STRENGTHENING OF RC ELEMENTS BY CFRP PLATES LOCAL FAILURE

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SUMMARY

Different failure modes can occur in reinforced concrete (RC) beams strengthened by fiber reinforced plastic (FRP) plates. These modes can be divided into two general categories of flexural and local failures. Local failures include the ‘peeling off’ of the FRP plate at the location of high interfacial stresses and shear failure of the concrete layer between the plate and the longitudinal reinforcement. The local failure modes prevent strengthened beams from reaching their ultimate flexural capacity and demonstrating ductility. Since in many cases the failure of such beams is governed by the local failure type, this paper concentrates on reporting tests and investigations made by many researchers to verify this problem and to develop a thorough understanding of the behavior of beams strengthened by CFRP plates. The final objective of the work undertaken by the author at the Technical University of Budapest, currently at the Center for Cement & Concrete at the University of Sheffield, is to help with the development of design methods applicable for such beams.

Keywords: FRP, CFRP, strengthening, peeling off failure

1. INTRODUCTION

Nowadays, strengthening of existing reinforced concrete structures has become one of the most important activities in civil engineering. The main reasons of strengthening structures are to restore and enhance the load bearing capacity to reduce deflection at service loading, or to limit the width and distribution of cracks in concrete.

Furthermore, structures often have to carry higher loads to fulfill new standards and regulations. In extreme cases they also have to be repaired due to accidental loading. Other reasons for repair and strengthening can be attributed to errors made during the design or construction stages.

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External strengthening of buildings and bridges by bonding steel plates has been applied in construction for over twenty years. The main disadvantages of using steel plates are steel corrosion in the adhesion zone and heavy weight of single plates. Consequently, corrosion protection is required and the handling of the heavy steel plates is difficult. These disadvantages have led to the use of fiber reinforced plastics (FRP) as alternatives to the steel plates.

FRP plates are increasingly used to replace steel plates due to their beneficial characteristics of being non-corrosive, non-magnetic, non-conductive, generally resistant to chemicals, and having a high strength to weight ratio.

Among different types of FRP materials, CFRP appears to be the most applicable in this field, regarding strength, stiffness, durability and fatigue characteristics. CFRP materials are also known to perform better at elevated temperatures (Rostásy, 1990), possess better damping characteristics and they have the best resistance to chemical corrosion compared to other FRP (Rostásy, 1990).

Theoretical studies and a large number of experiments have been carried out all over the world, in order to study the behavior of reinforced concrete structural elements strengthened with epoxy bonded CFRP plates. It has been found that, as with steel mode of local failure is the most likely failure when using CFRP plates. Analytical models have been developed for predicting the shear and normal stresses at the concrete/ FRP interface. Results from different analytical models are presented here for calculating the shear and normal interfacial stresses.

2. MATERIALS

Commercially available CFRP- plates are normally 1.0 - 1.5 mm thick and, 50 - 150 mm wide. They consist of 60 - 70% by volume of unidirectional carbon fibers of approximately 10 μm diameter (see Tab. 1), embedded in an epoxy resin matrix.

Tab.1 Properties of some available CFRP fibers

property	unit	TYPE					
		HM 300	HM 400	HM 500	HT 3.6	HT 4.5	HT 7.0
density	kg/m^3	1800	1800	1900	1700	1800	1800
tensile strength	kN/mm^2	4	2.7	2.4	3.6	4.5	7
Young's modules	kN/mm^2	300	400	500	230	250	300
failure strain	%	1.3	0.6	0.5	1.5	1.8	1.8

The adhesives are generally two component epoxy resins, mostly blended with quartz fillers (Neubauer, Rostásy, 1996). Their tensile strength of ≥ 30 N/mm^2 exceeds the one of concrete by more than a factor of 10. In addition, they exhibit low shrinkage and creep, as well as, high temperature and chemical resistance (Neubauer, Rostásy, 1996).

The mechanical properties of the CFRP plate in the longitudinal direction are almost exclusively governed by the fibers. Their stress-strain behavior is, as that of the fiber linear-elastic unto failure.

Although the matrix contribution to the strength of the plate is negligible, nevertheless the tensile strength of the matrix, which is 60-90 Mpa, is much higher than that of concrete, which is an essential fact for the transfer of bond stresses. The high ultimate strain of 3-5% of the matrix ensures the composite action of the fibers over the entire range of possible plate tensile stresses.

To satisfy strengthening and stiffening requirements, the designer may select the thickness of the CFRP plates and its stiffness. For the choice of the adhesive, the designer usually relies on the selection made by the manufacturer of the FRP system.

3. PRACTICAL CONSIDERATIONS

Before beginning to strengthen a concrete structure it is necessary to investigate the condition of the structure, to see whether it is suitable for strengthening. If the internal reinforcement has started to corrode or if the concrete is attacked by chemicals, the corroded bars or the attacked concrete or both of them should be removed and replaced before strengthening.

On site the concrete surface has to be sandblasted, to remove the latiance, and after sandblasting the surface must be vacuumed or cleaned with compressed air or water. The surface of the FRP plate must be clean so it is best to keep any peel ply on until just before applying adhesive.

The execution of the bonding work is of tremendous importance in order to achieve a composite action between the adherents. Reliable bond between concrete and CFRP plate via the epoxy adhesive is essential for the composite action of the strengthening member. This is especially important for the anchorage of the plate ends.

4. SOME DESIGN CONSIDERATIONS

The designer has no control over the existing structural element in need of strengthening, since the geometry and the properties of existing steel reinforcement and concrete cannot be modified.

Hence, the designer should determine first whether it is more economic to strengthen the existing structure or to replace it. It is often more complicated to strengthen an existing structure than to build a new one.

The feasibility of strengthening is also an important issue to be considered at the early stage, since achieving a good bond between the adherents depends on the concrete quality and texture and on the accessibility of the surface to be bounded.

4.1 Design for bending

The design for bending is generally carried out as for conventional reinforced concrete using multi layer reinforcement (see Fig.1) by using the assumptions of:

- Linear strain distribution through the full depth of the section.
- Complete composite action between plate and concrete.
- Isotropic behavior for FRP, epoxy, concrete and steel reinforcements.

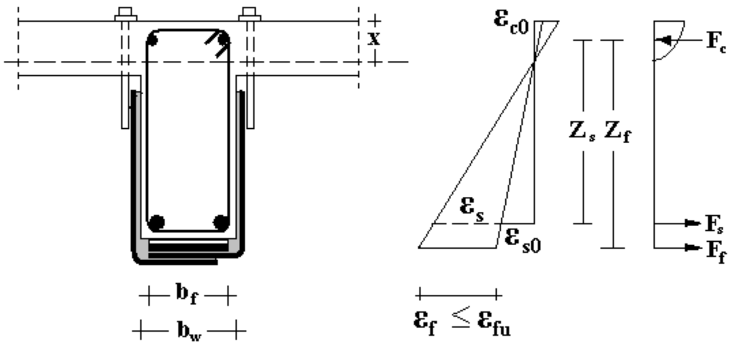


Fig.1 Strains and forces of a plate strengthened RC beam (Neubauer, Rostásy, 1996)

However, in order to satisfy the second assumption and to prevent the separation of the plate from the concrete at bending cracks, as well as to avoid yielding of the internal reinforcement under service load, a certain allowable plate strain in the ultimate limit state must not be exceeded (Neubauer, Rostásy, 1996). Test results and theoretical analysis have led to the following recommendations for the ultimate plate stain with the lower value to be applied (Neubauer, Rostásy, 1996):

$$\varepsilon_{fu} \leq 5\,\varepsilon_{sy} \qquad (1)$$

$$\varepsilon_{fu} \leq \varepsilon_{cu}/2 \qquad (2)$$

With: ε_{fu} = Ultimate plate strain for bending design

ε_{sy} = Yield strain of the internal steel reinforcement

ε_{cu} = Ultimate tensile strain of the CFRP-composite material

To maintain enough safety against damage of CFRP plates due to fire, the unstrengthened member should be able to carry the loads considering safety factors γ =1.4. On the other hand the degree of strengthening, η_B , is generally limited to:

$$\eta_B = M_u/M_{u0} \leq 2 \qquad (3)$$

With: M_u = Ultimate bending moment of the strengthened member

M_{u0} = Ultimate bending moment of the unstrngthened member

4.2 Local failure models

Local failure mainly occurs because of shear and normal stress concentrations at the plate end and at the flexural cracks present along the beam. The stress distribution shows significant stress concentration at the very ends of the FRP plate, as shown in Fig.2

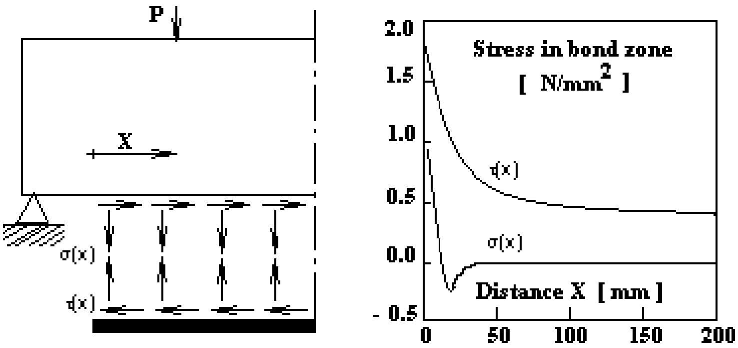


Fig.2 Shear and peeling stresses at the end of the plate in the bond zone

Nevertheless, experiments have shown that, during the phase of flexural cracking in concrete, the distribution of the shear and normal stresses along the adhesive- concrete interface changes dramatically from that of the elastic phase. In the area around each crack, high stress concentrations develop due to the presence of the FRP reinforcement that opposes the opening of the flexural crack. Estimation and prediction of these stresses are very important and must be taken into account either explicitly or implicitly in design considerations.

Malek, Saadatmanesh and Ehsani (1998) have developed an analytical model for predicting the shear and normal stresses at the concrete/ FRP interface. Their equations shown below have been worked out to calculate the maximum shear and normal stresses occurring at the cutoff point of the plate:

$$\tau_{max} = t_f (b_3 \sqrt{A} + b_2) \tag{4}$$

$$A = \frac{G_a}{t_a t_f E_f} \tag{5}$$

$$\sigma_{n,max} = \frac{K_n}{2\beta^3}\left(\frac{V_f}{E_f I_f} - \frac{V_c + \beta M_0}{E_c I_c}\right) + \frac{q E_f I_f}{b_f E_c I_c} \tag{6}$$

Equations such (4) and (6) can provide the possible tools for designing strengthened beams against local failure. The parameters in these equations can be simply calculated based on the mechanical characteristics of materials. The author of the above paper indicated that their results are in good agreement with both finite element and experimental results.

Arduini and Nanni (1997) have considered the adhesive/ concrete interface for analysing the shear and normal stress distribution, according to the model shown in Fig 2. The model allow for the study of the effects of stiffness and thickness of the adhesive layer:

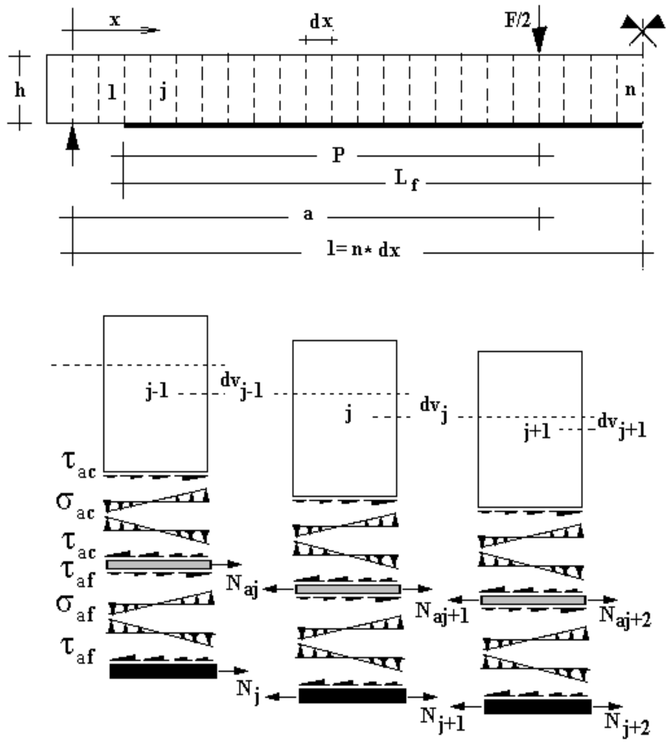


Fig.3 Analytical discrete model (Arduini, Nanni, 1997)

The shear stress distribution for each segment is considered to be triangular and the maximum value is calculated, for a generic segment j, from:

$$\left(\tau_{af,j}\right)_{\max} = \left(N_{j+1} + N_{a,j+1} - N_j - N_{aj}\right)\frac{2}{b.dx} \tag{7}$$

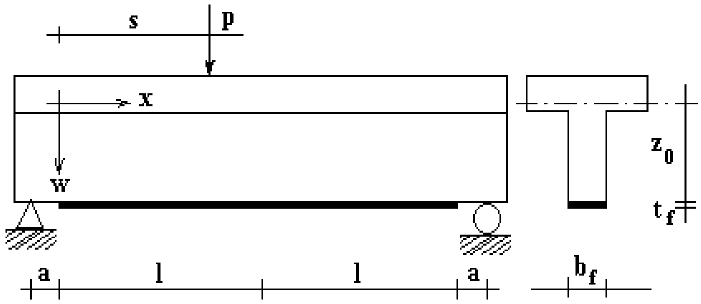
The normal stress distribution at the same interface is also triangular and its maximum value in a generic j segment of the beam is:

$$\left(\sigma_{af,j}\right)_{\max} = \left(N_{j+1} - N_j\right).\left(dv_j + t_a + t_f/2\right) + \left(N_{aj+1} - \left(N_{a,j}\right).\left(dv_j + t_a/2\right)\frac{6}{b.dx^2}\right) \tag{8}$$

With: N = normal horizontal force — b = width of beam
a = adhesive layer — f = FRP plate
c = concrete — t = thickness
af = adhesive layer / FRP plate interface — j = segment

Hence, by the above approach both the normal stress and the shear stress distribution can be calculated along the length of the plate.

Comprehensive research work has also been made by Täljsten (1997) by using the theory of fracture mechanics and deriving closed analytical equations for the shear and peeling stresses at the end of the plate. However, these equations are quite complicated to use and, therefore, only the results of the derivations of the shear stress is shown in equations 9 and 10, for the situation in Fig. 4.



The shear stress between the plate and the beam with the configuration shown in Fig.3. can be expressed as:

$$\tau(x)=\frac{G_a P}{2t_a E_c W_c}\frac{(2l+a-s)}{l+a}\frac{(a\lambda e^{-\lambda x}+1)}{\lambda^2} \quad (9)$$

Where:

$$\lambda^2=\frac{G_a b_f}{t_a}\left[\frac{1}{E_f A_f}+\frac{1}{E_c A_c}+\frac{z_0}{E_c W_c}\right] \quad (10)$$

5. CONCLUSIONS

CFRP plate bounding can enhance the flexural capacity of beams within certain limits.

Shear and normal stress concentrations near the cutoff point of the FRP plate and also flexural cracks must be considered in the design of reinforced concrete beams strengthened with epoxy bonded CFRP plates. These stresses may lead to failure modes such as peeling and debonding of the plate or local failure in the concrete layer between the FRP plate and longitudinal reinforcements of the beam.

A number of investigations on the stresses at the concrete/ FRP interface and closed form solutions of stress concentrations have been developed. The author will analyze these and compare them against experimental results before using them for the development of design guidelines for strengthening reinforced concrete beams with CFRP plates

6. ACKNOWLEDGEMENTS

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7. LIST OF NOTATIONS

A_c cross-sectional area of the reinforced concrete beam
A_f cross-sectional area of the plate
b_f width of the strengthening plate
b_w width of the web
b_2, b_3 parameters used in shear/normal stress equations
E_c modulus of elasticity concrete
E_f modulus of elasticity of the plate
G_a shear modulus of epoxy
I_c moment of inertia of concrete beam
I_f moment of inertia of the plate
K_n normal stiffness per unit area of epoxy
l Length of beam
M_0 bending moment in the concrete beam at the cutoff point due to external load
q external distributed load applied on concrete beam
t_a thickness of adhesive layer
t_f thickness of the strengthening plate
V_c shear force in the concrete beam
V_f shear force in the plate beam
W_c bending stiffness of concrete
β Coefficient used in nirmal stress definition

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