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Review

Premature failures in plate bonded strengthened RC beams with an emphasis on premature shear: A review

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The aim of the present study is to review the performance of plate bonded flexurally strengthened reinforced concrete beams. This paper also describes the methods and materials used for flexural and shear strengthening of reinforced concrete beams and weaknesses of plate bonded systems. The plate bonding method often has some serious premature debonding failure which can be classified as plate end debonding (end peeling), tension delamination and premature shear failure due to insufficient shear reinforcement. Premature failures must be prevented in order to utilize the full flexural capacity of flexural strengthened reinforced concrete beams. Premature shear failure is one of the major concerns of the flexurally strengthened reinforced concrete (RC) beams. Hence, proper design of external shear strengthening system is required for eliminating premature shear failure of flexurally strengthened RC beams. The review focuses on the possible model and design guideline available in the literature for eliminating premature failures. The paper also discusses a probable approach to eliminate premature shear.

Key words: Premature shear, strengthening, CFRP, debonding, eliminating shears.

INTRODUCTION

In general, reinforced concrete (RC) falls into two categories: flexure and shear. In order to take full advantage of the potential ductility of the RC members, it is desirable to ensure that flexure rather than shear govern ultimate strength because shear failure is sudden, brittle and catastrophic in nature which occurs with no advance warning of distress. Shear failure is more dangerous than flexural failure. For this reason RC beams must be designed to develop their full flexural capacity and assure a ductile flexural failure mode under extreme loading. However, if under-reinforced concrete beams required flexural strengthening, there is possibility of occurring shear deficiencies. From the practical point of view under-reinforced designed beam can take more loads due to flexural strengthening. Hence, shear failure needs to be eliminated in order to utilize the full flexural capacity. End anchorage system can be used to eliminate end delamination. Many of the existing and flexural strengthened RC beams have been found to be deficient in shear strength and in need of shear strengthening.

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Abbreviations: RC, Reinforced concrete; **CFRP,** carbon fibre reinforced polymer; **GFRP,** glass fibre reinforced polymer; **NSM,** near surface mounted; **FRP,** fibre reinforced polymers; **CDC,** critical diagonal crack; **IC debonding,** intermediate crack debonding.

MATERIALS AND METHODS USED FOR FLEXURAL STRENGTHENING

Using ferrocement laminate

Ferrocement is a thin composite material which is composed of cement mortar reinforced with uniformly distributed layers of continuous, relatively small diameter, wire meshes. The report containing the design and construction of ferrocement is published by the ACI committee 549 (ACI549-88R; ACI549.1-88R, 1988). Ferro-cement, being of the same cementitious material as reinforced concrete (RC), is ideally suited as an alternative strengthening component for the rehabilitation of RC structures (Paramasivam et al., 1998). It possesses higher tensile strength to weight ratio and a degree of toughness, ductility, durability and cracking resistance that is considerably greater than those found in conventional cement based materials.

The use of ferrocement was first introduced by Roumaldi (1987) and Iorns (1987) in the early 1980s. The ferrocement was utilized for its toughness, cracking resistance and ease of application to fit the difficult contours of structures. Initial investigation for using ferrocement laminate as strengthening components for the repair and strengthening of RC beam was carried out by Andrews and Sharma (1988).

Nassif et al. (1998) studied the behaviour of ferrocement concrete composite beams and the required area of steel mesh in the ferrocement layer to ensure overall adequate flexural response in comparison with a similar concrete section. For strengthening beams in flexure pre-fabricated ferrocement reinforcements were attached onto the beams tension face before the ferrocement matrix was cast to complete the laminate. All the beams strengthened without surface roughening (Ong et al., 1992) and without using the mechanical shear connectors (Paramasivam et al., 1994), exhibited localized horizontal cracks along the concrete/ferrocement interface and severe delamination of the ferrocement at failure.

Using externally bonded plates

Externally bonded steel plates and polymer composites are more significant materials regarding flexural strengthening purpose. Primarily steel plates were used for strengthening and repairing of RC members. Pioneering research with epoxy-bonded steel plates was carried out by L' Hermite and Breson (1967). Until 2010, many other researchers (Arslan et al., 2008; Macdonald and Calder, 1982; Roberts and Haji-Kazemi, 1989) have made several attempts to predict the behaviour and ultimate strength of flexurally upgraded RC structures and/or elements. The use of advanced composite fibre materials as external flexural reinforcement of concrete and other structures has progressed well in the past decade in selective applications where cost disadvantage is outweighed by a number of benefits such as corrosion resistant, low maintenance requirement, impact resistant, non conductive and non metallic, fire retardant, light weight and long life span. The main fibre types used are carbon (CFRP), glass (GFRP) and aramid (AFRP). There are two types of fibre reinforced polymers (FRP) materials currently available: plates and sheets (ISIS Educational Model 4, 2004). Carbon fibre reinforced polymer (CFRP) has relatively low modulus of elasticity and linear stress strain relationship up to rupture with no definite yield point (ISIS Educational Model 4, 2004).

Moreover, from literature it is found that 40% flexural strength enhancement is possible for RC beams strengthened with glass fibre reinforced polymer (GFRP) and 200% in case of CFRP (Pendhari et al., 2008). Thus, numerous research works are found for increasing flexural strength (Sharif et al., 1994; Saadatmanesh and Malek, 1998; Ashour et al., 2004; Chajes et al., 1994; Challal et al., 1998; Buyukozturk et al., 2004; Pham and Al-Mahaidi, 2004; Anania et al., 2005; Gao et al., 2005; Toutanji et al., 2006; Yang et al., 2009; Esfahani et al., 2007; Bogas and Gomes, 2008; Costa and Barros, 2010) attaching GFRP/CFRP plates to the soffit/tension face of the beams.

Using near surface mounted (NSM) slits/bars

The near surface mounted (NSM) reinforcement technique consists of placing the FRP reinforcing bars or strips into pre-sawn grooves in the concrete cover in the tension region of the reinforced concrete members and are bonded to the three sides of the groove using high-strength epoxy adhesive or cementitious grout (Täljsten B. and Carolin A., 2001). This technique has attracted extensive research in recent years (Lorenzis and Nanni, 2001; Lorenzis et al., 2002; Lorenzis and Nanni, 2002; Lorenzis et al., 2004; Lorenzis, 2000, 2004; Novidis et al., 2007; Lorenzis et al., 2000; Taljsten et al., 2003; El-Hacha and Rizkalla, 2004; Lorenzis and Teng, 2007; Al-Mahmoud et al., 2007; Kreit et al., 2008).

Configuration of the FRP reinforcements used for the NSM technique is controlled by the depth of the concrete cover (El-Hacha and Rizkalla, 2004). After installation, the NSM FRP reinforcements are protected against mechanical damage, wear, impact, and vandalism. This technique can also provide better fire resistance in the event of a fire (El-Hacha and Rizkalla, 2004); therefore, it could reduce the cost of fire protection measures.

Failure modes of plate bonded strengthened beams

Performance of flexurally strengthened RC beams are affected by several factors such as concrete strength and cover (Wu and Yoshizawa, 1999), level of loading (Shin and Lee, 2003), loading condition (Bonacci and Maalej, 2000, 2001), modulus of elasticity of CFRP and its center of gravity location relative to neutral axis (Heffernan and Erki, 1996), CFRP configuration (Gangarao and Vijay,

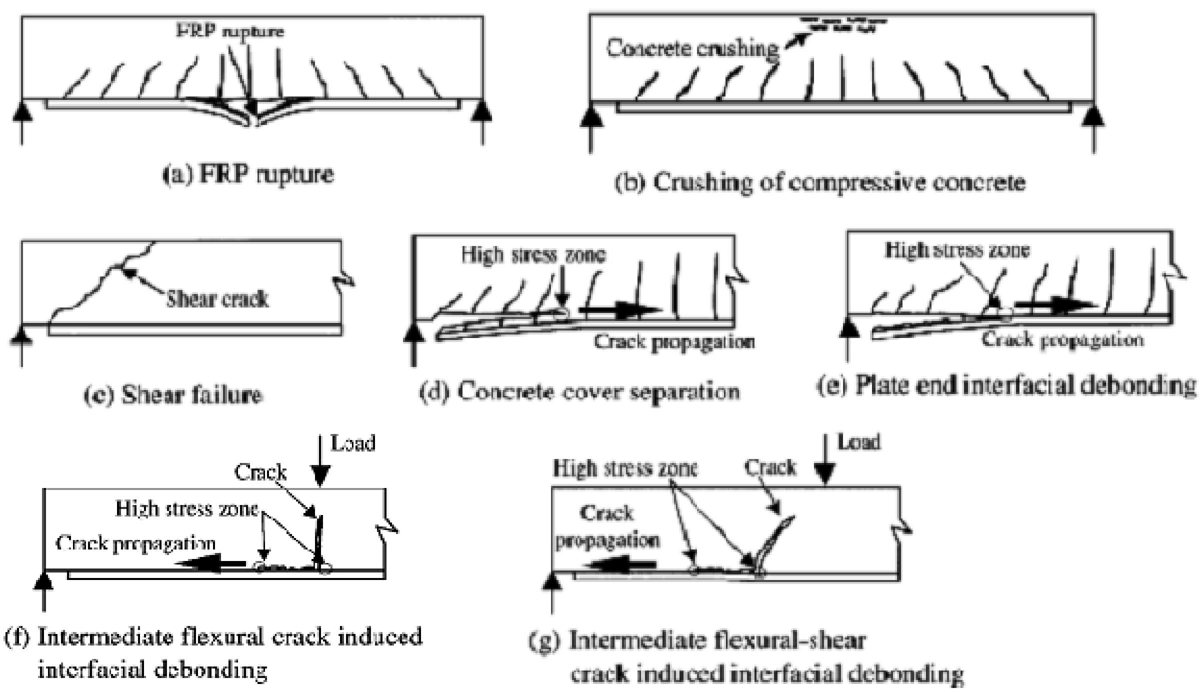

Figure 1. Failure modes of RC beams flexurally-strengthened with FRP soffit plate (Teng et al., 2003).

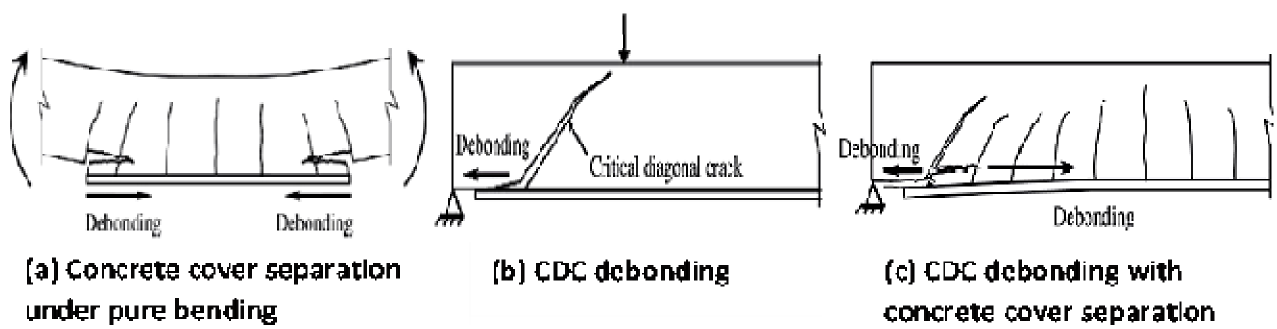

Figure 2. Debonding failure modes of plated RC beams (Yao and Teng, 2007).

1998; Brena and Macri, 2004), number of CFRP layers (Shahawy et al., 2001), width of laminate (Heffernan and Erki, 1996), length of laminate (Ramana et al., 2000) etc.

A number of failure moods for RC beams strengthened with FRP on tension faces have been observed in numerous experimental studies to date (Garden et al., 1998; Meier, 1995; Smith and Teng, 2003; Teng et al., 2003; Triantafillou and Plevris, 1992).

Existing studies showed there are seven types of failure modes of FRP plate bonded flexurally strengthened beams (Figure 1). These typical failure modes (Teng et al., 2003) are known as:

a) Flexural failure by FRP rupture
b) Flexural failure by concrete crushing
c) Shear failure
d) Concrete cover separation
e) Plate end interfacial debonding
f) Intermediate flexural crack induced inter-facial debonding and
g) Intermediate flexural-shear crack interfacial debonding.

The load carrying capacity of CFRP flexurally strengthened RC beams is often limited by the debonding failure modes shown in Figure 2. Those observed modes of debonding in FRP-plated beams can be broadly divided into two groups (Teng et al., 2003).

a) Those associated with high interface stresses near the ends of the bonded plate (failure mode (d) and (e)) which are collectively referred as plate end debonding.
b) Those induced by a flexural or flexural shear crack from plate ends debonding initiating in the high moment region and propagate towards one of the plate ends (failure modes (f) and (g) which are collectively referred as intermediate crack-induced debonding (or simply intermediate crack debonding or IC debonding). Similarly where a flexural crack is emphasized, intermediate flexural crack debonding is referred to as IFC debonding.

Plate end debonding may also occur in the form of critical diagonal crack (CDC) debonding Figure 2(b) (Oehlers and Seracino, 2004). A combination of CDC debonding with concrete cover separation is also possible Figure 2(c) (Smith and Teng, 2003). CDC debonding is induced by the formation of major shear crack intersecting the plate near a plate end and propagates from the point of intersection to the plate end along the plate- beam interface. CDC debonding failure is caused by insufficient shear reinforcement and is preceded by large shear crack opening in the anchorage region (Piotr and Björn, 2009). Concrete cover separation failure shows tearing-off of the concrete cover along the level of the steel tension reinforcement starting from a plate end. Plate end interfacial debonding also starts at plate end and propagates along the plate–beam interface. In all those debonding failure modes, failure occurs in the concrete, either adjacent to the adhesive layer (interfacial debonding) or at the level of the steel tension reinforcement (cover separation), showing that the plate is bonded to the beam in an appropriate manner with a strong adhesive. When the plate end distance is very small (distance between a plate end and the adjacent beam support), governing CDC may form, causing a CDC debonding failure Figure 2(b).

With the increase of the plate end distance, the CDC may fall outside the plated region which leads to concrete cover separation Figure 1(d). With these two modes, another combined mode of CDC debonding and concrete cover separation Figure. 2(c) may also occur. As the plate end moves further away from the support, the cover separation mode remains the controlling mode, and the plate end crack becomes increasingly vertical (Smith and Teng, 2003).

RC beams strengthened with a tension face steel plate, due to the much greater stiffness of the steel plate, IC debonding was not found in the laboratory test (Teng et al., 2003). Thus for steel plated beams, plate end debonding failures are much more likely to occur. In case of FRP plates, strength to stiffness ratio is much higher than that of steel plates. As a result, the FRP plate/sheet used for a particular strengthening application is generally much thinner or softer than corresponding steel plate of equivalent total tensile capacity and is much likely to debond at plate ends. The IC debonding failure mode is believed to be particularly important for relatively slender members and members strengthened with a relatively thin FRP plate/sheet. The mechanism of IC debonding is related to the formation of critical flexural crack at the tensile side of the concrete elements (Ombres, 2010). This critical flexural crack generally occurs at the section of maximum bending moment or where both the bending moment and shear force are higher (Yao et al., 2005). When a critical flexural crack is formed in the concrete, debonding initiates and propagates toward plate end (Teng et al., 2003). The tensile stresses released by the cracked concrete are transferred to the FRP plate; consequently high local interfacial stresses between the FRP plate and the concrete are induced near the crack. Further increase of the applied loading produces an increase both in the tensile stress in the plate and in the shear stress at the interface FRP–to-concrete near the crack. When the interfacial stress achieves the critical value, debonding initiates and then it self-propagates away from the crack (Sebastian, 2001).

There are two types of IC debonding failures, in the first one debonding occurs in presence of a single flexural crack; no other crack exists between the free end of the FRP plate and the crack where debonding initiates. In the second one the debonding occurs in presence of multiple cracks. In this situation the debonding propagation from the initiation crack to the adjacent crack is governed by the FRP tensile force distribution at both cracked sections. In addition, a succession of FRP plate debonding between adjacent cracks can occurs simultaneously with a sudden failure of the beams (Ombres, 2010).

Experiments on RC beams bonded with steel plates (Jones et al., 1982; Jones et al., 1988; Swamy et al., 1989; Oehlers, 1992; Hussain et al., 1995) have revealed that debonding of the soffits plate from the RC beam, typically with the concrete cover attached to the plate, is a common failure mode in these beams. This bonding failure of strengthened RC beams has been known as a typical case of brittle failure and indicates high interfacial shear or normal stresses caused by transfer of the tensile stresses from the bonded plate to the RC beam. The determination of interfacial stresses has been researched and several closed-form analytical solutions have been proposed (Smith and Teng, 2001; Teng et al., 2002; Ye, 2001; Adhikary et al., 2000; Raoof et al., 2000). MacDonald and Calder (1982) studied the behaviour of concrete beams externally reinforced with steel plates bonded to their tension flanges. Hamoush and Ahmad (1990) used the finite element method to predict the failure by interface debonding of the steel plate and the adhesive layer as a result of interfacial shear stresses. Swamy et al. (1987) investigated the effect of glued steel plates on the first cracking load, cracking behaviour, deformation, serviceability, and ultimate strength of RC beams. Some researchers (Saadatmanesh and Ehsani, 1991; Almusallam and Al-Salloum, 2001; Sevuk andArslan, 2005) indicated that bonded steel plates to the beam web can substantially increase flexural stiffness, reduce cracking and structural deformations at all load levels and contribute to the ultimate flexural capacity. It is the fact that gains in strength and stiffness are usually associated with a decrease in ductility. The ultimate and cracking load of the retrofitted RC beams with bonded steel plate depend principally on the compressive strength of concrete (f_c), nominal strength of web reinforcement, the yield strength of longitudinal bars, the tensile reinforcement ratio, shear span to depth ratio (a/d), the strength of steel plates, the area of steel plates, the anchorage lengths of steel plates, mechanical properties of epoxy adhesive and friction coefficient between steel plate and concrete. The bonding of continuous horizontal steel plates to the beam web is one convenient and effective method of enhancing the flexural strength of RC beams.

The plate bonding technique is becoming preferable for strengthening due to several advantages such as easy construction work, and minimum change in the overall size of the structure after plate bonding. The disadvantage of this method, however, is the danger of corrosion at the adhesive–steel interface, which adversely affects the bond strength (Sevuk and Arslan, 2005).

Methods of eliminating premature failures

Those failure modes previously discussed are undesirable because the strength of FRP/steel plate cannot be fully utilized; In addition, such premature failures are generally associated with a reduction in deformability of the strengthened members (Mander et al., 1988).

Debonding failures depend largely on the interfacial shear and normal stresses (Smith and Teng, 2001) between the beam and bonded plate. The determination of interfacial stresses has thus been researched for the last decades for beam bonded with FRP or steel plates. All of the solutions are based on the assumption that the interfacial stresses do not vary across the adhesive layer thickness. The solutions of Roberts (1989) and Malek et al. (1998) which are generally in terms of loading are the more appropriate among the available solutions and they give results almost similar to each other. This solution covers all the three common load cases (Single point load, double point load and UDL), and is based on more direct and simpler approach of deformation compatibility. On the basis of observation, Yang et al. (2007) proposed a simplified approximate solution by omitting some numerically minor terms in the rigorous solution (Yang et al., 2004). This simplified solution eliminates the complexity of the original one and is suitable for engineering applications with the aid of a portable calculator. By comparing with the rigorous solution, other approximate solutions and experimental results, the simplified solutions provide satisfactory predictions to the interfacial shear stress in the plated beams for symmetric loads. In the final part of that paper, extensive parametric studies were undertaken by using the simplified solution for strengthened beams with various ratios of design parameters. Observations were made based on the numerical results concerning their possible implications to practical designs. The simplified solution to the interfacial shear stress in the FRP-plated RC beams can be further exploited to develop a design method to predict the first debonding crack load. To this end, appropriate calibrations with adequate experimental results and field test data should be carried out using the reliability analysis.

Shear-bending interaction in predicting plate end debonding was first considered by Oehlers (1992) for steel-plated RC beams using a linear interaction curve which was later reviewed and assessed by Smith and Teng (2002a; 2002b). In recent years, some additional models have been published, Colotti et al. (2004) model covering debonding as well as other failure modes, Gao et al. (2005) model for concrete cover separation failures, and Oehlers et al. (2005, 2004) model for CDC debonding failures, which has classified all plate end debonding failures in a high-shear low-moment region as CDC debonding failures. This model should be interpreted to cover the debonding failure of a plate end, under the combination of a high shear force and a low moment, in all three debonding modes shown in Figure 1(d) and 2(b)-(c). For this reason, this model is called shear debonding strength model. Later Teng and Yao (2007) developed an accurate plate end debonding strength model for FRP-plated RC beams covering all possible plate end debonding failure modes. This model can be used for predicting debonding failure loads for the following two extreme cases:

(a) Shear debonding and
(b) Flexural debonding.

Several analytical models for IC debonding have been developed recently. Empirical models, which can be found in ACI 440 (2002), Teng et al. (2003), Lu et al. (2007) or Said and Wu (2007) provided solution straight forward and very convenient to use due to their simplicity. The design done according to these models leads to reduction of allowable strain in the FRP to avoid debonding which is simple to apply. By contrast, JSCE (2001) limits the maximum stress gradient in the FRP plate, but two key parameters including the fracture energy and the crack width are incompletely defined and

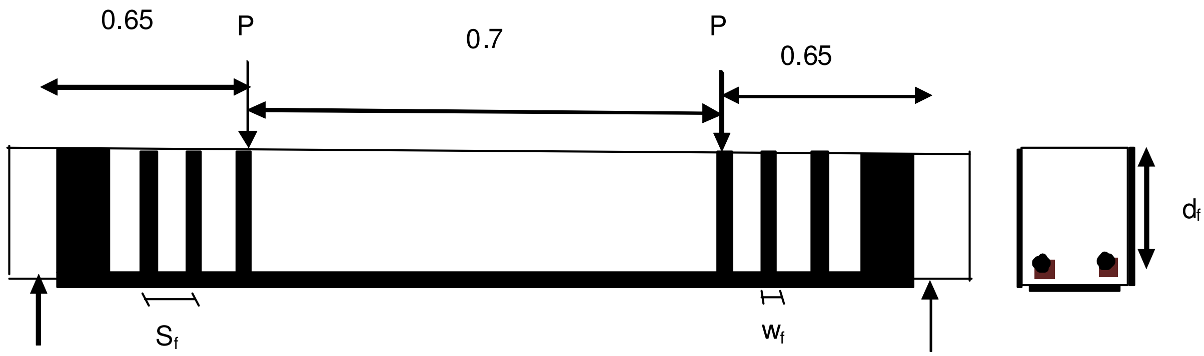

Figure 3. Strengthening scheme.

the method requires an involved iterative process of analysis. *fib* Bulletin (2001) provides three alternative approaches to avoid debonding failures by limiting (1) the FRP strain, (2) the maximum stress gradient in FRP plate, and (3) the shear force in the RC members (Figure 3). The first approach is similar to the strain limit approach of ACI 440 (2002) and suggests a FRP strain limit in the range of 0.0065-0.0085, which is much higher than results obtained from existing test results: further no specific value is suggested for a particular situation. The second approach is similar to JSCE (2001) approach but is more complicated for practical application. The third approach is suitable for application in design in terms of simplicity. Yao et al. (2005) assessed the above mentioned model using own test data and concluded that (1) IC debonding strength model of Teng et al. (2003) generally provides safe prediction of the experimental debonding strains but it becomes overly conservative and the scatter of its prediction is large, (2) the models of ACI 440 (2002), JSCE (2001) and *fib* Bulletin (2001) are not sufficiently safe for use in design. In addition, JSCE (2001) suffers from the tediousness of an iterative process of analysis.Teng et al., (2006) proposed a simple model to investigate the behaviour of the FRP-to-concrete interface between two adjacent cracks in flexurally strengthened reinforced concrete beams. It is an analytical solution, in which a bi-linear local bond-slip model is employed to predict the entire debonding process under various load combinations.

Smith and Gravina (2007) proposed a local deformation model for the analysis, considering local flexural deformations in a determinate structural flexural member at all stages of loading, from progressive formation of individual cracks up to initiation of the IC debonding failure. A simplified FRP-to-concrete bond-slip is used to determine the onset of debonding. Debonding occurs when a large slip has been reached such that minimal bond stress is present over one cracked block region and slip is predominantly in one direction. Wu and Niu (2000, 2007) have carried out several studies on the IC debonding failure and proposed an analytical model for predicting the debonding failure load assuming that cracks are smeared over the whole beam (smeared crack approach); consequently the debonding mechanism, in presence of multiple cracks, is similar to that of a single crack. The debonding failure load is assumed to be reached once the difference in magnitude between the FRP tensile forces over an equivalent transfer length exceeds the maximum transferable force in pull-off tests. The effectiveness of the Wu and Niu (2000, 2007) model was recently validated by Said and Wu (2008). The results demonstrate that debonding loads predicted by the model are much closer with experimental ones. The effects of variations in crack spacing and rate of change of moment on the IC debonding of plated members have been analysed by Liu et al. (2007). Rosenboom and Rizkalla (2007) proposed an analytical model which characterizes the interface shear stress based on two distinct sources, the change in the applied moment along the length of the member and stress concentrations at the intermediate cracks. Ombres (2010) proposed a theoretical non-linear model derived from a cracking analysis, founded on slip and bond stresses (Aiello and Ombres, 2004), and is adopted for the analysis of the debonding induced from intermediate flexural cracks in FRP-strengthened reinforced concrete beams. Analytical relationships of the bond-slip laws at the interfaces FRP-to-concrete and steel reinforcement-to-concrete are used. Through the model the strains and stresses in the concrete element for any loading level can be evaluated. The IC debonding occurs when the strains and stresses in the cracked element reach the values that correspond to the failure condition at the interface FRP-to-concrete that is the slip between the FRP and concrete reaches a critical value which causes separation of the FRP from the concrete. Through analysing the available experimental data, it is found that the proposed model furnishes good predictions of the intermediate debonding loads.

In comparison with experimental data, intermediate debonding FRP strain values predicted by the proposed model are conservative; however, in some cases, they are more accurate than that predicted by others models usually adopted for the analysis of the IC debonding failure. Results of a parametric study show that the proposed model allows to take into account the influence of several geometrical and mechanical parameters that are not considered in some current models.

Materials and methods used for shear strengthening

For strengthening and repair works several types of materials/methods can be used such as: Ferro-cement laminate (Paramasivam et al., 1997; Roumaldi, 1987; Iorns, 1987; Nassif and Najim, 2004), sprayed concrete (Diab, 1998; Taljsten, 2003), steel plate (Adhikary and Mutsuyoshi, 2006a,b; Adhikary et al., 2000; Costa and Barros, 2010; Barnes et al., 2001; Sinan et al., 2005), carbon fiber reinforced polymer (CFRP) laminate (Barros et al., 2007; Ozgur, 2008; Taljsten, 2003; Taljsten and Elfgren, 2000; Dias and Barros, 2010; Ozgur, 2006; Omar et al., 2001) and glass fiber reinforced polymer (GFRP) (Sundarraja and Rajamohan, 2009; Sundarraja et al., 2008).

CFRP and steel plates/strips are the most popular materials for strengthening. There have been a series of studies in the past for shear strengthening of RC beams using various techniques (Taljsten, 2003) such as follows:

1) Sprayed concrete method. In this method existing concrete on the top of the slab is removed, new stirrups are mounted around the existing cross section and new concrete is cast or sprayed onto the structure. If adhesion between the new and old concrete can be assured, this method is good from a technical standpoint. A new wider section with the steel reinforcement anchored in the compressive zone will give a higher shear capacity for the structure. However, the method is both time consuming and in many cases not cost effective.
2) Using steel tendons, either prestressed or non- pre-stressed. This method has also been used frequently for shear strengthening in the past. This method is a little bit easier to carryout than the previous method, but there is a risk that the bending reinforcement is cut off during drilling of the holes through the slab. Concrete casting above the bolts on the slab may also be necessary.
3) Shotcreate with steel fiber. This method can be used in the cases where a limited shear strengthening effect is required for. A drawback with this method is that the strengthening material is not anchored in the compressive zone.
4) Using pre-stressed steel straps. In this method steel straps have been wrapped around the section. Only a small amount of damage is needed in the cross-section. However, the straps are quite sensitive to impact loads or vandalism. If no recess is made in the slab, concrete casting or other overlays may be needed.
5) Using external bonded steel plates. Narrow, wide with or without openings, L-shaped steel plates or straps can be externally bonded with epoxy to increase the shear capacity of beam. This method is highly effective if roughening and cleaning of surface is ensured before the application of epoxy resin.
6) Strengthening using carbon fibre reinforced polymer (CFRP). Strengthening with CFRP materials can be used in various manners and orientations like side bonding, wrapping, U-jacketing and near surface mounted techniques (NSM). NSM is one of the most recent and promising strengthening techniques for concrete (RC) structures. NSM is based on the use of circular or rectangular cross section bars of carbon or glass fiber reinforced polymer (CFRP or GFRP) materials installed into pre-cut slits opened on the concrete cover of the elements to strengthen. NSM requires no surface preparation work and, after cutting the slit, requires installation of CFRP rod or sheet. As previously stated the major problem concerning this technique is making uniform groove in the beam side and proper fixation of CFRP rods or laminates into the grooves.

Previous work on enhancing shear capacity of flexural strengthened RC beams

Shear strengthening often forms a key part to eliminate premature shear failure of flexurally strengthen RC beams. Previously very limited work was found for shear strengthening to eliminate premature shear failure of flexurally strengthened RC beams.

Aprile and Benedetti (2004) proposed coupled flexural shear design of RC beams strengthened with FRP. It was noted that increase in the external reinforcement area does not always lead to the expected increase of the beam load capacity, due to the interaction of flexural and shear behaviour within the discontinuity regions of the strengthened element. This paper includes theoretical explanation of proposed truss model and comparison with other researcher's experimental results. Shear strengthening strategy was considered with FRP strips only two sides of the beam web.

To ensure flexural mode of failure it is desirable to prevent the debonding of flexural plate attached to the beam soffit. But using CFRP strips on the sides only may leads to the debonding of soffit FRP plate which was not taken into attention in this proposed model.

Toutanji et al. (2006) investigated the effect of multiple layers of CFRP sheets on contribution of flexural strength and different modes of failure. Beams strengthened with CFRP sheets at tension face, had much higher flexural capacity and were actually over reinforced. The design restriction for shear was considered and 50 mm wide CFRP strips were bonded at both sides oriented in 45°C. Due application of CFRP strips at sides only debonding failure was observed for the tension face CFRP plate.

Rasheed and Pervaiz (2003) compared experimental results of other researchers and concluded that there is a lower bound level for the FRP interfacial shear stress in the yield beam region upon plate separation failure. Externally U-wraps required preventing of premature plate separation when the interfacial shear stress exceeds this level.

Sundarraja and Rajamohan (2009) proved that for the beams bonded with inclined GFRP U-strips flexural failure was prominent than shear failure which avoids catastrophic failure of beams.

Challal et al. (1998) proposed design approaches for flexural and shear strengthening, but application and experimental prove to eliminate premature shear failure is seldom found.

Proposed design method for eliminating premature shear failure of flexurally strengthened RC beams using CFRP strips

The following design equations are proposed by ACI 440 (2002) for shear strengthening of RC beams:

Shear contribution required by CFRP strips

$$V_f = \frac{V_d - (V_c + V_s)}{\Phi\psi} \quad (1)$$

V_d = Ultimate shear force on the beam,
V_c = Shear resisted by Concrete
V_s = Shear resisted by stirrup

$$= 2 \times A_s' \times \frac{d - d'}{s} \times f_y' \quad (2)$$

(Alam, 2010)
Φ, ψ = reduction factor 0.85 and 0.95 respectively
Shear contribution by CFRP strips,

$$V_f = \frac{A_f E_f \varepsilon_{fe} d_f}{s_f} \quad (3)$$

Where A_f = Area of CFRP strip = $2 \times$ width $\times$ thickness $= 2 \times w_f t_f$, d_f = effective depth

s_f = center to center spacing of CFRP strip $< w_f + \frac{d_f}{4}$,
ε_{fe} = effective strain of CFRP
E_f = Modulus of elasticity
$\varepsilon_{fe} = 0.004 \leq 0.75\, \varepsilon_{fu}$,
for completely wrapped beams or

$$\varepsilon_{fe} = K_v \varepsilon_{fu}, \text{for beams with two or three sides laminated} \quad (4)$$

$$\text{Where, } k_v = \frac{k_1 k_2 L_e}{11900\, \varepsilon_{fu}}, \quad (5)$$

$$L_e = \frac{23300}{(n t_f E_{frp})^{0.58}}, \quad (6)$$

n is the number of sheets, t_f is the thickness of sheet.

$$k_1 = (f_c'/27)^{2/3} \quad (7)$$

$$k_2 = \frac{d_f - L_e}{d_f} \text{ for U-wraps or}$$

$$k_2 = \frac{d_f - 2L_e}{d_f} \quad (8)$$

This design guideline can be used for designing CFRP strips to eliminate premature shear failure of flexurally strengthened RC beams. A design example is worked out in the appendix using these design recommendations and BS8110-1 (1997).

CONCLUSION

The summary of the failure modes of flexurally strengthened RC beams can be classified as:

(1) Premature debonding failure which can be plate end debonding or end peeling (beams failed just after or after reaching tensile reinforcements yield strength without showing any ductility value)
(2) Tension delamination at midspan and
(3) Premature shear failure due to insufficient shear reinforcement.

End peeling occurs due to shear and normal stress at the end of the plate which can be minimized by using proper end anchor. Tension delamination occurs when a plate spans across flexural or shear crack, it can be seen that wherever a flexural crack touches the plate, a debonding crack would form along the edge of the plate. Researchers have found that debonding in the mid-span can be minimized by controlling the strain of CFRP laminate. Though researchers have found a solution to eliminate end peeling using appropriate end anchor, research on tension delamination and premature shear failure of flexurally reinforced strengthened RC beams is still limited.

In order to utilize the full flexural capacity of strengthened beams all of the stated failure modes should be prevented. Sometimes premature shear failure becomes crucial when the shear reinforcement present is no longer sufficient due to the increase in flexural capacity which increases shear forces. Hence, this excess shear must be accounted for to ensure that the strengthened beam does not fail by premature shear before attaining its full flexural capacity. A design example is worked and given in the appendix.

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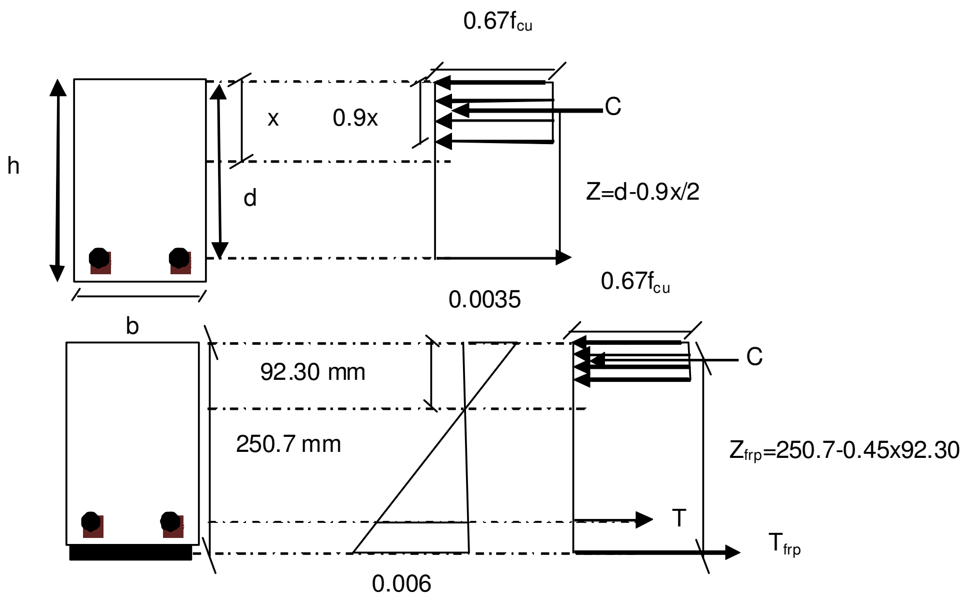
APPENDIX

Material properties:

Concrete	Main reinforcement	Shear reinforcement	CFRP
$f_{cu} = 38.0$ Mpa	$f_y = 551$ Mpa	$f_y{}' = 350$ Mpa	$\varepsilon_{fu} = 0.012$
$\varepsilon_{cu} = 0.0035$	$\varepsilon_u = 0.002$	$\varepsilon_u = 0.002$	$E_{frp} = 165$ Gpa



Let us consider a singly reinforced concrete beam of 125 mm wide and 250 mm deep, which is required to span 2.0 m between centers of supports. The longitudinal reinforcement of the beam consisting of 2φ12 mm and the shear reinforcement consists of 6 mm-diameter links at 160.0 mm c/c. The nominal cover was 37 mm. The beam was originally designed to carry a moment of 24.0 kN-m. Now it is required to carry a moment of 58.0 kN-m. To achieve this externally bonded CFRP plate will be attached to the soffit of the beam. ACI 440.2R-02 (2002) is used to design of external CFRP shear strips.

For flexural strengthening in literature maximum allowable strain 0.006 (Arya et al., 2002) is suggested to prevent FRP rupture and debonding where higher shear force and bending moment are present. Here, maximum allowable strain 0.006 is used for design.

$A_s = 2\Phi 12\text{mm} = 226.19\ \text{mm}^2, b = 125\ \text{mm},$
$h = 250\ \text{mm}, d = d_f = 213\ \text{mm}$

Now, Tensile force $T = A_s f_y = 226.19 \times 551 = 124.63$ kN
Compressive force, $C = 0.67 f_{cu} \times b \times 0.9x$ (9)

In case of equilibrium condition $T = C$

$$x = \frac{124.63 \times 10^3}{0.67 \times 38 \times 125 \times 0.9} = 43.5\ \text{mm}$$

Moment capacity of the original section $M = T \times Z$

$$= A_s f_y \times Z = A_s f_y \times (d - 0.45 \times x)$$
$$= 226.19 \times 551(213 - 0.45 \times 43.5)$$
$$= 24.1\ \text{kN} - \text{m}$$

$$\text{Maximum Shear force on the original beam} = \frac{M}{\text{shear span}}$$
$$= \frac{24.1}{0.65} = 37.0\ \text{kN}$$

Moment contribution required by CFRP plate,

$$M_{frp} = M_{required} - M_{original} = 58.0 - 24.1 = 33.9\ \text{kN} - \text{m}$$

$$\text{Area of CFRP required,} \quad A_{frp} = \frac{M_{frp}}{E_{frp} \times \varepsilon_{frp} \times Z_{frp}}$$

$$= \frac{33.9 \times 10^6}{165 \times 10^3 \times 0.006 \times (250.7 - 0.45 \times 92.3)} = 163\ \text{mm}^2$$

(Arya et al., 2002)

use $A_{frp} = 168\ mm^2\ (1.4mm \times 120mm)$

Moment capacity of strengthened beam,

$M_{strengthened} = M_{original} + M_{frp} =$
$24.1 + 168 \times 165 \times 0.006 \times (250.7 - 0.45 \times 92.3) \times 10^{-3}$
$= 58.88\ kN - m$

Maximum compressive force resisted by concrete,

$$C = 0.67 f_{cu}\, b \times 0.9x \quad \left(\text{When } x = \frac{d}{2}\right)$$
$$= 0.67 \times 38 \times 125 \times 0.9 \times 106.5 = 305.0\ kN$$

Total tensile force $= T + T_{frp}$
$= 124.63 + 168 \times 165 \times 0.006 = 290.95\ kN$

Hence, Total compressive force> Total Tensile force, ok
Shear capacity of the original beam:
Concrete shear stress of the original section

$$v_c = 0.79\left(\frac{100 \times A_s}{b_v \times d}\right)^{\frac{1}{3}}\left(\frac{400}{d}\right)^{\frac{1}{4}}\left(\frac{f_{cu}}{25}\right)^{\frac{1}{3}} \ldots\ldots\ldots(10)$$

(According to BS8110 – 1(1997))
$$= 0.79\left(\frac{100 \times 226.19}{125 \times 213}\right)^{1/3}\left(\frac{400}{213}\right)^{1/4}\left(\frac{38}{25}\right)^{1/3} = 1.00\ N/mm^2$$

Concrete shear capacity of the original section V_c

$= v_c \times b \times d = 1.0 \times 125 \times 213 \times 10^{-3} = 26.63\ kN$

Stirrup contribution to the shear capacity of the original section:

6 mm diameter stirrup was used @160 mm c/c
Using Equation 2

$$V_s = 2 \times A_s' \times \frac{d - d'}{s} \times f_y'$$

$$= 2 \times 28.27 \times \frac{176}{160}(= 1) \times 350 = 19.79\ kN$$

Total shear capacity of the original section $= V_c + V_s$
$= 26.63 + 19.79 = 46.42\ kN$

>Maximum shear force on the original beam (37.0 kN)

$$\text{Maximum shear force on strengthen beam} = \frac{M_{strengthened}}{\text{shear span}}$$

$$= \frac{58.88}{0.65} = 90.5\ kN$$

Excess shear $= (90.5 - 46.42) = 44.0\ kN$

Hence, internal stirrups are not sufficient for the shear force on the strengthened beam

Design of CFRP strips based on ACI 440 (2002) to eliminate premature shear failure:

According to design equation (1), Shear contribution required by CFRP strips,

$$V_f = \frac{\text{Excess shear}}{\Phi\psi} = \frac{44.0}{0.85 \times 0.95} = 54.5\ kN$$

Using equation (6), $L_e = \dfrac{23300}{(nt_f E_{frp})^{0.58}}$

$$= \frac{23300}{(1 x 1.4 \times 165 x 10^3)^{0.58}} = 18.0\ mm$$

Using equation (7), $k_1 = (f_c'/27)^{2/3} = (38/27)^{2/3} = 1.26$

From equation (8), $k_2 = \dfrac{d_f - 2L_e}{d_f} = \dfrac{213 - 2 \times 18.0}{213}$
$= 0.83$, for two sides laminated

Now from the equation (5), $k_v = \dfrac{k_1 k_2 L_e}{11900\, \varepsilon_{fu}}$

$$= \frac{1.26 \times 0.83 \times 18.0}{11900 \times 0.012} = 0.14$$

The effective strain ε_{fe} in FRP is assumed to be smaller than the ultimate strain $\varepsilon_{fu} = 0.012$.
This can be computed putting value of k_v and ε_{fu} in equation (4)
From the equation (4),

$$\varepsilon_{fe} = K_v \varepsilon_{fu} = 0.14 \times 0.012 = 0.0016 < 0.004$$

Now from the equation (3),

$$\text{Shear contribution by CFRP strips, } V_f = \frac{A_f E_f \varepsilon_{fe} d_f}{s_f}$$

$$= \frac{2 \times w_f t_f \times 165 \times 10^3 \times 0.0016 \times 213 \times 10^{-3}}{s_f}$$

$$= \frac{112.4\, w_f t_f}{s_f}$$

if we use $w_f = 30$ mm, $t_f = 1.4$ mm,
$s_f = 85$ mm and putting values in this equation
Shear contribution by CFRP strips, $V_f = 55.5\ kN > 54.5\ kN$.