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Design Guidelines for Concrete Beams Prestressed with Partially-Bonded Fiber Reinforced Plastic Tendons

by J. M. Lees and C. J. Burgoyne

Synopsis:

The bond between an aramid fibre reinforced plastic (AFRP) tendon and concrete has a significant effect on the flexural behaviour of a concrete beam pre-tensioned with AFRP. In particular, the performance of beams with prestressed AFRP tendons can be enhanced by the use of partially-bonded tendons.

Two types of partial bond are possible: intermittent bond, where sections of the tendon are alternately bonded and debonded from the concrete, and adhesive bond, where the tendon is coated with a resin of known, low shear strength. However, the choice between these methods, and the determination of the values of the various parameters required, are not trivial problems.

It is found that a major obstacle in the development of a generalised design procedure for the partially-bonded beams is the uncertainty regarding the rotation at which the concrete will fail. Nevertheless, insight into design aspects of the intermittently-bonded and adhesively-bonded beams is gained and a design methodology is proposed.

Keywords: adhesive; design; fiber reinforced plastics; partial bond; rotation capacity

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INTRODUCTION

Beams prestressed with FRP tendons are susceptible to two modes of failure, both of which degrade the performance or economy of the material. If the tendons are fully bonded to the concrete then the strain in the tendon rises very quickly with increasing curvature - the tendon snaps when it reaches its ultimate strength since it has no ductility. The result is a high moment capacity (since the full strength of the tendon is being used), but a low rotation capacity.

The normal alternative is to unbond the tendon apart from a short length at the end needed to anchor the tendon. When such a beam is loaded it tends to form a single, large crack at the position of maximum moment and to fail by concrete crushing at a large rotation. However, the tendon does not strain to the same extent, so the force in the tendon, and hence the moment capacity, are both lower.

A high rotation capacity is not to be confused with ductility. Very little inelastic work is being done when an unbonded beam fails. Rotation capacity allows moments to redistribute, which is frequently why sections are required to be "ductile" and it allows energy to be stored, but that energy is not dissipated so the sections are not truly ductile.

The authors have carried out a study of an alternative form of construction, in which the bond between the tendon and the concrete is strictly controlled. This limits the relative slip between the tendon and the concrete, which allows the relationship between the force in the tendon and the rotation to be controlled. Experiments were carried out which demonstrated that these partially-bonded beams had the same strength as fully-bonded beams and rotation capacities at least as high as the unbonded beams (1,2).

The partial bond is achieved in one of two ways; intermittently-bonded beams have relatively short bonded lengths of tendon alternating with relatively long unbonded lengths, while adhesively-bonded beams have tendons with a coating of known, low, shear strength. Figure 1(a) shows part of a typical intermittently-bonded beam, with a crack forming in unbonded region 3, and a consequent

increase in force in the tendon from its initial prestress level P_0 to T_3. Figure 1(b) shows an adhesively-bonded beam where the tendon force increases by bond, over a region of length L_{ab}, from the original value P_0 to the tendon force T_3 at the crack location.

An analysis of these beams has been made where sections of the beam are modelled as simple rigid bodies so that all rotation takes place at crack locations. Equilibrium and compatibility considerations, taking account of the movement of the tendon relative to the concrete, then allow the moment-rotation relationship to be established for the beam. These have been shown to model well the behaviour of test beams, and to allow the modes of failure to be determined (3).

The problem now is to reverse that analysis so that it may be used for design. How can the lengths of the intermittent bonded and unbonded regions, or the required shear strength of the adhesive, be determined? Consideration is not given here to how these surface finishes will be provided; that is a matter of manufacturing technology.

DESIGN PROCEDURE

It is expected that at the serviceability limit state, the prestressed beams will remain uncracked. However, after first cracking occurs, the behaviour of the beam is directly related to the bond characteristics. Since the tendon is either regarded as being fixed to the concrete or sliding past it, it is sufficiently accurate to represent the bond behaviour by a simple model as shown in Figure 2. The values of the two parameters τ_{max} and τ_{frict} can be found by testing.

In order to develop a design procedure, a knowledge of the concrete failure behaviour is required. By convention, concrete is assumed to fail in compression at a fixed strain, typically 0.0035, but this value is known to be dependent on the gauge length and the quoted value is frequently associated with a gauge length of 200 mm. Hillerborg (4) disputes this theory and suggests that the concrete fails at a given shortening, which is a fixed material property.

In the rigid body analysis used to predict the behaviour of the partially-bonded beams (2), the geometrical relationship at a crack is used to find the forces and extensions through the tendons (see Figure 3). In particular, the product of the neutral axis depth, $n.d$, and the rotation determines the shortening of the top fibre of the beam. Hence, if Hillerborg's theory is correct, it follows that the maximum allowable rotation will be inversely proportional to the neutral axis depth. However, if the concrete fails at a fixed strain, then the length over which the shortening occurs is required in order to determine when the concrete fails.

This problem has not significantly exercised those dealing with beams with steel tendons, since such beams are assumed to be ductile if designed as under-reinforced sections. The precise value of the limiting concrete strain makes very little difference to the resulting section or its reinforcement. But it remains a very important question for design of sections with FRP tendons.

It will be assumed here that the maximum rotation that could occur at a hinge prior to concrete crushing, θ_{ult}, can be specified for a particular beam from a knowledge of the dimensions and the concrete properties.

Intermittently-bonded beams

For design purposes the lengths of the bonded and unbonded regions must be determined. The bonded lengths must be sufficient to ensure that bond does not break down under load. The force to be transferred is the difference between the ultimate strength of the tendon, P_{ult}, and the initial prestress level, P_0 (as in the case where $T_3 = P_{ult}$ in Figure 1(a)). When the maximum bond stress of the material, τ_{max}, is known, the length over which the force is transmitted can be calculated. As further cracks form, the force could increase on either side of the bonded region and hence the minimum bonded length should be twice that calculated in the previous step. For a tendon of diameter ϕ, the minimum bonded length L_b is therefore:

$$L_b = \frac{2(P_{ult} - P_0)}{\tau_{max}\pi\phi} \qquad (1)$$

If θ_{ult} for a particular beam has been determined, the unbonded length of tendon that will allow the tendon to be fully utilised at that rotation can then be found so that the tendon spanning the crack will be assumed to be the ultimate strength, thus $T = P_{ult}$.

A rectangular stress block of depth $0.9n.d$ and stress $0.67f_{cu}$ will be assumed for the concrete at failure. The neutral axis depth is then:

$$n.d = \frac{P_{ult}}{0.67f_{cu}b \cdot 0.9} \qquad (2)$$

where f_{cu} is the compressive cube strength of the concrete and b is the width of the beam.
The extension of the tendon, δL, is:

$$\delta L = d.(1-n)\sin(\theta_{ult}) \qquad (3)$$

If it is assumed that there is little friction between the tendon and the concrete, then the unbonded length that is required to allow the extension is:

$$L_{ub} = \frac{\delta L E_t A_t}{P_{ult} - P_0} \qquad (4)$$

where E_t and A_t are the Young's Modulus and cross-sectional area of the tendon.

The choice of the unbonded lengths will enable the designer to choose the mode of failure, i.e. concrete crushing or tendon rupture. If one or other material is chosen to fail preferentially then a balanced section analysis is no longer applicable since the force in the concrete does not necessarily equal the breaking load of the tendon.

Discussion of intermittently-bonded beams: This analysis takes no account of the slip that is occurring in the bonded regions. It will usually be found that the unbonded lengths are much longer than the bonded ones, so the relative movement will not be significantly affected by a small amount of additional slip in the bond. However, it is relatively simple to make allowance for such slip in the analysis (2).

Design of adhesively-bonded beams

For these beams the adhesive bond shear strength is the crucial parameter. If the bond strength is too low, cracking will be limited and the beam will tend to behave as an unbonded beam with low moment capacity; if too high, the rotation capacity will reduce and the beam will fail due to tendon rupture.

It will be assumed for simplicity here that the bonding layer does not have a high initial breakdown stress, so that $\tau_{max} = \tau_{frict}$. All the transfer of force between the tendon and the concrete thus takes place in the length L_{ab} (Figure 1(b)); at the ends of this region the force in the tendon will be P_0. The required tendon extension across the crack is still given by equation 3, so that L_{ab} is given by:

$$L_{ab} = \frac{2\delta L E_t A_t}{P_{ult} - P_0} \qquad (5)$$

Note that this length is independent of the bond characteristics of the tendon - it represents the length over which it is desirable for slipping to be taking place so that the tendon will not snap prematurely.

To ensure that the this length of slipping can be achieved, the required bond shear strength, τ_{frict}, can be found from:

$$\tau_{frict} = \frac{2(P_{ult} - P_0)}{\pi\phi L_{ab}} \qquad (6)$$

This is an optimum value, so no safety factor is required.

<u>Discussion of adhesively-bonded beams</u>: This is necessarily a simplified analysis. In particular, it assumes that the rotation at a single crack, and the slip of the tendon associated with it, are unaffected by any adjacent cracks. The formation of a second crack, near the first, is controlled by three factors. The first is the build-up of tensile stresses in the beam away from the existing crack, caused by the loading. This will be governed by the form of the bending moment diagram which depends on the type of structure being designed. The second is the redistribution of the compressive force which, at crack locations, is necessarily constrained to lie in the compression zone. However, in the uncracked region nearby, this force, which almost certainly lies outside the middle third of the beam, will cause tensile stresses. This behaviour is essentially limited by St Venant's theorem and is unlikely to cause a crack closer than one beam depth to the initial crack.

The final cause of tensile stress in the beam is the transfer of tensile force by means of bond, and this is where a complex interaction can develop. High bond forces mean a rapid build-up of tensile stresses, with a corresponding reduction in crack spacing, but are associated with low values of L_{ab}. Low bond forces imply a large crack spacing but also a longer force transfer zone.

It has also been assumed that the force in the non-slipping region remains at P_0. This may not be true, and is easily allowed for by reducing the amount of force that has to be transferred by τ_{frict}. When designing suitable tendon coatings, the minimisation of τ_{max} would be one of the design parameters.

FAILURE MODES

The crushing of concrete in compression has traditionally been considered to be a brittle failure mode and, in conventional design with steel reinforcement, beams are under-reinforced.

When using FRP tendon materials the only way to avoid failure due to the tendons' snapping is to use an over-reinforced section where the capacity of the concrete is less than that of the tendons. An over-reinforced beam is thus designed to fail due to the concrete crushing and the behaviour of the compression zone becomes crucial. If spiral or rectangular hoops of reinforcement are included in a concrete compression specimen, the plastic capacity, and hence the ductility, of the concrete increases markedly. Failure due to concrete crushing then becomes a more attractive mode of failure (5).

There would be scope to combine the concepts of concrete confinement and partial bonding into a single beam specimen. In this manner it would be possible to achieve large rotations, a high ultimate load capacity and some real ductility.

CONCLUSIONS

1. A more general understanding of the rotation at which concrete fails is necessary.

2. The behaviour of the partially-bonded beams is influenced by the tendon bond parameters, the crack spacing, the tendon properties and the concrete properties.

3. The concept of partial bonding can be applied as a new design basis for FRP-pre-tensioned concrete structures.

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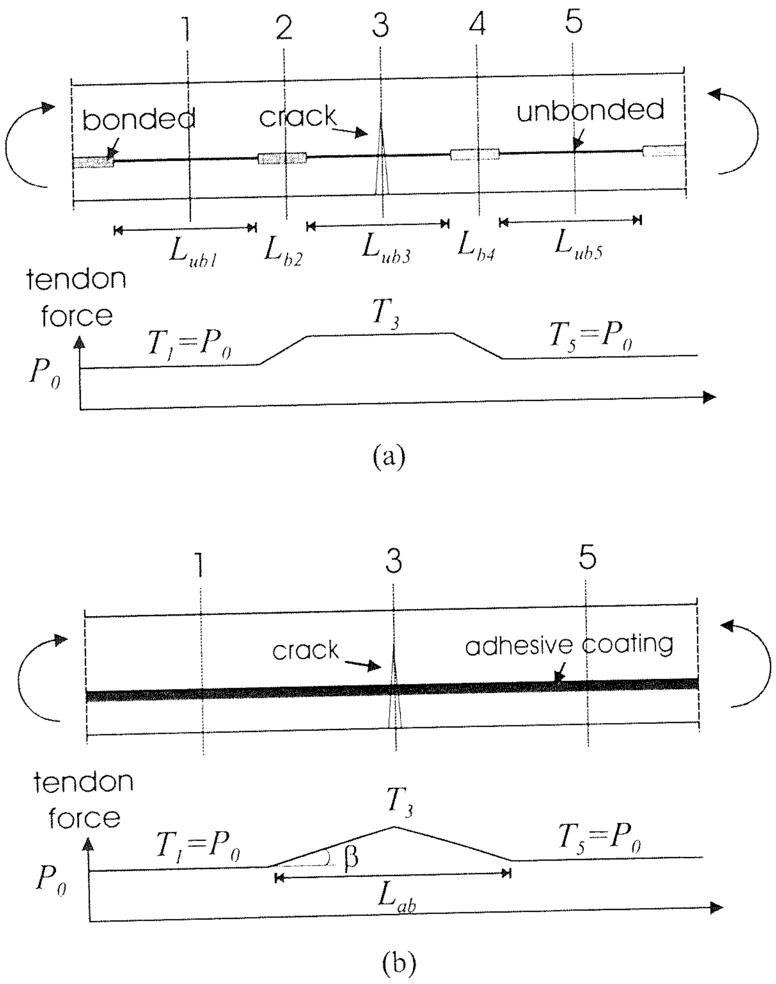

Fig. 1—Schematic change in tendon force at crack locations: a) intermittent bond; and b) adhesive bond.

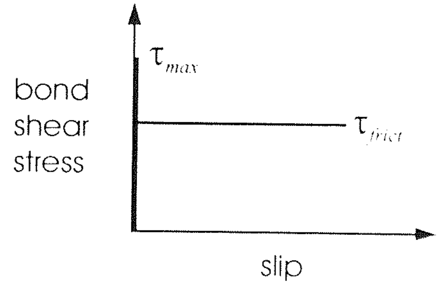

Fig. 2—Assumed bond shear stress versus slip relationship.

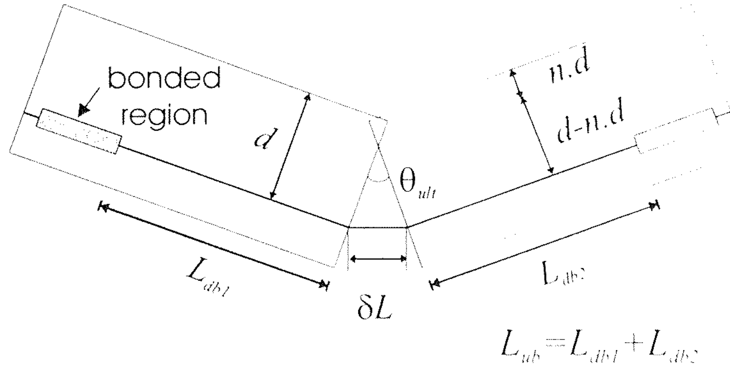

Fig. 3—Rigid block rotation.