 1999)	$P_u = b_b\sqrt{\dfrac{2G_f E_p t_p}{1+(E_p t_p b_b / E_c t_c b_c)}}$
Neubauer and Rostásy Model (Neubauer and Rostásy 1997)	$P_u = \begin{cases} 0.64k_p b_p\sqrt{E_p t_p f_t} & L \geq L_e \\ 0.64k_p b_p\sqrt{E_p t_p f_t}\cdot\alpha & L < L_e \end{cases}$ $\alpha = \left(\dfrac{L}{L_e}\right)\left(2-\dfrac{L}{L_e}\right) \quad L_e = \sqrt{\dfrac{E_p t_p}{2f_t}} \quad G_f = c_f f_t$

Iso's Model (JCI 2003)	$\tau_u = 0.93\,(f_c')^{0.44} \qquad P_u = \tau_u L_e b_p$ $L_e = 0.125(E_p t_p)^{0.57} \quad if\ L > L_e : L_e = L$
Yang Model (Yang *et al.* 2001)	$P_u = (0.5 + 0.08\sqrt{0.01\,E_p t_p / f_t})\cdot b_b L_e \tau_u$ $L_e = 100\ mm \qquad \tau_u = 0.5\cdot f_t$
Izumo Model (Izumo *et al.* 1999; JCI 2003)	$CFRP : P_u = \left(3.8 f_c'^{0.67} + 15.2\right) L\ E_p b_p t_p$ $AFRP : P_u = \left(3.4 f_c'^{0.67} + 69\right) L\ E_p b_p t_p$
Chen and Teng Model (Chen and Teng 2001)	$P_u = 0.427\beta_p\beta_L\sqrt{f_c'}L_e \quad L_e = \sqrt{E_p t_p / \sqrt{f_c'}}$ $\beta_p = \left[\dfrac{2-(b_p/b_c)}{1+(b_p/b_c)}\right]^{0.5} \quad \beta_L = \begin{cases} 1 & L \geq L_e \\ \sin\left(\dfrac{\pi L}{2L_e}\right) & L < L_e \end{cases}$
Holzenkämpfer Model (Holzenkämpfer 1994; Niedermeier 1996; Blaschko *et al.* 1998)	$P_u = \begin{cases} 0.78b_p\sqrt{2G_f E_p t_p} & L \geq L_e \\ 0.78b_p\sqrt{2G_f E_p t_p}\cdot\alpha & L < L_e \end{cases}$ $\alpha = \left(\dfrac{L}{L_e}\right)\left(2-\dfrac{L}{L_e}\right) \quad L_e = \sqrt{\dfrac{E_p t_p}{4f_t}}$ $G_f = c_f f_t k_p^2 \quad k_p = \sqrt{1.125\left(\dfrac{2-b_p/b_c}{1+b_p/400}\right)}$

van Gemert Model (van Gemert 1980)	$P_u = 0.5\cdot b_p\cdot L\cdot f_t$
Challal *et al.* Model (Challal *et al.* 1998)	$\tau_u = 0.5\,\tau_{\max}^{debonding} = 2.7/(1+k_1\tan 33^o)$ $k_1 = t_p(E_a b_a / 4E_p I_p t_a)^{0.25}$
Yuan *et al.* Model (Yuan *et al.* 2001)	$P_u = (\tau_f b_b \delta_f)/(\lambda_2(\delta_f - \delta_1))\sin(\lambda_2 a)$ *a is determined by solving :* $\lambda_1\tanh[\lambda_1(L-a)] = \lambda_2\tan(\lambda_2 a)$ $\lambda_1^2 = (\tau_f/\delta_1 E_p t_p)(1+(E_p t_p b_b / E_c t_c b_c))$ $\lambda_2^2 = (\tau_f/(\delta_f - \delta_1)E_p t_p)(1+(E_p t_p b_b / E_c t_c b_c))$

Where, b_c is the concrete section width, b_p is the width of the bonded FRP plate (mm), b_a is the width of the adhesive, c_f is a constant determined from a regression analysis of FRP pull test, E_p is the modulus of elasticity of the bonded FRP plate (MPa), E_a is the modulus of elasticity the adhesive, f'_c is the concrete compressive strength (MPa), f_t is the concrete surface tensile strength determined in a pull-off test according to DIN 1048, G_f is the fracture energy, k_p is a geometric factor related to the widths of the concrete and the bonded FRP plate, L is the bonded length (mm), L_e is the effective bond length (mm), I_p is the second moment of area of the FRP plate, P_u is the bond strength of a joint (N), t_a is the thickness of the adhesive, t_p is thickness of the bonded FRP plate (mm), β_L is a geometric bond length coefficient, β_P is a geometric width coefficient, τ_u is the ultimate shear stress (MPa), and τ_f and δ_l are the maximum shear stress and corresponding slip on the shear stress-slip (bond-slip) curve with a maximum slip of δ_f.

3 FELXURAL STRENGTHENING OF RC BEAMS/SLABS USING FRP

Strengthening of reinforced concrete beams and slabs was traditionally performed using externally bonded steel plates. One of the failure modes encountered was debonding of the steel plate which involved cracks progressing along the length of the RC member in the concrete cover, along the line of the flexural reinforcement, or in the adhesive material layer. The thickness of the laminated steel plate influences the stresses leading to debonding failures (Roberts and Haji-Kazemi 1989) with a limiting value for the plate width to thickness ratio b/t of 50 suggested by Swamy and Jones (1987). The adhesive material layer thickness also affects the behaviour of the strengthened members. However, Swamy and Jones (1987) argued that an adhesive layer thickness of 1.5 mm – 8.0 mm would not have a significant impact on the load capacity.

3.1 *Failure Modes of RC Beams/Slabs Strengthened Using FRP Plates*

As an alternative to bonded steel plates, many experimental investigations have been concerned with failure modes of reinforced concrete beams/slabs strengthened with FRP laminates (Ritchie et al. 1991; Saadatmanesh and Ehsani 1991; Triantafillou and Pleveris 1992; Chajes et al. 1994; Hefferman and Erki 1996; Shahawy et al. 1996; Arduini and Nanni 1997; Maalej and Bian 2001; Rahimi and Hutchinson 2001; Sayed-Ahmed et al. 2004; Lu et al. 2005; Hosny et al. 2006a; Esfahani et al. 2007). The failure modes can be separated into two categories based on the duration of composite action between the materials. When composite action is maintained until the ultimate load is reached, failure can occur in one of three modes depending on the reinforcement ratio and the shear strength of the beam:

- concrete crushing prior to or following yielding of the steel reinforcement,
- tensile rupture of the FRP, or
- shear failure of the concrete beam.

However, when composite action is not maintained until the ultimate load is reached, premature failure results from debonding of the FRP laminates, termed interfacial debonding (Teng et al. 2002; Lu et al. 2005, Sayed-Ahmed et al. 2004; Hosny et al. 2006a). Interfacial debonding is the most common mode of failure for RC beams strengthened in flexure using externally bonded FRP laminates. Currently codes of practice (CSA S806-02 and ACI 440.2R-02) and proposed design procedures (ISIS Canada 2001) can overestimate the flexural strength of reinforced concrete members with bonded FRP laminates through not accounting for interfacial debonding (Sayed-Ahmed et al. 2004; Hosny et al. 2006a and 2006b; Esfahani et al. 2007).

Interfacial debonding may occur as shown schematically in Figure 2 through (Smith and Teng 2002a and 2002b; Teng et al. 2002; Lu et al. 2005; Oehler et al. 2003; Teng et al. 2004; Esfahani et al. 2007):

- concrete cover separation,
- plate-end interfacial debonding,
- intermediate (flexure or flexure shear) crack-induced interfacial debonding,
- critical diagonal crack induced interfacial debonding.

Loss of composite action resulting from unevenness of the concrete surface is quite easy to conceptualize. However, failure resulting from debonding in the anchorage zone or in the vicinity of cracks is not as intuitive and the mechanisms are more difficult to understand.

Failure initiating in the uncracked anchorage zone may be referred to as ripping or end peel failure (defined above as plate-end interfacial debonding and concrete cover separation). Such failure is characterized by the formation of an inclined crack from the soffit of the beam to the level of the conventional flexural reinforcement. Cracking proceeds along the level of the internal reinforcement until the laminate is completely separated from the beam. This failure mode is found to occur frequently in beams where the laminate is terminated far from the supports and is commonly encountered for beams strengthened with steel plates.

Peeling of the composite laminate initiating at the location of a shear/flexure crack (defined above as intermediate crack-induced interfacial debonding) is characterized by a relative vertical displacement of the FRP across the crack opening. Once this has occurred there is a vertical component of the force in the FRP that puts the concrete in direct tension. When this vertical component exceeds the tensile strength of the concrete, cracking propagates back toward the support. The strength of the adhesive is not a limiting factor in this mode since it is stronger than the concrete. The concrete that remains bonded to the laminate after failure demonstrates crack progression through the concrete. This particular failure mode has not yet been thoroughly quantified by the research community. This failure mode has also been termed mid-span shear debonding or MSD (Bakay 2003).

3.2 *Parameters Influencing Failure Modes of Beams with Bonded FRP Plates*

3.2.1 *Plate Thickness*

Sharif et al. (1994) argued that thin FRP plates bonded to relatively lightly reinforced sections would fail as a result of laminate rupture. Increasing the plate thickness would drive the failure mechanism toward ripping or plate end interfacial debonding. Similar results were presented by Rahimi and Hutchinson (2001) who indicated that thickening of the laminate plate moved the failure toward the beam ends, indicating an increase in normal and shear stress with increasing plate thickness.

3.2.2 *External Anchorage*

External anchorage can take many forms with FRP laminated beams. Beneficial results by simply laminating the entire beam and allowing the reaction force to provide restraint have been reported by Ross et al. (1999) and Hutchinson and Rahimi (1993). Conversely, much more sophisticated designs incorporating angled steel sections, compression and side plates and wrapped FRP sheets have been employed. With no guidelines available, design relies on engineering judgment. Compiling results from numerous tests Bonacci and Maalej (2001) concluded that in about half of the cases where special anchorage was detailed, failure still resulted from plate separation. This illustrates the need for detailed design guidelines regarding external anchorage if it is to be used to alter failure mode, ductility or strength.

The use of bolts to anchor laminates is a successful means of preventing ripping failure but can result in the initiation of other brittle failure modes (Sharif et al. 1994). The shear strength of the bolt-anchored beams was estimated at 150% of that of beams without bolt anchorage. Another technique using a powder-actuated fastening system was examined by Lamanna et al. (2004). However, these techniques have not yet been applied in practice.

Using a variety of anchorage systems, primarily varying the number of external U-shaped steel stirrups, Spadea et al. (2000) tested the effects of anchorage on strength, failure mode and ductility. Beams with no external anchorage were stronger than non-strengthened beams but less strong than beams with external anchorage. Efficiency, based on strain in the FRP at ultimate, was less for non-anchored beams compared to anchored ones. Internal reinforcement details were found to determine the most effective type of external anchorage that can be used to increase both strength and ductility in FRP plated beams.

In further work Spadea et al. (1998) discussed the importance of additional anchorage in maintaining composite action between the external laminates and the concrete beam. External anchorage is required at both the beam-ends and intermittently in the span to ensure composite action up to failure. For one of the beams laminated with a CFRP plate and no additional external anchorage tested by Bakay (2003), composite action was lost at approximately 85% of the ultimate load. Although local debonding did occur in beams where the external reinforcement was anchored, separation was confined to local regions and the process was much less destructive to the overall structural performance. The conclusion was that external anchorage is best used to increase structural ductility although changes in failure mode can be observed with differing amounts and arrangements of anchorage.

Research by Shahawy and Beitelman (1999) on T-sections showed that full wrapping of the section resulted in full utilization of the concrete with failure resulting from crushing of the concrete. Beam sections where only the soffit was laminated failed when a crack developed at the level of the flexural reinforcement followed soon after by delamination of the concrete in the cover region with the laminate still bonded. It was concluded that partial wrapping is not an effective means of strengthening or rehabilitation.

Ritchie et al (1991) conducted tests with laminated beams that initially failed as a result of the concrete ripping mechanism. A system of external anchorage was developed that was able to prevent this mode of failure. An interesting conclusion reached was that for each beam the relationship between the force that needs to be transferred from the plate to the concrete and the bond area should be determined. It was found that this relationship would depend on the concrete strength and the applied loading.

3.2.3 *Laminate Orientation*

Norris et al (1997) tested several beams externally reinforced with composite laminates applied at various angles to the beam axis. The laminates were extended to within 25.4 mm (1 inch) of the support to simulate conditions in the field and had varying degrees of web coverage. These authors concluded that strength enhancement and failure mode were related to the direction of the reinforcing fibre. Off-axis application of the CFRP resulted in more ductile failures and was preceded by warning signs such as CFRP peeling and snapping sounds. A secondary conclusion was that brittle failure modes associated with the use of CFRP might be avoided by using particular combinations of fibres and orientations.

3.2.4 *Plated Length*

The difference between the end peel/ripping failure mechanism and failure initiating from a crack tip in the constant moment region was examined by Sebastian (2001). He concluded that curtailment of the bonded plates far from the support would increase the likelihood of end peel failure. The association between the amount of laminate plating within the shear span and ripping failure was also reported by Yang et al. (2003).

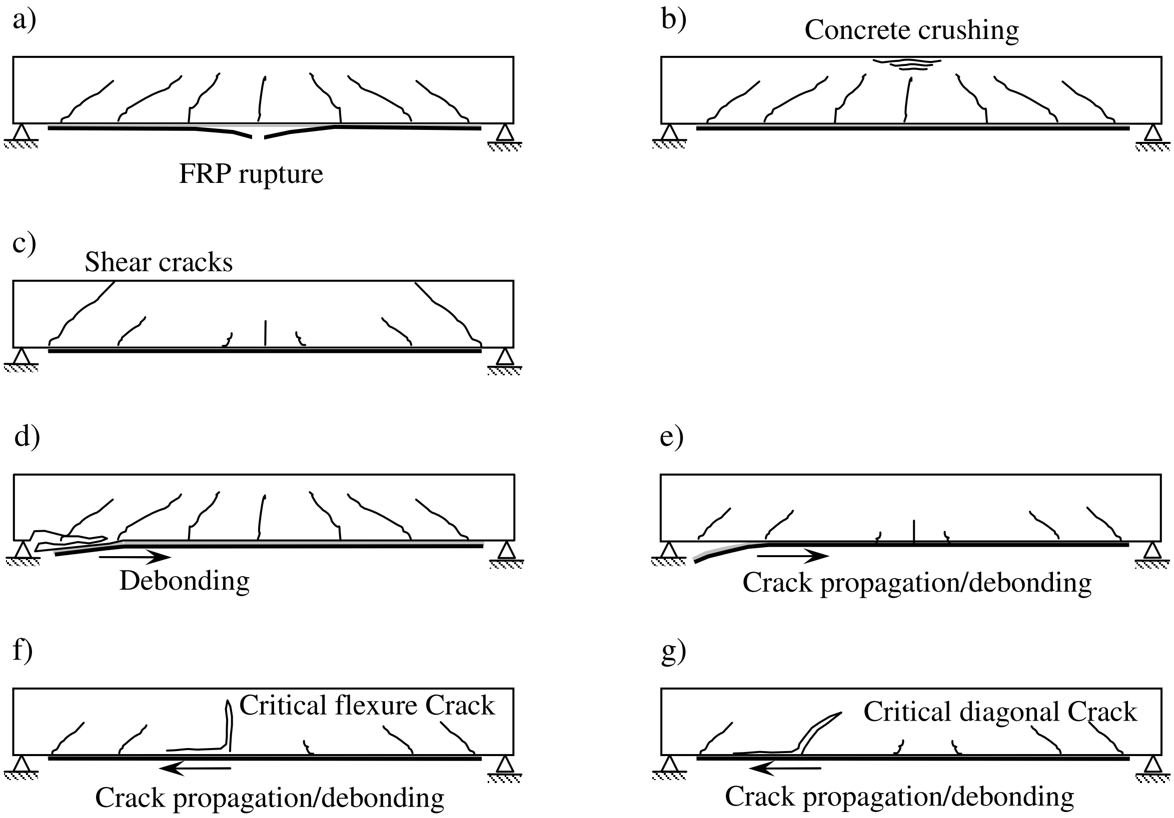

Figure 2. Failure modes of RC beams with bonded FRP-strips: a) flexure failure by FRP rupture; b) flexure failure by concrete crushing; c) shear failure; d) concrete cover separation; e) plate-end interfacial debonding; f) flexure crack-induced interfacial debonding; g) critical diagonal crack-induced interfacial debonding

Bonding laminates to the full length of beams increased strength with respect to other strengthened beams (Hutchinson and Rahimi 1993). This behaviour was attributed to the additional boundary conditions namely the vertical reaction at the support. Failure of the beams occurred when a shear crack propagated from the tensile zone to the external load point.

3.2.5 *Plate Stiffness*

Shahawy and Beitelman (1999) reported that premature failure of rigid plates resulting from end peel can be eliminated through the use of less rigid FRP fabric. Similar findings by Sebastian (2001) concluded that the use of stiff plates would contribute to the likelihood of end peel failure.

3.2.6 *Prestressing*

The effect of prestressing the FRP laminates on beam behaviour was investigated by Wight et al (2001). Prestressing the FRP significantly increased the cracking load compared to the non-stressed sheet. Beams with prestressed FRP failed due to sheet rupture while the non-stressed beams debonded at a section of combined moment and shear.

3.2.7 *Renforcement Ratio*

Ross et al. (1999) investigated the effect of the ratio between composite cross sectional area and reinforcement area for its effect on strength increase and mode of failure. Failure of heavily reinforced beams resulted from crushing of the concrete in the compression zone accompanied by apparent shear type cracks between the conventional reinforcement and the laminates. Lightly reinforced sections failed as a result of delamination of the FRP laminates. Heavily reinforced sections displayed less displacement and utilized a smaller percentage of the plate's ultimate tensile capacity. The strength increase was determined to depend on composite ratio, reinforcement ratio and the bond achieved between the laminates and concrete.

A small amount of FRP in conjunction with a wide bonding surface and low shear stress was thought to suppress the FRP debonding mechanism (Bonacci and Maalej 2001). For FRP rupture failures, appreciable strength gains, defined as a strength ratio of 1.5 or higher, were only obtained with lightly reinforced beams, approximately 20% of balanced. A similar relationship between strength increase and reinforcement ratio was reported by Arduini and Nanni (1997).

3.2.8 *Shear Stiffness*

Triantafillou and Plevris (1992) argued that failure originating at the base of a shear crack is controlled by the shear stiffness of the tensile reinforcement. The steel reinforcement and FRP laminates resist shear primarily through dowelling action. In this study the relation between the ultimate failure load and the combined shear stiffness was determined. Experimental coefficients were based on a small specimen size. Hutchinson and Rahimi (1993) concluded that unidirectional composites should not be expected to increase the shear capacity of composite beams.

3.2.9 *Influence of Additional Parameters*

Various other parameters have been investigated but on a much more limited basis.

- Sandblasting a specimen increased the ultimate load but had no effect on the mechanism of failure (Arduini and Nanni 1997).
- Strengthening is more effective in the case of deep members (Arduini and Nanni 1997).
- Preloading beams prior to applying FRP laminates had no effect on their performance (Rahimi and Hutchinson 2001).
- The ultimate capacity of FRP laminated beams is highly dependent on the concrete cover properties.
- The amount of shear reinforcement might be a factor in debonding failures.

From the above, a large variety of factors can be seen possibly to influence failure by interfacial debonding and consequently the premature failure of the FRP strengthened beams or slabs. A number of models have been proposed for interfacial debonding.

4 PLATE-END DEBONDING STRENGTH MODELS

FRP plate end debonding has been extensively investigated and various models (Table 2) have been proposed (Varastehpour and Hamelin 1997; Sadadatmanesh and Malek 1998; Wang and Ling 1998; Ahmed and van Gemert 1999; Tumialan et al. 1999; Raoof and Hassanen 2000; Smith and Teng 2002a,b; Teng and Yao 2007). Some other models were initially developed for beams with bonded steel plates and used without any modification for FRP plates (Oehlers 1992; Ziraba et al. 1994; Jansze 1997; Raoof and Zhang 1997). Smith and Teng (2002a,b) assessed some of these models versus many data available from literature. Teng et al. (2002) generally classified the plate end debonding models into three categories:

- Shear capacity based models: debonding failure strength is related to the shear strength of concrete without evaluating the interfacial debonding stress between the bonded plate and the concrete.
- Concrete tooth models: these models use the concept of a concrete "tooth" between two adjacent cracked surfaces. An effective length for the bonded plate is defined over which the shear stress is assumed to be uniform. Debonding occurs when this shear stress exceeds the tensile strength of concrete. In these models, the contribution of the shear capacity of the beam to the failure mode is open to question because it seems that failure is controlled by the flexural crack spacing in the concrete cover. Despite this, it is acknowledged in all these models that further understanding of the shear phenomenon is required with many unresolved issues remaining. Using the value of the maximum stress in the bonded plate determined from these models and the methods of strain compatibility or non-linear finite element analysis, the external loading required to create such a stress can be determined and the beam capacity estimated.
- Interfacial stress based models: these models adopt more logical assumptions but are labour intensive compared to the previous models. A concrete element adjacent to the end of a bonded plate is subjected to τ, σ_y and σ_x: shear stress, transverse normal stress perpendicular to the adhesive layer and the bonded plate (the peeling stress) and longitudinal stress, respectively.

5 INTERMEDIATE CRACK INDUCED INTERFACIAL DEBONDING

Plate-end interfacial debonding is a common mode of failure for reinforced concrete beams with bonded steel plates. Many investigations have been performed on this mode of failure. Thus, most of the previous models for interfacial plate-end debonding of bonded FRP laminates were based on initial models developed for steel plates.

In contrast, reinforced concrete beams with bonded FRP plates commonly suffer intermediate (flexure or flexure-shear) crack-induced interfacial debonding (Figure 2). The best descriptions of this mode of failure are provided by Meier (1995), Teng et al. (2002), Bakay (2003), Teng et al. (2003), Teng et al. (2004), Yuan et al. (2004), Chen et al. (2007), Eshsgani et al. (2007). Bakay (2003) argued that bending deformation of beams results in the creation of a flexural crack in the soffit of the beam. When shear forces also act, a vertical displacement can occur across the crack resulting in flexural forces in the composite laminate and tensile stresses in the concrete. When these tensile stresses exceed the tensile strength of the concrete a crack will begin to propagate parallel to the length of the beam in the concrete cover. The layer of concrete remaining bonded to the laminate indicates failure is through the concrete, not the adhesive.

Table 2. plate end debonding strength models

Model Name	Model
1. Shear capacity based models	
Oehlers Model (Oehlers 1992; Oehlers and Moran 1996)	$\frac{M_p}{M_{up}}+\frac{V_p}{V_{up}}\le 1.17$ $\frac{M_p}{M_{up}}\le 1.0 \quad and \quad \frac{V_p}{V_{uc}}\le 1.0$ $M_{up}=\frac{E_cI_{trc,c}f_{ct}}{0.901E_pt_p} \qquad f_{ct}=0.5\sqrt{f'_c}$ $V_{up}=V_c=[1.4-d/2000]b_cd[\rho_s f'_c]^{\frac{1}{3}}$ $where: [1.4-d/2000]\ge 1.1 \qquad \rho_s=A_s/b_cd$
Smith and Teng Model (Smith and Teng 2003)	$0.4\frac{M_p}{M_{up}}+\frac{V_p}{V_{up}}\le 1.0 \qquad if\ V_p\ge 0.6V_{up}$ $\frac{M_p}{M_{up}}\le 1.0 \qquad if\ V_p<0.6V$
Teng and Yao Model (Teng and Yao 2007; Yao and Teng 2007; Oehlers *et al.* (2004))	$\left(\frac{M_p}{M_{up}}\right)^2+\left(\frac{V_p}{V_{up}}\right)^2\le 1.0$ $M_{up}=\frac{0.488M_{uc}}{(\alpha_{flex}\alpha_{axial}\alpha_w)^{1/9}}\le M_{uc}$ $\alpha_{flex}=\frac{E_cI_{trc,c}-E_cI_{trc,o}}{E_cI_{trc,o}}$ $\alpha_{axial}=\frac{E_pt_p}{E_cd} \qquad \alpha_w=\frac{b_c}{b_p}\le 3$ $V_{up}=V_c+V_p+\varepsilon_{v,e}V_s$ $V_s=\frac{A_{sv}E_{sv}d}{s_v} \qquad \varepsilon_{v,e}=\frac{10}{(\alpha_{flex}\alpha_E\alpha_t\alpha_w)^{1/2}}$ $\alpha_E=E_p/E_c \qquad \alpha_t=\sqrt[3]{t_p/d}$
Jansze Model (Jansze1997; Ahmed and van Gemert 1999)	$V_{up}=\tau_{up}b_cd$ $\tau_{up}=0.18\sqrt[3]{\frac{3d}{B_{\mathrm{mod}}}}\left(1+\sqrt{\frac{200}{d}}\right)\sqrt[3]{100\rho_s f_c^{/}}$ $B_{\mathrm{mod}}=\sqrt[4]{\frac{(1-\sqrt{\rho_s})^2}{\rho_s}da^3} \qquad \rho_s=\frac{A_s}{b_cd}$ $B_{\mathrm{mod}}>B \quad modified\ shear\ span=(B_{\mathrm{mod}}+B)/2.$
Ahmed and van Gemert Model (Ahmed and van Gemert 1999)	$V_{up}=(\tau_{up}+\Delta\tau_{up})b_cd$ $\Delta\tau_{up}=\tau_{up}b_cd\left(\frac{S_s}{I_sb_p}-\frac{S_p}{I_{trc,c}b_a}\right)$ $+6188.5\left(\frac{\tau-4.121}{b_cd}\right)$ $\tau=\left(0.15776\sqrt{f'_c}+\frac{17.2366\rho_sd}{B}\right)+0.9\frac{A_{sv}f_{yv}}{s_vb_c}$
2. Concrete Tooth Models	
Raoof and Zhang Model (Raoof and Zhang 1996; Zhang *et al.* 1995; Raoof and Zhang 1997)	$\sigma_{s\min}=0.154\frac{L_ph_1b_c^2\sqrt{f_{cu}}}{h'b_pt_p\left(\sum O_{bars}+b_p\right)}$ $L_p=smaller\ of\ L_{p1}\ or\ L_{p2}$ $L_{p2}=\begin{cases}(21-0.25l_{\min})l_{\min} & l_{\min}\le 72mm\\ 3l_{\min} & l_{\min}>72mm\end{cases}$ $l_{\min}=\frac{A_ef_{ct}}{u_s\left(\sum O_{bars}+b_p\right)}$
Wang and Ling Model (Wang and Ling 1998)	$l_{\min}=\frac{A_ef_{ct}}{u_s\sum O_{barrs}+u_pb_p}$ $u_s=0.313\sqrt{f'_c} \qquad u_p=1.96\ MPa$
Raoof and Hassanen Model (Raoof and Hassanen 2000)	*Model I*: $L_{p2}=\begin{cases}(24-0.5l_{\min})l_{\min} & l_{\min}\le 40mm\\ 4l_{\min} & l_{\min}>40mm\end{cases}$ *Model II*: (*only for* $u_p=0.8\ MPa$) $L_{p2}=\begin{cases}(11.6-0.17l_{\min})l_{\min} & l_{\min}\le 56.5mm\\ 2l_{\min} & l_{\min}>56.5mm\end{cases}$
3. Interfacial Stress Based Models	
Ziraba *et al.* Model (Ziraba *et al.* 1994)	*Model I : plate end interfacial debonding* $\tau=\alpha_1 f_{ct}(C_{R1}V_o/f'_c)^{1.25} \qquad \sigma_y=\alpha_2C_{R2}\tau$ $C_{R1}=\left[1+\sqrt{\frac{K_s}{E_pb_pt_p}}\left(\frac{M_o}{V_o}\right)\right]\frac{b_pt_p}{I_{trc,p}b_a}(d_p-x_{trc,p})$ $C_{R2}=t_p\sqrt[4]{k_n/4E_pI_p}$ $K_s=G_ab_a/t_a \qquad K_n=E_ab_a/t_a$ $\tau+\sigma_y\tan\phi\le C\rightarrow$ $V_{up}=\frac{f'_c}{C_{R1}}\left[\frac{C}{\alpha_1f_{ct}(1+\alpha_2C_{R2}\tan\phi)}\right]^{0.8}$ $\alpha_1=35^o,\ \alpha_2=1.1,\ \phi=28^o$ $C=4.8\sim 9.5\ MPa$ *Model II : concrete cover separation* $V_{up}=V_c+kV_s \qquad k=2.4e^{-(0.8C_{R1}C_{R2}\cdot 10^6)}$ $V_c=(\sqrt{f'_c}+100\rho_s)(b_cd/6)$ $V_s=(A_{sv}f_{yv}d)/s$
Varastehpour and Hamelin Model (Varastehpour and Hamelin 1997)	$\tau=0.5\sqrt{\beta}(\lambda V_o)^{1.5} \qquad \sigma_y=C_{R2}\tau$ $V_{up}=\frac{1.6\tau_{\max}^{2/3}}{\lambda\beta^{1/3}}$ $\lambda=\frac{t_pE_p}{I_{trc,c}E_c}(d_p-x_{trc,c}) \qquad \beta=\frac{1.26B}{h_c^{0.7}t_pE_p}$ $\tau_{\max}=\frac{5.4}{1+C_{R2}\tan 33} \qquad C=5.4\ MPa \qquad \varphi=33^o$

Saadatmanesh and Makek Model (Saadatmanesh and Malek 1998)	$M = a_1(x+a)^2 + a_2(x+a) + a_3$ $\tau = t_p(b_3\sqrt{\frac{G_a}{t_a t_p E_p}} + b_2) \qquad \sigma_x = \frac{M_0^*}{I_c}x_c$ $\sigma_y = \frac{k_n}{2b_a\beta^{*3}}\left(\frac{V_p}{E_pI_p} - \frac{V_o^* + \beta^{*3}M_o}{I_cE_c}\right) + \frac{qE_pI_p}{b_bI_cE_c}$ $\sigma_1 = \frac{\sigma_x+\sigma_y}{2} + \sqrt{\left(\frac{\sigma_x+\sigma_y}{2}\right)^2 + \tau^2} \le f_{ct}$ $f_{ct} = 0.295(f_c^{/})^{\frac{2}{3}}$ $where: b_2 = \frac{E_p}{I_{cp}E_c}(d_p - x_{cp})(2a_1a + a_2)$ $b_3 = \frac{E_p}{I_{cp}E_c}(d_p - x_{cp})\omega$ $\omega = \left[(a_1a^2 + a_2a + a_3) + \frac{2\alpha_1E_pt_pt_a}{G_a}\right]$ $M_o^* = M_o + 0.5h_cab_p\tau$ $\beta^* = [(k_nb_p)/(4b_aE_pI_p)]^{1/4}$ $V_o^* = V_o - 0.5h_cb_pt_p(b_3\sqrt{G_a/(t_at_pE_p)} + b_2)$ $V_p = -0.5b_pt_p^2(b_3\sqrt{G_a/(t_at_pE_p)} + b_2)$
Tumialan *et al.* Model (Tumialan *et al.* 1999; Mirza *et al.* 1979)	$\tau = \bar{C}_{R1}\frac{E_p}{E_c}V_o \qquad \sigma_y = C_{R2}\tau \qquad \sigma_x = \frac{M_o}{I}x_c$ $\bar{C}_{R1} = \left[1 + \left(\frac{K_s}{E_pb_pt_p}\right)^{\frac{1}{2}}\left(\frac{M_o}{V_o}\right)\right]\frac{b_pt_p}{I_{cp}b_a}(d_p - x_{cp})$ $\sigma_1 = \frac{\sigma_x+\sigma_y}{2} + \sqrt{\left(\frac{\sigma_x+\sigma_y}{2}\right)^2 + \tau^2} \le f_{ct}$ $f_{ct} = 0.689\sqrt{f_c'}$

Where, a, p, c and s *are* subscripts refer to adhesive, FRP, concrete, and steel, respectively, A_e is the area of concrete in tension, A_s is the tension steel reinforcement area, A_{sv}, s_v, f_{uv} are the total cross sectional area, the longitudinal spacing and yield stress of the stirrups, respectively, b_c, b_p and b_a are the beam section width, widths of the FRP plate and width of adhesive, respectively, B is the shear span, B_{mod} is the modified shear span, C is the coefficient of cohesion, C_{R1},C_{R2} are obtained numerical solution (Roberts 1989) , d is the effective depth of the section, d_p is the depth from the compression face to the bonded plate, E and G are the modulus of elasticity and shear modulus, respectively, f_{ct} is the cylinder splitting tensile strength of concrete, f_{cu} is the concrete cube crushing strength ($f_c^{/}$=0.8 f_{cu}), h_1 is the distance from the centroid of the tensile steel reinforcement to the soffit of the beam, $h^{/}$ is the net height of the concrete cover measured from the base of the steel tension reinforcement to the soffit of the concrete beam, k is an empirical stirrup's efficiency factor, K_s and K_n are the shear and normal stiffness, respectively, L_p is the effective length for end anchor, L_{p1} is the length of the bonded plate in the shear span, $I_{trc,c}$ is the cracked second moment of area of FRP plated section transformed to concrete, I_c is the second moment of area of uncracked concrete section, I_{cp} is the second moment of area of uncracked concrete section with bonded FRP plate transformed to concrete with x_{cp} as the NA depth, $I_{trc,o}$ is the cracked second moment of area of the section, $I_{trc,p}$ is the second moment of area of a cracked plated section transformed into FRP (or steel according to the original derivation) having $x_{trc,p}$ as the NA depth, I_p is the second moment of area of the FRP plate, I_s is the cracked second moment of area of steel-plated section transformed to concrete, M_o is the bending moment at the end of the plate, M_{uc} is the unplated concrete section ultimate moment, O_{bars} is the total perimeter of the tension reinforcement bars, q is the uniformly distributed load (if exists), S_p and S_s is the first moment of area of the FRP plate and an equivalent steel plate about the NA of the cracked section transformed to concrete, t_p and t_a are the thickness of the bonded plate and the adhesive layer respectively, respectively, u_s is the steel to concrete average bond strength, u_p is the FRP plate to concrete average bond strength, V_c, V_p, and $\varepsilon_{v,e}V_s$ are the contributions of concrete, soffit plate and shear reinforcement to the beam's shear capacity, respectively, V_c is the strength of the beam without shear reinforcement calculated according to AS 3600 1988, V_o is the shear force at the end of the plate, V_{up} is the shear force at the plate end causing interfacial debonding, x is the distance along the bonded plate from its end (x=0 at the plate end), α_1 and α_2 are empirical factors defined by numerical investigation (Ziraba *et al.* 1994), $\varepsilon_{v,e}$ is the strain in the steel shear reinforcement, ϕ is the angle of internal friction, σ_{smin} os the longitudinal stress in the bonded steel plate at the initiation of peeling failure, σ_x is the longitudinal stress at the end of the bonded plate due to bending moment M_o determined from simple bending analysis for an uncracked section, σ_y is the peeling stress at the end of the bonded plate, and τ is the peak interfacial shear at the end of the bonded plate.

Meier (1995) concluded that the shearing effect resulting from vertical offset could be attributed to the following factors:

- load: axial force, shearing force, bending moment;
- geometry: concrete, steel reinforcement, composite laminate;
- mechanical properties: concrete, steel reinforcement, composite laminate;
- crack geometry: micro/macro roughness, width and vertical offset; and
- maximum composite laminate plate strain.

This discussion, however, is concerned primarily with behaviour following the formation of a crack between the line of flexural reinforcement and the laminate. In some instances propagation of this crack is very rapid leading to immediate failure, making prevention of such a crack a priority. In other instances, crack propagation is stable, progressing with increasing deformation of the beam.

Meier and Kaiser (1991) stated that peeling of the laminate as a result of the formation of shear cracks occurred in beams with relatively thick laminates and high levels of reinforcement near the load points. Meier concludes that the cover concrete between the FRP and the steel reinforcement is susceptible to relative vertical displacements from shear cracks in the concrete beam.

Fanning and Kelly (2001) presented research where the initial goal was to determine the relation between the length of the bonded plate within the shear span and the shear span length. Their belief was that this was an important factor in the brittle failure modes commonly witnessed with FRP strengthened beams. In their study, ten beams were constructed in pairs with varied plate lengths, and subject to 4-point bending. The compressive strength of the concrete was 80 MPa. Beams with plates bonded along their entire length were described to have failed due to the initiation of a shear crack at the soffit of the beam in line with one of the external load points. The remainder of the beams with smaller plated lengths failed as a result of ripping, initiating near the plate end. For beams failing as a result of ripping it seemed there was a relation between the failure load and the strain gradient in the bonded plate length in the shear span at failure. The failure load was higher when the mechanism of failure shifted from end peel/ripping to debonding in the region of a shear crack.

Buyukozturk and Hearing (1998) suggested that failure of beams at the location of shear cracks can depend on such things as shear reinforcement, crack configuration before strengthening, laminates length, and relative stiffness' of the laminates, adhesive and concrete.

Varastehpour and Hamelin (1996) conducted tests to see how anchorage affected the behaviour of laminates reinforced beams. Initial testing showed that the beams were failing as a result of failure of the concrete cover between the reinforcement and the laminates. As a result, external anchorage in the form of full height bonded angle plates was used in the shear span of two specimens. Mechanical anchorage (bolts) was successful in increasing the ultimate capacity by 8%, but was unable to prevent debonding of the laminates. All of these separation failures were a result of inadequate capacity in the concrete cover layer. When bonded angle plates were used, full flexural capacity was attained. Bonded angle plates provided the necessary anchorage for the bottom plate and prevented horizontal and diagonal shear cracks from developing in the concrete cover region beneath the flexural reinforcement.

Triantafillou and Plevris (1992) believe the occurrence of debonding at the location of a shear crack is related to the crack geometry and material properties. Since failure occurred beneath any conventional shear reinforcement, the steel and composite laminates provide the majority of the resistance through dowel action. Their equation relates the ratio of the crack opening and the combined shear stiffness of the materials to the externally applied load. In another experimental program (Bakay 2003) 8 beams were constructed, 7 being laminated with FRP. Beams with a low area fraction of FRP failed due to rupture of the composite laminates. All beams failing due to FRP delamination had an FRP area fraction greater that 0.43%.

Reinforced concrete T-sections with externally applied composite laminates were tested by Matthys et al. (2003). Here the effect of external anchorage was determined by testing beams with and without bolts as fasteners. The beam without bolts failed due to the end peel mechanism while the beam with external anchorage failed away from the anchorage due to vertical displacement. The bolts were instrumented and found to be resisting the normal stress that initiates ripping failure. Compared to the control beam the unanchored and anchored beams were 1.25 and 1.5 times stronger respectively.

A comprehensive study of various means of externally anchoring composite laminates was undertaken by Swamy and Mukhopadhyaya (1999). One test series was designed to determine the result of using FRP as a substitute for steel reinforcement while the second series investigated the effect of lower concrete strength and various means of external anchorage. Success in replacing conventional reinforcement by an appropriate amount of composite laminates will depend highly on the failure mode. In these tests debonding failure occurred which prevented full utilization of both the concrete and laminates. CFRP tension plate debonding was determined to be reliant upon the concrete strength. The amount of internal shear reinforcement or conventional flexural reinforcement did not appear to influence the plate separation failure to any appreciable degree. U shaped anchorage was successful in preventing vertical displacement but not horizontal bond slip.

Garden et al. (1998) tested a variety of beams with varying amounts of plate prestress and different forms of end anchorage. In all beams without plate prestressing, failure resulted from separation of the laminates at the base of a shear crack causing vertical displacement. Even the 25% prestress in the 1.0 m beam was unable to alter this mode of failure. Plates stressed to 50% and beyond failed from tensile fracture of the plate instead of plate separation resulting from vertical displacement. The thickness of the concrete layer and the length with which it remains bonded to the laminates were found to be functions of the shear span to beam depth ratio. Failure of the longer 4.5 m beams occurred in the vicinity of the load point whereas failure of the shorter 1.0 m beams occurred approximately half way along the shear span. The width of the concrete remaining attached to the laminates was equal to the beam width at the location of failure but tapered to equal the plate width. Differing forms of anchorage did not alter the mode of failure for the unstressed beams but did prevent complete separation as the plate sagged between the anchorages (Garden and Hollaway 1998).

McKenna (1993) reinforced a series of 18 reinforced concrete beams with varying amounts and orientations of CFRP laminates. The aim of the study was to determine the strengthening effect of varying amounts of FRP on uncracked and pre-cracked specimens and the effect of off-axis lamination. All beams were reinforced with 700 mm2 of tensile reinforcement and 200 mm2 of compression reinforcement. 11.3 mm diameter stirrups spaced at 95 mm provided shear reinforcement. The thickness for one ply of laminates was 62.5 mm2 and the elastic modulus of the CFRP was 56.12 GPa. Beams were 2.0 m in length with a clear span of 1.925 m and a constant moment region of 0.641 m.

Failure of all beams laminated with one layer of CFRP was initiated by buckling of the compression reinforcement. Subsequent to this, the FRP was observed either to rupture or delaminate as a result of relative vertical displacement in the shear span. The same sort of behaviour was seen in beams strengthened with two and three layers of CFRP; that is, buckling of the compression steel followed by either rupture or delamination of the laminates. McKenna attributes the difference in failure load and mode in some of these instances to arbitrary cracking of the concrete, and the width of the crack initiating failure. Plate separation in some of the off axis orientation tests took considerably more time but resulted in little if any strength increase. Conclusions resulting from this testing included:

- The compressive and tensile strength of the concrete used do not appear to affect the maximum load at which the carbon fibre sheet fails significantly.
- When CFRP sheets are used for external strengthening of reinforced concrete beams, the shear capacity of the concrete at the sheet to concrete interface must be confirmed.
- Use of CFRP sheets more evenly distributes flexural and flexural-shear cracks along the length of members, except in local regions where crack widths greater than 1 mm may develop indicating imminent sheet failure.
- Crack heights in the constant moment region are decreased while crack height in the combined moment and shear region is increased.

Riad (1998) and Sayed-Ahmed et al. (2004) tested 11.6 m long HC-type bridge girders that were taken from an existing bridge near the City of Calgary. These beams suffered from an inadequate amount of cover and were subject to the harsh climactic conditions in the area. Various strengthening measures were investigated including external post tensioning and CFRP lamination. The girders were graded visually and classified by the amount of apparent damage that had been sustained. Beams at opposite ends of the visual grading spectrum that were tested before strengthening, behaved nearly identically. Thus initial condition did not play a substantial role in the overall capacity of strengthened beams. Shear capacity was determined to be adequate and strengthening was focused on increasing the flexural strength that was about 60% of the shear capacity. Beams with spalled concrete were repaired using a grout to match the original profile of the girders. Two girders were strengthened with CFRP had laminated plates placed on the bottom of both webs with anchor sheets provided at either end of the girder.

In the first beam, an inclined crack propagated from the level of flexural reinforcement to the level of the CFRP laminates at a load of 260 kN. This crack originated under an external load point, in the region of combined highest moment and shear. Plate separation initiated at the location where the inclined crack contacted the CFRP and propagated towards the support. The maximum load sustained was 401 kN. It is interesting to note that there was progressive debonding with increasing load and that the failure was not nearly as sudden as reported in many other instances. Compressive failure of the concrete occurred approximately at the ultimate load level.

The second specimen behaved nearly identically and the maximum load attained was 396 kN. Strengthening increased the girder capacity by less than 12%. Strain compatibility was said to be lost following the development of the critical inclined crack.

Additional 2.0 m long test specimens were created in an attempt to reproduce the failure mode witnessed with the HC-type girders. Acting under the assumption that failure was a result of inadequate capacity in the cover concrete, beams of different cover areas were created all with the same steel and CFRP reinforcement. All of these test specimens failed from the ripping mechanism initiating at the plate end. Cracking similar to that observed in the girders was observed but none of these cracks led to failure of the beam. The authors concluded that the geometrical shape of the beam affects the strength enhancing ability of CFRP. Anchorage of the composite laminates was also determined to be a factor for overall performance. Surprisingly, despite the similar failure modes of the three beams, predicted strength was surpassed in the case of the second beam while the other two fell significantly below anticipated values.

5.1 *Intermediate Crack-Induced Debonding Strength Models*

Chen et al. (2007) argued that the debonding models currently in use are all based on simple pull-off tests.

However, intermediate crack-induced debonding occurs in two scenarios. In the first, no significant (major) crack exists between the free end of the bonded laminates and the significant crack where debonding initiates: typical for reinforced concrete beams or slabs with low reinforcement ratios. The stress state of this first scenario is almost similar to the simple pull-off tests. Thus, for intermediate flexure crack induced debonding resulting mainly from flexure cracks; and due to its approximate similarity with the simple pull-off test, the debonding model defined in Table 1 may be applicable (Chen and Teng 2001). This model was recently adopted for flexure strength prediction of full scale tests performed on hollow core slabs with bonded CFRP strips, and yielded acceptable results (Hosny et al. 2006). In this investigation, the maximum stress in the bonded FRP strips at failure was given according to Chen and Teng (2001) equation by:

$$\sigma_{up} = \alpha\beta_p\beta_L\sqrt{\frac{E_p\sqrt{f_c'}}{t_p}}$$

$$\beta_p = \left[\frac{2-(b_p/b_c)}{1+(b_p/b_c)}\right]^{\frac{1}{2}} \tag{2}$$

$$\beta_L = \begin{cases} L \geq L_e: & 1 \\ L < L_e: & \sin\left(\dfrac{\pi L}{2L_e}\right) \end{cases}$$

where α is an empirical factor which was calibrated against experimental data for beams and slabs a recommended values for α ranges between 0.38 and 0.43. b_p and b_c refer to the FRP plate and the concrete beam width respectively, L is the length of the FRP plate beyond the maximum moment location, E_P and $f_c^{/}$ are the elastic modulus of the FRP plate and the concrete compressive strength, respectively (both in MPa), t_p is FRP plate thickness in mm and L_e (in mm) is the effective bond length of the FRP plate which is defined by:

$$L_e = \sqrt{\frac{E_p t_p}{\sqrt{f_c'}}} \tag{3}$$

On the other hand, in the second scenario, one or more significant cracks exist between the debonding initiation crack and the free end of the bonded laminates. In this situation, the stress state is totally different from that of the simple pull-off tests. Thus, Chen et al. (2007) proposed the following equation for the ultimate load of a bonded FRP plate between two significant cracks:

$$\begin{aligned} P_u &= \frac{b_b\sqrt{2\cdot G_f E_p t_p}}{\sqrt{1-\beta^2}} \qquad L \geq \frac{1}{\lambda}\arccos\beta \\ &= \frac{b_b\sqrt{2\cdot G_f E_p t_p}\,\sin(\lambda L)}{1-\beta\cos(\lambda L)} \qquad L < \frac{1}{\lambda}\arccos\beta \\ \lambda &= \sqrt{\frac{\tau_f^2}{2G_f}\left(\frac{1}{E_p t_p}+\frac{b_b}{b_c E_c t_c}\right)} \\ &= \frac{\tau_f}{\delta_f E_p t_p}\left(1+\frac{E_p b_p t_p}{E_c b_c t_c}\right) \end{aligned} \tag{4}$$

where τ_f and δ_f are the local bond strength and the maximum slip of the bonded laminates between the two cracks, respectively, β is the ratio between the forces in the bonded laminates at the two cracks locations and G_f is the fracture energy which is the defined by the area under the bond-slip model adopted in the calculations for this joint. Despite their attempt to simplify it, the proposed equation of Chen *et al.* (2007) still contains implicit parameters which are very hard to evaluate practically.

The ACI 440.2 (2008) adopted a model for FRP debonding which is similar to the one poposed by Chen and Teng (2001) and Teng et al. (2002, 2004). and applied by Hony et al. (2006 a,b) and Bakay et al. (2009) to their experimental programmes. The model limits the effective strain in the FRP laminate to prevent the intermediate crack induced debonding failure mode. The limiting value for the effective FRP strain is given by:

$$\xi_{fd} = 0.41\cdot\sqrt{\frac{f_c^{/}}{n\cdot E_f\cdot t_f}} \quad \leq 0.9\xi_{fu} \tag{5}$$

where ε_{fd} is maximum strain allowed in the FRP laminate to prevent the debonding, $f_c^{/}$ is the 28 days standard concrete cylinder compressive strength, E_f and t_f are the elastic modulus and the thickness of the FRP laminate, respectively and n is then number of laminates layers. Equation 5 was calibrated using average measured values of FRP strains at debonding and the database for flexural tests experiencing intermediate crack induced debonding to determine the best fit coefficient of 0.41 (ACI 440.2, 2008).

6 SUMMARY

The behaviour of FRP rehabilitated structures has yet to be conveniently and accurately modelled in many situations. For example, better understanding of their failure modes will allow for more precise designs that will balance safety and cost. One of the most common failure modes of RC beams/slabs strengthened in flexure through external bonding of FRP laminates is debonding of the FRP from the surface.

Composite action between the bonded FRP laminates and the concrete section is very much related to the bond-slip behaviour between the two materials. Some of the currently available models for estimating bond strength of the bonded CFRP laminates to concrete are based on empirical relations calibrated to experimental data, while others are based on fracture mechanics theories, again with many parameters calibrated to experimental data. Design models have also been proposed by adopting simple assumptions and verified against test data. In all these models, the stress state simulates a pull-off test performed on a concrete specimen with a bonded FRP plate subjected to tension.

The failure modes of reinforced concrete members strengthened with bonded FRP laminates can be separated into two categories based on the duration of composite action between the materials. When composite action is maintained until the ultimate load is reached, failure can occur in one of three modes: concrete crushing, tensile rupture of the FRP, or shear failure of the concrete beam. When composite action is not maintained until the ultimate load is reached, premature failure results from debonding of the FRP laminates. This failure mode is termed interfacial debonding failure and is the most common mode of failure. Interfacial debonding may occur in the following modes: concrete cover separation, plate-end interfacial debonding, intermediate (flexure or flexure shear) crack-induced interfacial debonding, or critical diagonal crack induced interfacial debonding.

Models for plate-end debonding are generally classified into three categories: shear capacity based models, concrete tooth models, and interfacial stress based models. The shear capacity models relate debonding failure strength to the shear strength of concrete without evaluating the interfacial debonding stress between the bonded plate and the concrete. The concrete tooth models use the concept of concrete "tooth" between two adjacent cracked surfaces. An effective length for the bonded plate is defined over which the shear stress is assumed to be uniform. Debonding occurs when this shear stress exceeds the tensile strength of concrete. The interfacial stress based models adopt more logical assumptions but are labour intensive compared to the previous models. A concrete element adjacent to the end of a bonded plate is subjected to shear stress, transverse normal stress perpendicular to the adhesive layer and the bonded plate (also known as the peeling stress) and longitudinal stress.

Crack-induced interfacial debonding was discussed but it is evident that there is a lack of currently available models for this type of debonding compared to the plate-end debonding.

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