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# Flexural failure modes of steel plate-strengthened reinforced concrete elements

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**Abstract.** Failure of plate strengthened flexural reinforced concrete (RC) member can be crushing of the concrete at the compression face or premature debonding of the plate from the concrete element at the tension face. Debonding can be classified into three groups; namely, intermediate crack debonding (ICD), delamination and plate-end debonding (PED). Intermediate crack debonding is caused by flexural or flexural-shear stress, and plate-end debonding and delamination are caused by interfacial stress. This paper reviews the failure modes of flexural strengthened RC elements in bending, using epoxy-bonded steel plates. It also attempts to establish the width-to-thickness ratios of plates that can achieve the full flexural strengths of the strengthened element and promote a ductile failure.

**Keywords.** Reinforced concrete, flexural strengthening, epoxy bonding, steel plates, premature failure, ductility and stiffness, width-thickness ratio.

## Introduction

The failure modes of normally reinforced concrete (RC) elements, in bending, are well established in literature and can be adequately taken care of in the design. Although the flexural failure modes of the steel plate-strengthened RC elements are identifiable, they are not yet adequately mitigated in design. Investigations have been ongoing for several decades to try and develop the design parameters for these elements. The structural behaviour of beams and slabs is similar in most respects, however, slabs have their own unique structural properties in that their thicknesses are considerably smaller than their other dimensions, shear reinforcement is not normally required due to higher shear span/depth ratio [1] and their flexural reinforcement spacing differs from that of beams. Due to these factors, the results of beams strengthening studies cannot be just extrapolated to slabs.

## 1. Failure modes

A structure is believed to have failed if it can no longer fulfil the purpose for which it was designed. For ordinary RC element in bending, failure is usually by flexure which occurs when the reinforcing bars at the tension face yield and the concrete at the compression face crushes as shown in Figure 1. However, in a plate strengthened

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flexural RC member, failure can imply crushing of the concrete at the compression face or debonding of the plate from the concrete element at the tension face. The latter premature failure mode can be intermediate crack debonding (ICD), delamination or plate-end debonding (PED). According to Teng *et al* [2] this premature failure can be categorized into two main groups of (i) flexural or flexural-shear stress induced failure (intermediate crack debonding) and (ii) interfacial stress induced failure (plate-end debonding and delamination). It should be noted that cracking at the concrete-plate interface usually leads to slip (difference in strain between the plate and adjacent concrete), which is a partial interaction problem, and this precedes debonding [3].

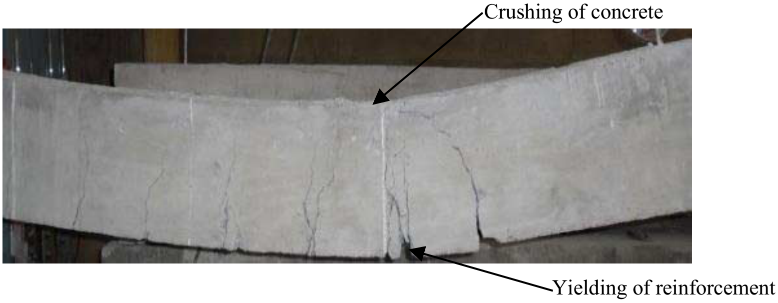
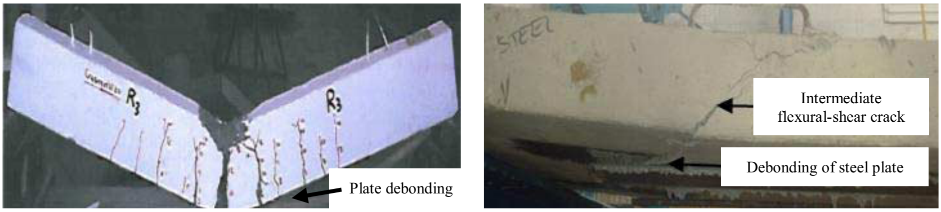


**Figure 1.** Yielding of reinforcement and crushing of concrete

## *1.1. Intermediate crack debonding*

Intermediate crack debonding is the separation of a steel plate from the concrete surface in externally bonded (EB) elements. It occurs in zones of high bending moment or where both the bending moment and shear force are high, after the formation of flexural or flexural-shear cracks, and propagates towards the end of the strengthened RC element [4]. The strain in a bonded plate is high at the point of high moment of the element, and because the concrete has lower strength than the adhesive, this failure mode usually occurs in the concrete, at the adhesive-concrete interface [3]. As the crack forms and propagates, the tensile stress from the concrete is transferred to the externally bonded plate (EBP). This stress continues to increase as the loading increases until a critical level is reached at which debonding is initiated. Interfacial debonding commences from the high stress zone and propagates from the crack point towards the end of the plate [2, 4, 5]. If the failure is dominated by flexural crack as shown in Figure 2(a), then debonding will commence from the mid-span and propagation will be due to the widening of the flexural cracks [6]. This failure mechanism is often referred to as intermediate flexural crack (IFC) debonding, and is common in slender members [4].

When debonding is caused by flexural-shear cracks, as shown in Figure 2(b), it is called intermediate flexural-shear crack debonding. In this case, debonding propagation is due to the combination of widening of the crack and the induced stress caused by the relative vertical movement between the cracked faces of the beam before the formation of a diagonal shear crack [3]. It should be noted that intermediate crack debonding does not usually lead to complete peeling off of the steel plate from the entire beam length as the cracks never reaches the ends of the beam [5]. According to Liu et al. [3] intermediate crack debonding is difficult to prevent, but compared to plate-end and critical diagonal crack debonding, it gives more ductile failure. The strain variation is not linear at the cracked section of a RC member, but linear at the uncracked section.



(a) Debonding caused by flexural cracks [6]

(b) Debonding caused by flexural-shear crack [7]

**Figure 2.** Intermediate crack debonding caused by flexural and flexural-shear failures

### *1.2. Delamination or rip-off failure*

Delamination is partial separation of the steel plates from the concrete, together with a part of the concrete cover [8]. It is a common failure mode of externally bonded strengthened beams or slabs, and is usually initiated at the curtailment end of the strengthening plate if the epoxy glue is strong enough to prevent plate separation. This leads to a rip-off of the concrete cover [9], as shown in Figure 3. From Figure 3, it can be observed that the crack started as a small diagonal crack at the end of the plate, which then extended to the reinforcing bars. A possible cause of this crack is the abrupt change in stresses from the steel plate to the concrete. The presence of the reinforcing steel forced the crack to propagate along the bars, towards the mid-span. Due to the influence of the high shear and bending stresses (biaxial stress) at the loading point, the crack changed direction and propagated at about 60°, towards this point [9]. Since the crack is more than 45° (typical of unplated beams), this leads to reduced shear capacity. According to Jumaat et al. [4], the tearing of the concrete cover along the internal reinforcing bars in this failure mode is an indication of the existence of a strong bond between the concrete and the plate.

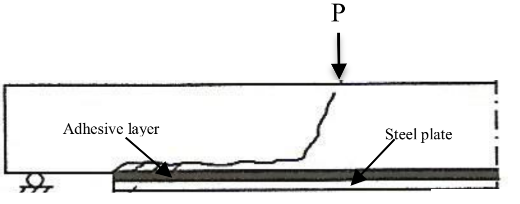


**Figure 3.** Delamination or rip-off failure [9]

### *1.3. Plate-end debonding*

Plate-end debonding or plate separation is caused by high interfacial normal and shear stresses at the end of the plate, and usually leads to the peeling off of the plate, with little or no concrete [8]. It usually occurs before yielding of the reinforcing steel bars and the steel plate at the tension face, and prior to crushing of the concrete at the compression face. This mode of failure can be initiated either by flexural or diagonal shear cracks, depending on the zone of plate end curtailment. If the plate-end curtailment is in the zone of maximum bending moment (center of the beam) the plate separation will be initiated by the formation flexural cracks [10]. Similarly, if the plate-end curtailment is in the zone of high shear force, which is usually at the end of the beam, then plate separation will be initiated by the formation shear cracks [11]. The shear initiated plate separation is common for members of low overall span/depth ratio and with low plate width-to-thickness ratio.

Hollaway and Mays [12] showed that plate separation in steel plate strengthened beams is common if the shear span/depth ratio ($a_v/h$) ranges from 2.22 and 5.00. For shear span/depth ratio greater than 3.5, flexural and shear cracks are narrower than that of unplated beam, and initiation of plate separation shifts from plate free end to any point of widest shear crack, within the shear span area. Shear plate-end debonding occurs within a short time, making it more problematic to handle than that of flexural crack [4, 10, 13]. The chances of occurrence of plate-end debonding failure increase with increase in the flexural strengthening steel plate thickness [5, 8].

Figure 4 shows an example of plate-end interfacial debonding due to shear cracks. It can be observed from the figure that as plate separation propagated towards the mid-span, it changed into a diagonal crack, which extended towards the loading point. The latter failure mechanism is called critical diagonal crack (CDC) debonding and usually occurs after the formation of large crack, which may be due to insufficient shear reinforcement [4].

**Figure 4.** Plate-end debonding failure

To mitigate these premature failures, bolts have been used as anchorages of plate strengthened RC elements [5, 14, 15]. Anchor bolts prevent separation of the plate from the concrete and improve the stiffness [5, 16], however, it has been concluded that anchor bolts do not prevent debonding or facilitate the achievement of full composite action [14]. According to Aykac et al. [5], the use of bolts to anchor thin plates actually had negative effects on the modulus of toughness (MOT) and ultimate load-carrying capacity of the beams. This was attributed to reduction in the cross-section of the thin plate as a result of the bolt-hole. Bolted plates may introduce localized stress concentration around the bolt-hole and may be obstructed by the internal reinforcing bars. This can be critical in situations where adequate cover was not provided at the initial construction phase and initial constructional detailed information is absent. In addition, bolting does not usually give good aesthetic finish, and additional work and cost of these complementary devices can make strengthening more expensive and discouraging.

Provision of other anchorage systems such as extended plates or sheets over the supports and bonded angle sections at the ends, to mitigate these failure modes has been reported from other investigations [10, 11, 12, 17]. According to Hollaway and Mays [12] strength improvement of anchored over unanchored plating is relevant only for $a_v/h$ less than 4.0 and is of little benefit for higher ratios under serviceability loading.

It was also observed that cracks in the plated beams usually occur at the concrete-adhesive interface and are finer and more uniformly distributed within the constant moment zone than in the unplated beams [12]. Sena-Cruz et al. [17] reported on the use of mechanically fastened and externally bonded reinforcement (MF-EBR) technique with the aim of mitigating the premature failure in strengthened beams. It was shown that MF-EBR beams gave higher deflection (an indication of ductility) before failure, than those of EBR and NSM. Under monotonic loading, the load carrying capacities of MF-EBR beams over that of EBR beams were higher by about 37%, while NSM strengthened beam had the highest load carrying capacity under post-fatigue monotonic tests [17]. It should be noted that mitigation of these failure modes by extension of the plates or sheets over the supports can be difficult to achieve in an already built structure. In some cases this can actually damage the supports, which in most instances are RC supports.

## 2. Effects of plate dimensions on the failure modes

Aykac et al. [5] showed that there is inverse relationship between the strengthening steel plate thickness and the modulus of toughness (MOT) of strengthened beams, because the lower the plate thickness the higher was the beam's MOT. On the other hand, the rigidity of the strengthened beams was found to be directly proportional to the thickness of the plates. Strengthening with steel plates thicker than 5 mm increased the stiffness; however, the ductility was reduced, which usually led to premature failure [9]. In this work, flexural failure occurred in plate thickness up to 3 mm, at a curtailment distance not exceeding 500 mm from supports and with 5 mm thick plate, at a curtailment distance not exceeding 100 mm from supports. For higher plate thicknesses and curtailment distances, the failure was a combination of flexure and shear, and rip-off. This means that curtailment of the strengthening steel plates should be as close as possible to the supports. Bruwer and Dundu [7] achieved full composite action and yielding of the steel plates in RC slabs strengthened with 6 mm thick plates but had debonding and shearing failure in RC slabs strengthened with 8 mm thick plates. Hollaway and Mays [12] recommended the use of as wide as possible plates in beams of low ($a_v/h$) for even distribution of flexural strain across the beam section. This emphasizes the need to establish some design parameters such as plate width/thickness ratio (b/t) and safe ($a_v/h$) for efficient use of steel plates in strengthening slabs, in particular.

Based on the above investigations, it can be concluded that for structural elements in which ductility requirements override that of stiffness, steel plates of thickness less or equal to 6 mm are considered better. On the other hand, where stiffness is of higher importance, thicker steel plates than 6 mm can be used. Whilst ductility is very important for general RC elements in bending, stiffness is also important in special structures like bridges which are constantly subjected to combination of static and dynamic loading. In addition, curtailment of the strengthening steel plates should be closer to the supports as much as practically possible.

## 3. Conclusions

This paper has highlighted premature failures [intermediate crack debonding (ICD), delamination and plate-end debonding (PED) associated with this technique and the need for further investigations. It is noted that most of the published investigations on the plate strengthening of RC elements are on beams of low shear span/beam depth ratios ($a_v/h$), and very few studies have been carried out on slabs. In these investigations, the elements have been tested mainly under static loading while dynamic loading was rarely considered. As noted previously, slabs have unique structural properties that are different from the properties of beams. Slab thicknesses are considerably smaller than their other dimensions, shear reinforcement is not normally required due to higher shear span/depth ratio and their flexural reinforcement spacing differs from that of beams. Due to these factors, design parameters such as plate width/thickness ratio (b/t) and plates spacing (s) need to be developed for the slabs.

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