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# A partial-interaction ductility model for FRP plated RC flexural members

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ABSTRACT: Structural engineers have long recognised that ductility is a major consideration in the safe design of reinforced concrete structure. FRP retrofitting is now a well established technique for strengthening reinforced concrete structures but because FRP is a brittle material this can lead to the misconception that FRP retrofitted structures behave in a brittle fashion which has restricted the use of FRP retrofitting. Furthermore, quantification of the ductility of FRP plated RC structures is much more difficult than the quantification of the ductility of unplated RC structures as the latter depends only on the limit of concrete crushing. Hence the necessity in FRP retrofitted RC members to fully quantify the rotational capacity for all limits. In this paper a novel ductility model for FRP internally or externally strengthened or FRP confined RC members is described which fully simulates the behaviour of flexural members all the way through to concrete softening and then member failure. The model simulates the rotation due to material non-linearity, flexural cracks and the occurrence of both disturbed and undisturbed regions. However and more importantly, the model fully simulates the behaviour of the hinge in the region where concrete softening occurs. It is shown that in the hinge the rotation is limited by concrete softening and that this softening limit can be derived directly from shear-friction theory. It is also shown that the rotation of the hinge is also limited by intermediate crack (IC) debonding of the FRP reinforcement and by yield-penetration failure of the steel reinforcing bars. All of these rotational limits have been quantified and the mechanisms used in the quantification is presented in this paper. This research will allow the expansion of FRP reinforcement into regions where ductility is required and hence considerably expand the use of FRP.

## 1 INTRODUCTION

Much of the initial research on FRP plating was on FRP pultruded plates adhesively bonded to the tension face of reinforced concrete (RC) beams or slabs. This form of retrofitting tends to debond at early strains (Oehlers and Seracino 2004, and Mohamed Ali M.S. et al 2006) and often debonds prior to yielding of the tension steel reinforcing bars and, consequently, prior to crushing of the concrete; this has led to the proviso in most guidelines that FRP retrofitted members should be treated as brittle  which may be true for externally bonded (EB) FRP pultruded tension face plates but it is certainly not the case with other types of FRP retrofitting.

Some examples of ductile member behaviour of FRP retrofitted reinforced concrete members are shown in Figs. 1 to 4  (Oehlers and Seracino 2004, Seracino et al 2007, Oehlers et al 2007a). Figure 1 shows the hogging region of a reinforced concrete beam that has been strengthened with a pair of near surface mounted FRP strips (Liu et al 2006). The herringbone formation of cracks associated with intermediate crack (IC) debonding prior to failure can be clearly seen, as well as the pronounced deformation of the beam, and also at the support can be seen the horizontal cracks associated with concrete crushing or softening. These are all signs of member ductility of a reinforced concrete beam that has been strengthened with brittle FRP reinforcement. Figure 2 shows the hogging region of a beam which has been strengthened with a single FRP NSM strip on each side of the beam and because these NSM strips are closer to the neutral axis it has allowed greater rotation and, hence, member ductility. A further example of ductility is shown in Fig. 3 where the concrete was wrapped with FRP. In the case of columns, FRP wrapping increases the ductility by confining the concrete. However in the case of beams, ductility is usually achieved because the thin FRP wrap allows high IC debonding strains. Finally, Fig.4 shows a reinforced concrete beam that has been strengthened by bolting FRP plates to the sides. Indications of member ductility are the large deformation as well as concrete crushing at mid-span which is due to the ductile nature of the bolt connection.

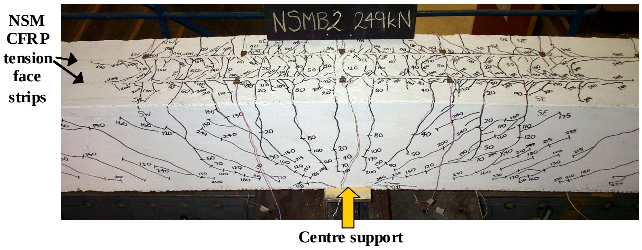


Figure 1. FRP NSM tension face plated beam

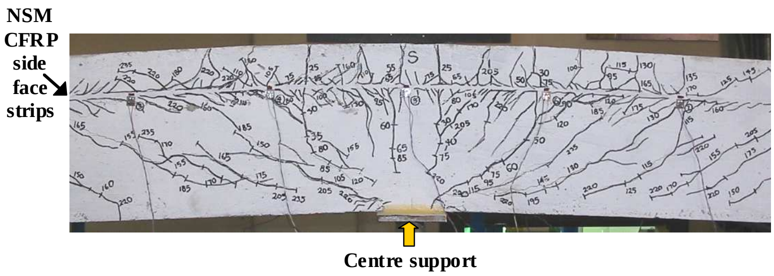


Figure 2. FRP NSM side face plated bam

Figure 3. FRP wrapped

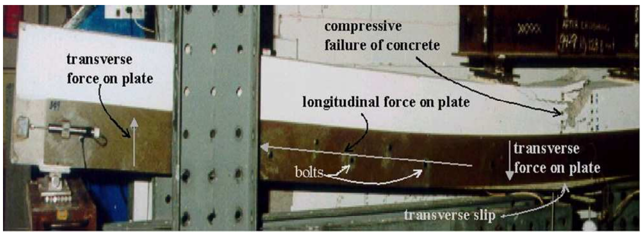


Figure 4. FRP plates bolted to RC beam

Quantifying the ductility of reinforced concrete members has been an ongoing problem for the last fifty years (Barnard and Johnson 1965 and Wood 1968). Progress has been very slow and often relied on empirically derived solutions (Baker, 1956, Sawyer 1964, Corley 1966, Mattock 1967, Priestley and Park 1987, and Panagiotakos and Fardis 2001) and it is only recently that mathematical models have been derived for quantifying the member ductility of unplated members (Fantilli et al 1998, and 2002, and Debernardi and Taliano 2002). It will be shown how this has led to a ductility model or ductility mechanism which is the subject of this paper and which has now been partially quantified (Oehlers et al 2005, Mohamed Ali, M.S. et al 2007, Haskett et al 2007, and Oehlers et al 2007b)

## 2. ROTATIONAL REGIONS OF AN FRP PLATED RC BEAM

A major reason for the difficulty in quantifying the member ductility of FRP plated reinforced concrete members is not the brittle nature of the FRP but because of the complex softening nature of the concrete which is illustrated in Fig. 5 (Oehlers et al 2007b). Path O-A is the ascending or first branch of the stress-strain relationship and is a material property. In contrast, path A-D-C is the descending or second branch, the start of which is given by $\sigma_{start}$ at $\varepsilon_{start}$, and which is a pseudo-material property that quantifies the shear-friction crushing wedge shown in Fig.4 and also in Fig.3 as well as in the eccentrically loaded prism in Fig. 6.

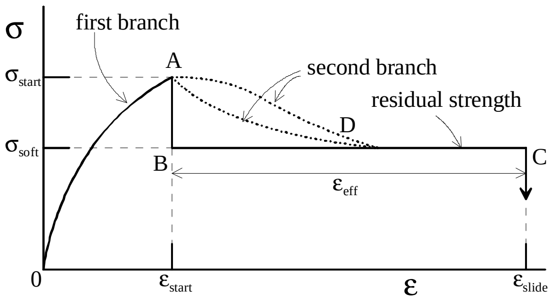


Figure 5. Stress-strain behaviour of concrete

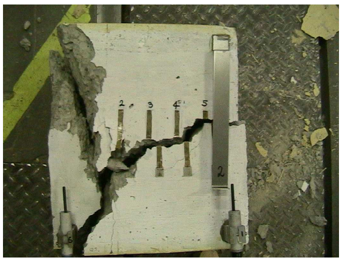

Figure 6. Failure shear-friction wedge

An FRP tension face plated beam is illustrated in Fig. 7 (Haskett et al 2007). This beam can be separated into two distinct regions: the non-hinge region, over the lengths z, where the concrete in compression is in the ascending or first branch of its stress/strain relationship O-A in Fig.5, as shown at the stress profile at D in Fig. 7; and the hinge region where the concrete is crushing or softening that is the concrete is in its descending or second branch of its stress/strain relationship A-D-C in Fig. 5, as shown at the stress profile at E in Fig. 7. The non-hinge region, in which the stress in the concrete is increasing with load, can be analysed through standard procedures of equilibrium or compatibility. In contrast, the hinge region cannot be analysed directly through equilibrium and compatibility because concrete softening prevents any numerical simulation from directly working (Barnard and Johnson 1965, Wood 1968, and Oehlers 2006)

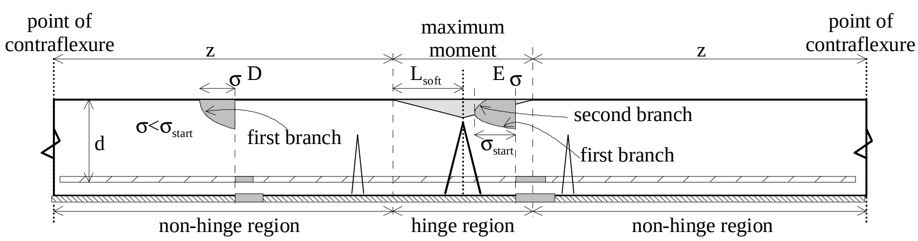


Figure 7. Rotational regions of FRP plated beam

## 3. NON-HINGE ROTATION

The behaviour of the non-hinge region in Fig. 7 is illustrated in Fig. 8 (Oehlers et al 2005, and Mohamed Ali M.S. et al 2007); the non-hinge region is defined as the region of the beam outside the concrete softening zone so that the concrete throughout the non-hinge region lies in the first branch O-A in Fig. 5. Standard numerical models such as finite element analyses or segmental layered analyses (Oehlers et al 2005, and Mohamed Ali M.S. et al 2007) can be applied. The behaviour is complex as rotation is affected by slip between the reinforcement and the concrete as shown by the bond characteristics in Fig. 9 where it can be seen that the bond characteristics can be idealised as bi-linear with a peak shear-stress of $\tau_{max}$ and a slip $\delta_{max}$ beyond which the interface shear stress is zero and that this characteristic can be applied to reinforcing bars, NSM strips and EB pultruded plates. The region where there is significant slip is referred to as the partial-interaction region in Fig. 8 such that the slip-strain ds/dx and the slip s are not zero. Beyond this region where the slip is minimal is referred to as the full-interaction region which in this case ds/dx = s = 0. The behaviour is further complicated by flexural cracking which requires interface slip and, furthermore, when the flexural cracks are closely spaced disturbed regions are formed where standard forms of compatibility cannot be applied. The rotation of the non-hinge region is particularly important when yielding of the steel reinforcement occurs before concrete softening as this will cause wide flexural cracks and subsequently large concentrations of rotation at the flexural cracks where yielding has occurred.

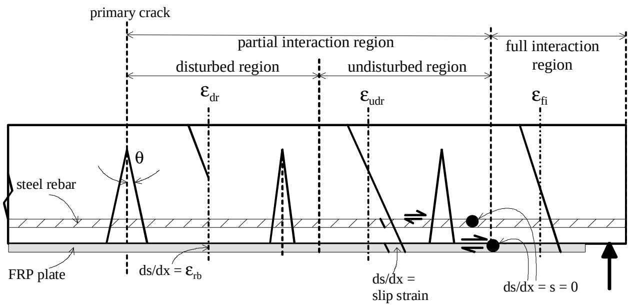


Figure 8. Partial-interaction non-hinge region

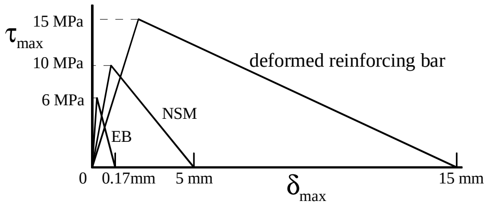


Figure 9. Reinforcement partial-interaction bond characteristics

## 4. HINGE ROTATION

The rotation of the hinge region in Fig. 7 is shown enlarged in Fig. 10 where it can be visualised as a rigid body rotation across a major crack (Mohamed Ali M.S. et al 2007). This rotation is limited by either concrete crushing in the softening zone or the rotation limit due to the slip capacity of the reinforcement $(\theta_{fract})_{limit}$. Partial-interaction intermediate crack (IC) debonding theory (Mohamed Ali M.S. et al 2006) can be applied directly to determine the slip at fracture of the FRP reinforcement or at IC debonding. For example, a lower bound to the slip at debonding is given by the slip capacity $\delta_{max}$ in Fig. 9b so that the rotation limit in Fig. 10 is simply $\delta_{max}/h_{FRP}$ where $h_{FRP}$ is the distance of the FRP plate to the crack tip. IC debonding theory has also been used to determine the slip of the steel reinforcing bar $\delta_{bar}$ at fracture after strain hardening or at IC debonding (Haskett et al 2007).

The major difficulty has always been in quantifying the rotational capacity of the concrete softening wedge, $(\theta_{slide})_{limit}$ in Fig. 10, and this has arisen because concrete softening has been treated as a material property. However, recently concrete softening has been treated as a mechanical mechanism using shear-friction theory (Fantilli et al 1998, and 2002, Debernardi and Taliano 2002) and this has allowed a solution to be found (Mohamed Ali, M.S. et al 2007, Oehlers et al 2007b). Shear-friction theory has been used to quantify the force $P_{soft}$ in the concrete wedge in Fig. 10 (and hence the effective stress $\sigma_{soft}$ in Fig. 5) of depth $d_{soft}$ and length $L_{soft}$ and also to determine the angle of wedge $\alpha$ which has the weakest failure plane. This can be used in the analysis in Fig. 11 to determine the variation in curvature along the length $L_{soft}$ and, hence, the softening rotation. However, the limit to this rotation occurs when the wedge slides across the failure plane as in Fig. 6. From partial-interaction theory (Oehlers et al 2007b), an upper bound the slip across the interface C-D in Fig. 11 is simply the strain at the commencement of softening, $\varepsilon_{start}$ in Fig. 5, times $L_{soft}$. The slip capacity has to be determined from tests such as those in Fig. 12 where it can be seen that the slip capacity increases with confinement (Oehlers et al 2007).

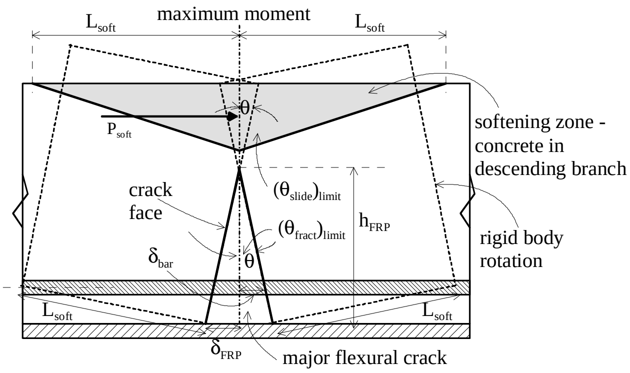


Figure 10 . Rigid body rotation of hinge

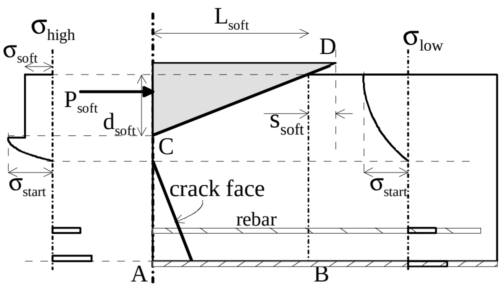


Figure 11. Partial-interaction softening rotation limit

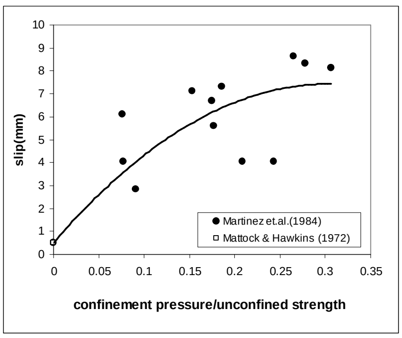


Figure 12. Shear-friction slip capacity

## 5. CONCLUSIONS

A partial-interaction numerical model with partial-interaction limits has been described that simulates the beam rotation at all stages of loading and in particular when the concrete softens.

It has been shown that the limit to rotation due to the FRP reinforcement debonding is simple to quantify using well established intermediate crack debonding theory. This FRP plated RC beam ductility model will be used to develop design rules for the ductility of FRP plated members to help engineers design FRP plated members specifically for ductility as opposed to strength.

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