Restoration and strengthening with fibre reinforced polymers: issues to consider

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ABSTRACT: Fibre Reinforced Polymers (FRP's) have the advantages of light-weight, high strength and excellent durability which make them attractive for use in certain types of rehabilitation. However, there are disadvantages in terms of performance under heat and ultraviolet which need to be considered, and there is not much known about the long term bonding properties. Additionally, features peculiar to the use of these materials need to be considered – a new mode of failure observed in reinforced beams, the modes of failure of FRP – wrapped columns and time dependent effects, including relaxation of the FRP due to flow of the bonding epoxy under shear.

1 INTRODUCTION

Fibre Reinforced Polymers (FRP’s) constitute a set of advanced composite materials that have been considered in recent years for various applications in the construction industry. While there have been applications in new structures – some made entirely of FRP’s (eg: Harvey 1993, Cooper 2004) – considerable effort has been placed on their use for rehabilitation and/or strengthening of reinforced concrete structures (ISIS Canada 2001b, FIB 2001). In comparison, much less work has been performed with masonry (Lisessel & Gayeyov 2003). With the latter, there has been some consideration of the potential use of FRP’s in the restoration of historical structures.

There are some distinct advantages that FRP’s would bring to a restoration project. Unidirectional FRP’s in which the fibres (the ones of main interest are Glass (GFRP) and Carbon (CFRP)) are aligned in the single direction during manufacture of the product, are very strong. (Some CFRP sheets, strips and rods have ultimate tensile strengths in the order of 3500 MPa.) The products are also very light and thus add little weight to the structure, potentially negating the need to strengthen supporting members, and having negligible effect on the inertial response in an earthquake. FRP’s do not corrode like steel, and thus promise durability and longevity well beyond our expectations of unprotected steel. In order to achieve that durability, FRP’s must currently be protected from ultraviolet light, which degrades the polymer matrix in which the fibres are embedded. New binders are being developed which are expected not to suffer from this problem.

A second area of development is in the stress–strain behaviour of these materials. Currently they are essentially linear elastic to failure, with little ductility. Combinations of fibres are now being investigated to produce materials with non-linear behaviour and much greater strain capacity. The lack of ductility has led to reconsideration of the desired failure mode at ultimate. Whereas with steel, the aim is to ensure the steel yields before the concrete or masonry crushes; with FRP’s it is the other way round. Sudden rupture of the FRP is to be avoided and warning of failure is to be obtained by producing relatively large deformations from stable crushing of the compression material (ISIS Canada 2001a).

In our view, there are additional issues which need to be considered when using FRP’s in rehabilitation projects. We will touch briefly on the effects of heat and the durability of the bond between the FRP and
There has been some study of the durability of the bond between the FRP and the underlying substrate concrete or masonry (e.g. Green et al. 2000). Initially, moisture alone could cause degradation: the effects of freezing and thawing have also been examined. Experiments are underway to study the effects of exposure under different climate conditions of FRP's bonded to concrete (Labossiere et al. 2003). What is of interest is an accelerated test that simulates the long-term effects correctly. Debonding caused by environmental factors can not be afforded if FRP's are to live up to their potential as a long-term solution for strengthening and rehabilitation.

4 FAILURE MODES AND STRENGTH DESIGN

4.1 General

Estimates of how much strength gain can be achieved by a particular application of FRP's are based on an assumed mode of failure. The potential modes of failure are established from observations of tests. However, it is not always clear what sequence of events is that leads to failure, and therefore what the limiting design criterion should be. For example, the failure of masonry arches reinforced with FRP's bonded to the intrados or extrados can be induced by one of four events – crushing of the masonry in compression, sliding between the units and the mortar due to shear, rupture of the FRP in tension, or debonding of the FRP if a mechanism develops in the arch (Briccoli Bati & Rovero 2000, Valluzzi et al. 2001). Which mechanism occurs depends on the material properties, the relative amounts of masonry and reinforcement, the geometry of the arch and the loading conditions. Models of the system can be used to assess which is the most likely mode of failure (Briccoli Bati & Rovero 2001), and adjustments made to the levels of reinforcement if further refinement of the strength gain or type of failure (ductile/brittle) is desired (Valluzzi et al. 2001).

Similarly with masonry walls: considerable (thirty fold) increases in strength can be achieved, as well as large increases in deformations (Albert et al. 2001) through the application of externally bonded FRP. The modes of failure have been described and models have been developed to predict them (Kuzik et al. 2003). However, difficulties and inaccuracies can arise with strengthening if the actual mode of failure is not recognized. Two examples involving the addition of externally bonded FRPs are given below.

4.2 FRP strengthened RC beams

A substantial amount of work has been performed on strengthening reinforced concrete beams through the application of FRP strips to the soffits of the beams. This will provide increased flexural strength. Shear strength can be improved by the application of FRP sheets to the sides of beams. Design equations have been developed (eg. ISIS 2001b, FIB 2001) which can provide reasonable estimates of the increases which can be expected, so long as the failure mode is as

![Figure 1. Loss of strength of CFRP (Leadline) with temperature (Sayed-Ahmed & Shrive 1999).](image-url)
expected. One such mode in flexural strengthening is debonding of the laminate from its ends, typically near the support of the beams. (When a FRP strip is applied to the underside of the beam in situ, to strengthen the beam, it is usual to take the strip up to the support, but not lift the beam so the FRP can be taken in under the support.) Debonding occurs when the shear transfer to the strip is greater than can be carried by the bond between the strip and the concrete. Anchorage of the strip by additional means is required.

However, mid-span shear debonding has recently come to light as a new mode of failure (Riad et al. 1998, Swamy et al. 1999, Sayed-Ahmed et al. 2004), Figure 2. In this case, debonding of the FRP strip begins under a load point near mid-span, at a load lower than expected and predicted for flexural failure (11% gain in strength vs. 25% predicted (Sayed-Ahmed et al. 2004)). The crack is initiated in the high moment plus shear zone at the edge of the high constant moment zone (in four-point bending). The debonding progresses from under the load-point back toward the support – the reverse of the direction of the other debonding mode. This new mode of failure appears to be a result of insufficient strength in shear in the concrete between the flexural steel reinforcement and the FRP strip (Bakay 2003). As the beam is loaded initially, strain compatibility occurs between the concrete in compression and the steel and FRP in tension. The loads carried in each material can be determined. Once the steel yields, any further increase in load causes an increase in the stress in the FRP only. Thus the concrete between the steel and the FRP has to carry all the additional shear from that which existed when the steel yields. The concrete beneath the steel reinforcement cracks in diagonal tension and the section of this concrete in the centre of the beams displaces downward relative to the concrete on the other side of the crack. With the FRP in tension, there is thus a component of force pulling it off the concrete (Figure 3), resulting in debonding. An analytic model of this sequence of events was able to predict the strengths observed for different beams with different concretes with acceptable agreement (Bakay 2003). The new mode of failure thus needed both to be recognized and understood before reasonably accurate predictions could be made of its occurrence. The same sequence can also develop from a higher starting point in the beam section if there is inadequate shear reinforcement above the flexural steel.

4.3 FRP wrapped columns

Wrapping of columns with FRP’s will only be a possible solution in historical structures for cases where the column is not publicly visible, or where the column is parged, and fresh parging on top of the wrapping is acceptable. Estimates of the increase in strength that can be achieved are based either on the FRP wrap reaching its ultimate strain (ISIS Canada 2001a), or an analysis of published results and curve fitting on the conservative side of those results (Spoelstra & Monti 1999). For the FRP to confine a column actively, the FRP needs to be applied prestressed. Otherwise the FRP relies on the column dilating near failure in order to be activated and resist further expansion of the column.

When a uniaxial fibre sheet is wrapped around a column, the direction of the fibres is circumferential, thus providing the maximum strength and stiffness to resist dilation of the column. When a wrapped, reinforced concrete column fails, a crack develops parallel to the direction of compression, splitting the fibres. The ultimate strain of the FRP has been reached across this split, but only locally (Pessiki et al. 2001, Scholefield 2003): the rest of the FRP is at strains well below ultimate. Failure occurs when the wrap rips circumferentially with cracks running between the fibres around the circumference of the column at both the top and the bottom of the longitudinal.
crack (Figure 4) (Scholefield 2003). Basing column strength on a global achievement of ultimate strain in the wrap, when in fact it is a local condition, leads to an observed inconsistency in strength prediction (Shrive et al. 2003). Recent evidence (Scholefield 2003) suggests that the spalling of the concrete is driven by buckling of the reinforcement, complicating the issue further. Such a failure can not occur in plain masonry columns and indeed a different behaviour is observed. In this case, the mortar joints crush and the units crack (longitudinally) into multiple pieces. The column becomes rubble, contained within the FRP surround. Large strains can be accommodated without rupture of the bag (Masia & Shrive 2003), large strains can also be achieved with FRP wrapped RC columns if the circumferential cracks are prevented from rapid propagation. This can be achieved by using multi-directional sprayed FRP or potentially by applying FRP sheets such that the fibres run circumferentially in the one layer and longitudinally in another (Scholefield 2003). The latter needs to be investigated.

The majority of tests to date have been performed on concentrically loaded columns. The effect of eccentricity might be expected to be quite drastic, with more rapid reduction in axial capacity with increasing moment than occurs with a normal reinforced column. The few tests that have been performed (Chaalal & Shahawy 2000, Parvin & Wang 2001, Li & Hada 2003) have shown failure develops through circumferential tensile splitting of the FRP on the longitudinal tensile face, and bursting of the concrete through the compression face. Bidirectional fabrics improved performance. Eccentricity in loading did reduce the confining effect on the columns.

There is thus a need to develop a method for predicting the strength increase that can be achieved based on the actual mode of failure. The strain capacity is clearly influenced significantly by the mode three fracture toughness of the wrap combination used. Thought needs to be given as to the strengthening that might be obtained with eccentrically loaded columns, and how that can be modeled and predicted.

5 CREEP

The effects of creep in structures are not widely understood. Consider for example a concentrically loaded reinforced concrete column. When load is applied, we can apply the conditions of compatibility and equilibrium to determine the stresses in the steel and the concrete. In time, the concrete creeps — wants to increase its compressive strain. This means that to maintain compatibility, the strain in the steel must also increase. In turn, this means that as long as the steel is in the elastic part of its idealized stress—strain behaviour, the stress in the steel will increase. To maintain equilibrium, the stress in the concrete must therefore decrease over time. The effect of creep in this case therefore, is that the deformation increases and the stresses redistribute (steel stress increases, concrete stress decreases) over time. These are the two usual consequences of creep in any structure.

If the reinforcement is not elastic, but also creeps, then the redistribution of stresses depends on the relative speeds and amounts of creep in the two materials. Stresses can both rise and fall in time (Reda Taha & Shrive 2003). The redistribution of stress due to creep has been recognized as the likely cause of the collapse of the Civic Tower of Pavia (Shrive & Huizer 1991, Binda et al. 1992), and can cause cracking to develop in structures over time. It is therefore important to understand what may happen in a structure that is rehabilitated with FRPs.

While Aramid and Glass FRP’s creep, Carbon FRP’s do not creep at normal service stresses. However, there is another source of potential time-dependent behaviour in the system — creep of the epoxy bonding agent between the FRP and the underlying structure. We have begun to examine the possible effect of this by loading two beams in flexure (Masia et al. 2004). The beams have the same internal steel reinforcement and were cast from the same batch of concrete. One of the beams had two CFRP strips glued to the tension side. The beams were loaded one after the other on the same day in four-point bending. As expected, the central deflection of the FRP reinforced beam was much lower than that of the beam without the strips. The central displacements of the beams have been monitored over time and are shown in Figure 5. Both beams creep as expected. In the beam without the FRP strips, the concrete creeps which increases the curvature and the deflection. The increased curvature increases the stress in the steel. This in turn requires an increase in concrete force to maintain zero total axial force. However, the moment is unchanged so the lever arm must decrease. Thus the concrete stress decreases, but
the area of concrete which is loaded increases as the neutral axis moves down in the beam and the lever arm is thus reduced. The effective modulus of the concrete is thus decreasing over time and the second moment of area of the transformed section is also a function of time.

The same sequence of events happens in the beam with FRP strips. The presence of the strips makes the beam stiffer initially—sufficiently stiff so that the concrete did not crack in tension. As the curvature increases from the creep of the concrete the tensile strain increases and there comes a time when the concrete will begin to crack in the tension zone. This was observed in the tests: a flexural crack in the concrete was noticed several months after loading. This will cause a change in the second moment of area in addition to the shifting neutral axis. Another feature which adds complexity to the problem is relaxation of the FRP stress due to flow of the epoxy binding agent gluing the FRP to the beam. Shear occurs across the interface in order for tension to develop in the FRP. Such shear occurs in any shear zone of the beam, where the moment is changing. We attempted to measure slip of the FRP from this mechanism with simple spring gauges touching the ends of the strips. Some movement (off loading the FRP) was measured in the first few weeks after loading (Figure 6).

The creep of the beam was modeled using the approaches in the CEB-FIP model code (1990) and the ACI Committee 209 recommendations (ACI 1992), obtaining a best fit for the first beam (no FRP). Creep for the second beam (with FRP strips) was predicted, allowing for the increase in stiffness from the FRP but assuming no flow in the epoxy binding agent. The creep is underpredicted at longer ages (Figure 5). Whether this is due to the change in second moment of area (from uncracked to cracked section), to the relaxation of load in the FRP due to flow in the epoxy, or to a combination thereof is under analysis. Initial results indicate the effect is due to the combination of factors.

6 CONCLUSION

The use of FRP's in restoration and rehabilitation of historic structures is an attractive option. Large gains in strength can be achieved for some situations with minimal additional weight. The need to "hide" the FRP is an obvious consideration in any scheme, but there are other features that need to be considered in the full assessment of a scheme: the effects of creep and relaxation of the FRP through bond flow; the need for toughness in the FRP arrangement to reduce crack propagation and rapid failure; the need to ensure that the mode of failure is understood, and that all modes have been considered; and that suitable precautions are taken to obtain sufficient fire resistance and durability in the bond.

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REFERENCES


