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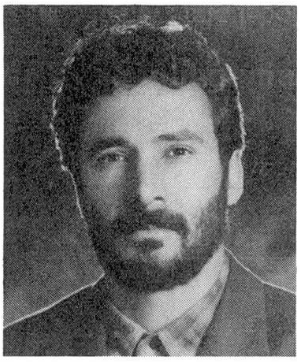

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Summary

Carbon fiber reinforced plastic (CFRP) can be bonded to the tension face of reinforced concrete beams to increase the flexural capacity. In this type of beams, high interface shear stress may result in debonding of plate and premature failure in concrete beam. To design of these beams failure mechanism and relationships between external load and shear strees distribution at concrete/plate interface must be considered. This paper present one important premature failure mode and the method to determine average of shear stress at interface.

1. Failure of concrete layer between the plate and steel

Different failure modes exist for a beam strengthened by FRP plate. First the classical rupture of the beam should be mentioned: either by the plate tensile failure (mode I), or by the concrete crush in the compression zone (mode II). However in this method of strengthening, possibility of premature failure exists at the interface because of the separation of plate (Zhang, 1995). The analysis of different beams shown, the crack pattern when the failure occurs could be defined by figure 1. The cracks propagate in tensile zone of the beam in the concrete layer between FRP plate and the reinforced steel. A part of concrete between two consecutive cracks is working similarly as a cantilever beam. These individual uncracked portions of concrete tend to bend under the influence of shear stresses at their end during the loading. When the concrete tensile stress at the section in contact with steel (point A) is higher than the concrete tensile strength (f't), debonding could appear suddenly. The above observations suggest a possible failure mode which is controlled by the characteristics of the individual teeth between two consecutively cracks in the concrete cover. If we neglect interaction between each teeth supposing an elastic behaviour for each canteliver beam, the tensile stress in the point A could be calculated as:

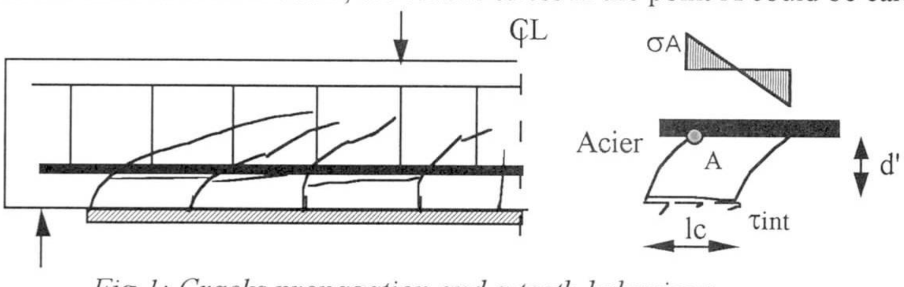

Fig.1: Cracks propagation and a teeth behaviour

$$\sigma_A = M_A \cdot (l_c / 2) / I_t \qquad (1)$$

where It is the second moment of area of the section, equal to $I_t = b.l_c^{\,3} / 12$ and:

$$M_A = \tau_{int} \cdot l_c \cdot b_p \cdot d' \qquad (2)$$

where τ_{int} is the average interface shear stress which is reasonably assumed to be uniformly distributed and lc is the height of the section in cantilever beam equal to the flexural crack spacings in the beam and finally d' is the concrete layer thickness between the FRP plate and the reinforced steel. With the substitution of equation 2 by equation 1 and by using $\sigma_A = f't$ (ultimate resistance of concrete in traction), the admissible shear stress at the interface is obtained:

$$\tau_{adm} = (f'_t . l_c / 6d') / (b / b_p) \qquad (3)$$

This equation explains the debonding criteria due to the concrete cover failure and failure appears, when the shear stress value at the interface reaches τ_{adm}. This proposed theoretical model also depends on the crack spacing size (lc). The experimental result of the different large scale beams show that, lc is more or less equal to the average stirrup distance (S) in the shear zone.

2. Shear stress distribution at the interface

It is obvious that in order to anticipate the debonding of the plate, it is necessary to determine the distribution of the shear stresses at the level of the interface during the loading. In this part, we suggest a new equation to determine the maximal shear stresses at the interface plate/concrete on the basis of a parametric study (Varastehpour, 1996). To simulate the non-linear behaviour of material in the distribution of the maximum stress at the interface, we examined the effect of the different variables, such as rigidity and thickness of the plate, geometry of the section, the loading mode,etc. As a consequence of this parametric study, we introduce a factor β, made up of the different variables which have an important influence on the distribution of shear stress at the interface. Figure 2 shows the dispersion of the maximum normalized value of (multiplied by β) the shear stress for different examples determined by a non-linear software as a function of $\beta.\lambda.V$ on a logarithmic scale. By using regression analysis, the best fit line was traced in order to determine the relationship between shear force and shear stress.

$$\tau_{int} = \frac{1}{2} . \beta^{0.5} . (\lambda . V)^{1.5} \qquad (4)$$

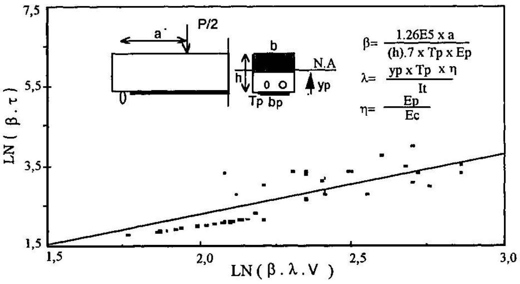

Fig.2 Regression to evaluate the shear stress-load relationships

5. Test results and conclution

The theoretical study of the beam strengthened with a FRP plate, allows to think that the mechanical behaviour (rigidity, resistance) depends strongly on the interaction of the plate/concrete interface. The ultimate capacity of the beam could be determined by the premature failure due to the debonding of the plate. The failure criterion defined in this paper show rupture of the concrete layer situated between the reinforced steel and the FRP plate. Equation [4] is suggested in this study. It allows us to estimate the distribution of the shear stress at the interface. To determine the separation load, it is necessary to solve this equation knowing the admissible interface stress (τ_{int}), and the relation between the applied load and the shear force. For example in the case of a beam under four points bending ($V=p/2$):

$$P_{sep} = \frac{3.2\, \tau_{adm}^{2/3}}{\lambda . \beta^{1/3}} \qquad (5)$$

The admissible shear stress, in this equation, is given by equation [3] according to the premature failure criteria. In the case of beams reinforced by thick plates, this separation load (P_{sep}) corresponds to the ultimate capacity of the beam. Figure 4 shown comparison of the ultimate load, result of differents test and theoretical value when we used classical method or equation [5].

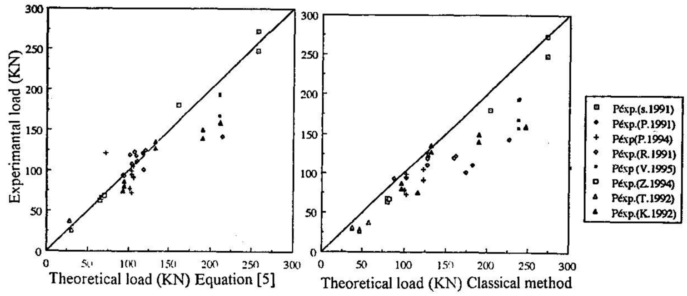

Fig 3: Comparison between experimantal and therotical result

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