Should FRP be bonded to Concrete?

C.J.Burgoyne

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Engineering Department
University of Cambridge
Trumpington St.
Cambridge CB2 1PZ

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Synopsis: The question of whether it is right to bond tendons made of glass, aramid or carbon fibres to concrete has not yet been directly addressed. This paper discusses the various issues involved, and concludes that in many cases, these tendons should remain unbonded.

All the new materials which have a high enough stiffness and a low enough creep show a linear elastic response right up to failure, with little or no ductility. This contrasts with steel, even very high tensile steel, which shows a considerable reduction in stiffness at high loads. In a bonded beam, when cracks form on the tension face of the concrete, very high strains are generated across the crack. With a steel tendon, local yield must occur, with a consequent reduction in cross-section area, which leads to debonding of the bar on either side of the crack. This allows the strain at the crack to reduce below its theoretical maximum value. In calculation, average steel strains are used, which ignore any local increase at the crack positions, but there are some controversial code rules which limit the (average) steel strain to less than the material can actually sustain.

When new materials are used, the local yielding mechanism is no longer available, and the concept of using average strains is no longer justified. In concrete reinforced with FRP, the whole strain capacity of the fibres is available, and it is unlikely that fibre failure will occur before the concrete strains become unacceptable. But in prestressed concrete, much of the fibre strain capacity is absorbed in the prestress, leaving a tendon very sensitive to high strains in the vicinity of cracks.

There is a move to increase the ductility of beams reinforced or prestressed with FRP, by the use of (FRP) cages in the compression zone. This will increase the chances of a bonded tendon snapping before crushing of the concrete occurs. These mechanisms are not present in unbonded tendons, where high local strains do not occur, and indeed the change in stress in the tendon is small. It has been argued, for steel tendons, that this is an economic disadvantage; for FRP tendons, however, it is shown here to be beneficial.

The intention of raising this matter at the conference is to engender debate on the topic, before systems start to become widely used.

Dr Chris Burgoyne is a lecturer at Cambridge University, and has been carrying out research into the properties of aramid ropes for a number of years. His other research interests include the application of expert systems to the design of prestressed concrete, and the general philosophy underlying structural design. He was formerly employed at Imperial College, London.

INTRODUCTION

Prestressing tendons and reinforcing bars, made from new materials such as glass, aramid or carbon fibres, are now becoming widely available; the properties of the parallel-lay ropes or FRP pultrusions made from them are now understood. They are seen as potential replacements for steel in areas where corrosion, weight or the magnetic properties of steel pose problems.

The temptation is to think of "replacing steel with new materials", without, in many cases, going back to first principles. We have become so used to the way we make reinforced and prestressed concrete that we take for granted the reasons why we do things the way we do. When something changes (in this case the introduction of new materials), we fail to consider whether our original assumptions remain valid. This route could lead us to some potentially dangerous or costly mistakes, which could set back the use of new materials for a very long time. It is in anticipation of this problem that the paper has been written.

Before we consider what our new structures should look like, it is important to consider why we build steel reinforced structures the way we do. That requires some questioning of very basic assumptions.

DESIGN WITH BONDED STEEL

With very few exceptions, reinforced and prestressed concrete structures bond the steel to the concrete. Why? There are two main reasons; one associated with corrosion, and the other with the strain capacity of the steel.

Passivation

Steel rusts when exposed to both oxygen and water. This corrosion can be prevented if the steel is held at a high pH, typically in the range 11-13, when a passivation reaction takes place at the surface of the steel (Figure 1)(1). Corrosion is not prevented completely, but a fine surface layer is formed which prevents further corrosion taking place. Concrete, fortuitously, provides exactly the correct environment, where the natural alkalinity of the concrete passivates the steel. It is always assumed that the concrete must be in direct contact with the steel, although this has not been established with any certainty. A small void next to the steel, with no access to the outside world, will either be dry or contain a small amount of water, which will have the same pH as the adjacent concrete. In neither case will corrosion take place. Grout used in prestressing ducts has far more water in it than is needed to make the cement hydrate; most is used to make the mix liquid. This water will eventually evaporate, leaving small voids in the grout. To the best of the author's knowledge, there is no evidence of corrosion taking place in such small voids.

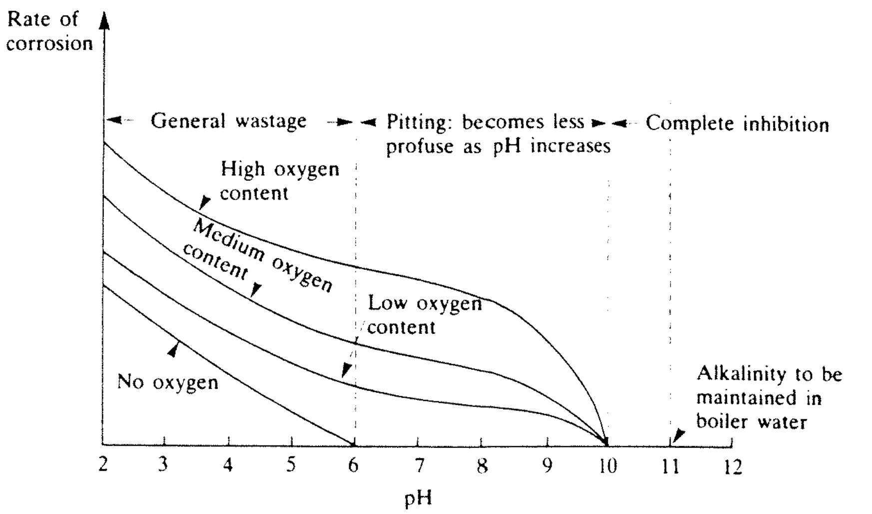

Figure 1. Rate of corrosion, as a function of pH. (taken from reference 1)

If the void is larger, and there is free passage of air and water vapour between the void and outside, then the passivation is lost and corrosion can occur. The same depassivation will occur when carbonation, caused by the diffusion of atmospheric CO_2, reduces the alkalinity of the concrete. But carbonation, in properly compacted concrete, will take decades, so this is assumed not to be a problem if concrete is properly detailed.

The recent decision by the Department of Transport in the UK to ban the use of grouted post-tensioned concrete bridges is a direct result of concerns about corrosion taking place in voids within the ducts (2).

Strain Capacity

The other reason why we bond steel to concrete has to do with their strain capacity. Most concrete, as used in reinforced construction, has an approximately linear response up to a compressive strain of about 0.0012. This is remarkably similar to the strain capacity of reinforcing steels. Thus, we can design reinforced concrete (RC) beams, with the steel taking the tensile strains, and concrete the compressive strains; the neutral axis of the beam is close to the mid-depth of the beam, making full use of both materials (Figure 2a). This is a remarkably good arrangement for two materials which just happened to be both available and relatively cheap at the same time. Both materials share in carrying the load, and the curvatures that are induced are minimised by keeping the neutral axis close to the mid-plane.

Prestressed concrete (PSC) works in much the same way. The concrete is usually of better quality, with a linear strain response up to about 0.0018, and the steel has a much higher linear strain capacity (about 0.006). If we built reinforced concrete with prestressing tendons, we would have to under-reinforce the beam to a great extent to make any significant use of the steel's capacity; very large curvatures would occur if both materials were to reach their strain limits at the same time (Figure 2b). We overcome the problem by *pre-straining* the steel, usually to a strain of about 0.004. This leads to a capacity to resist additional strain of about 0.002, which is very close to that of the concrete. Once again, we find the materials in balance, with a neutral axis close to the mid-plane (Figure 2c). There is the added benefit that the additional strain in the steel increases the stress, which in turn increases the moment capacity. As we shall see later, this mechanism is not so effective when the steel is unbonded.

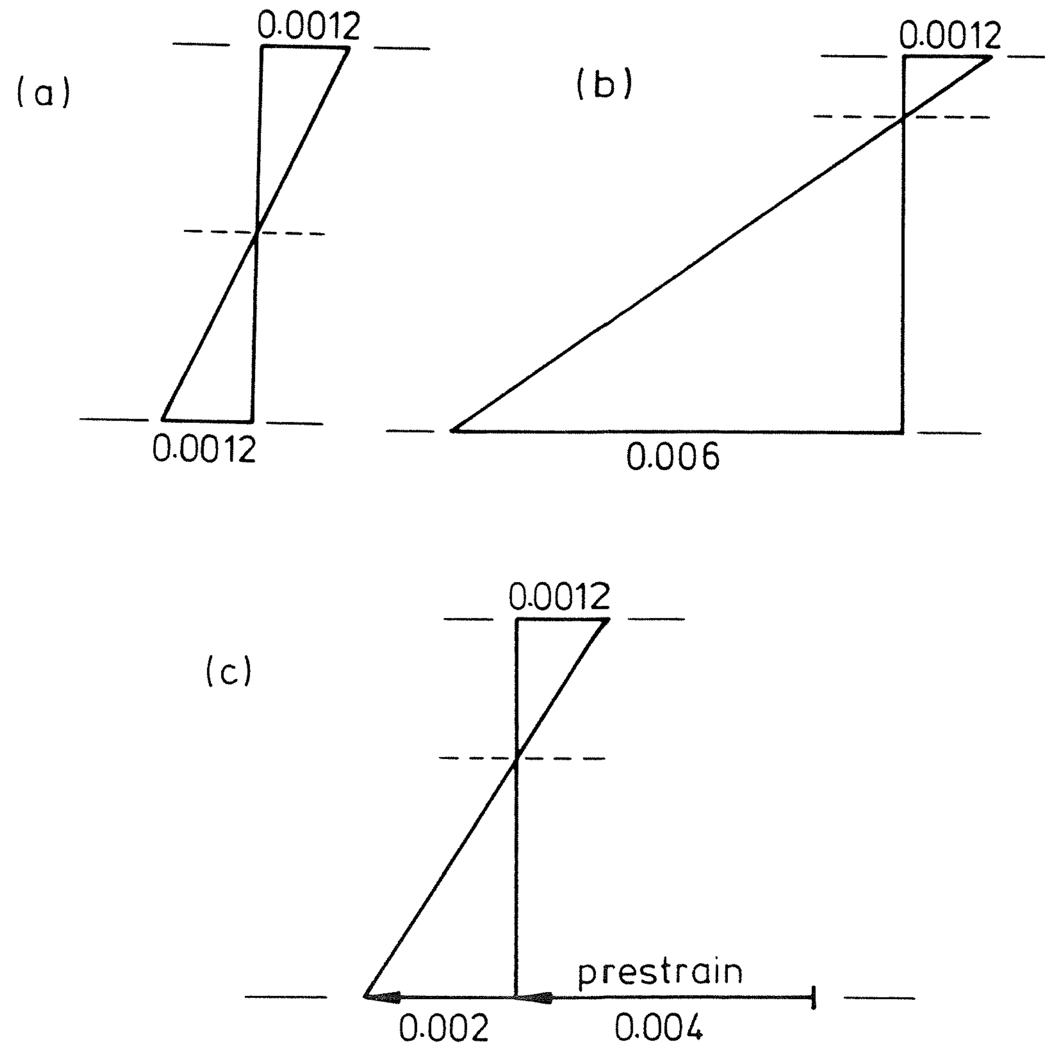

Figure 2. Strain distributions in:-
(a) typical reinforced concrete beam
(b) beam reinforced with untensioned prestressing steel
(c) typical prestressed beam.

So we have two very good reasons for bonding steel to concrete in both reinforced and prestressed concrete. What happens when the concrete cracks, in normal use for RC, and when overloaded for PSC?

Cracked Concrete with Bonded Steel

The first thing to note is that the strain in the concrete, at the crack, is technically unbounded. Consider two points in the concrete, one on each side of the crack, that were in contact before the crack opened. They are now separated by the width of the crack, so the strain is infinite. If the steel were perfectly bonded to the concrete on both sides of the crack, then the steel would also have to have infinite strain, leading to failure by snapping of even the most ductile steel.

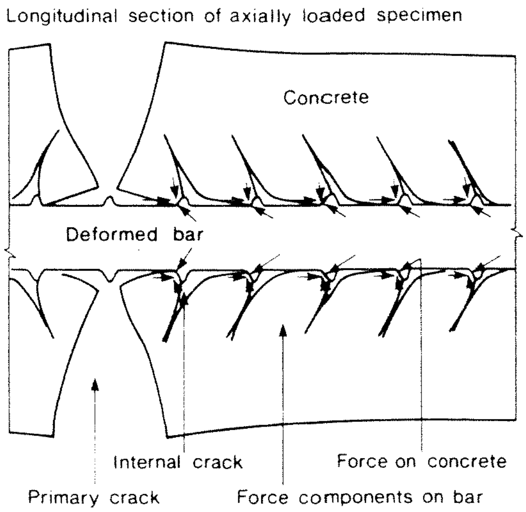

Figure 3. Secondary cracking in vicinity of primary crack.
(taken from reference 3.)

This clearly does not happen, because as soon as the steel starts to yield it gets thinner since the volume of the steel is sensibly constant. We thus get a small amount of debonding at the surface of the bar, until the average strain along the debonded length of the bar is less than the yield strain. If the bar is deformed, with raised ribs, there will be secondary cracking in the concrete (3), which also serves to extend the debonded length and reduce the effective strain (Figure 3).

We take this process for granted. We never try to calculate the actual strain in the steel, being quite happy to assume a linear variation in strain through the depth of the beam (as in Figure 4), even though the compression concrete acts homogeneously, while the concrete in tension is cracked and debonded from the steel. We assume that the steel strain can be taken as the same as that in the adjacent concrete, although we know that debonding takes place and that the concrete strain is wrong. Strain gauges attached to reinforcing bars invariably give the "wrong" strain. Those placed between cracks give a lower than expected strain, since the steel and concrete are acting compositely, whereas those that

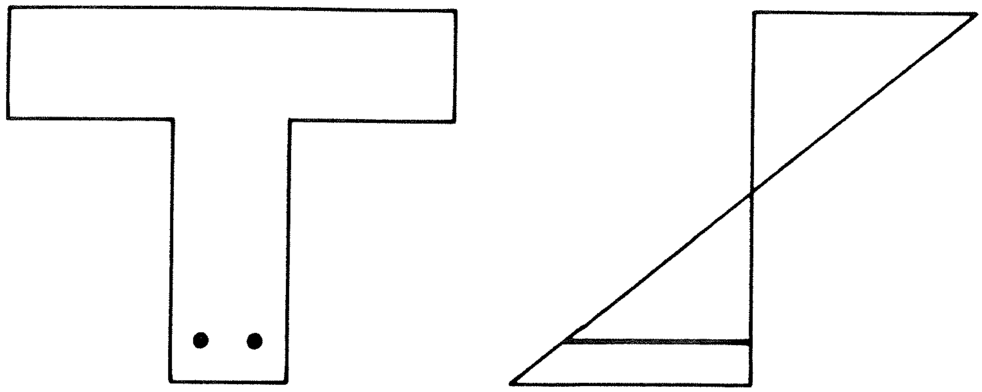

Figure 4. Linear strain distribution assumed through beam.

happen to have been placed at crack positions show much higher strains than expected. The actual strain distribution along the beam in the vicinity of the crack would be as shown in Figure 5.

The controversial rule in the CEB/FIP model code (4) that limited the additional (average) strain in the steel to 1%, even though prestressing steels can normally sustain strains of about 3%, was probably based on the idea that the actual peak strain would be considerably higher than 1%.

We accept this because it works. There is ample evidence that structures designed in this way behave as expected. The local yielding and debonding keeps the situation under control. The strain in both concrete and steel *when averaged over several cracks*, does follow the assumptions above, and when the structure is taken to failure, yielding of the steel becomes general anyway, since we always under-reinforce our beams, so our prophecies become self-fulfilling.

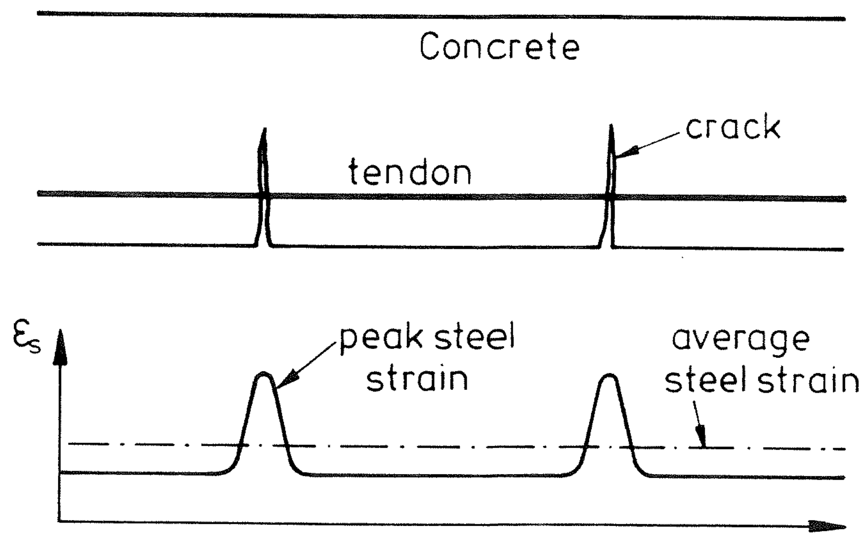

Figure 5. Actual strain distribution along tendon in a cracked beam.

NEW MATERIALS

What will we expect to happen to beams with bonded reinforcement made from glass, aramid or carbon? The first thing to note is the stress-strain curve. Figure 6 shows typical responses for these materials (5), in the form of fibre; they are all effectively linear up to a sudden, brittle failure. Bonding the fibres together with resin will not alter these curves much; the contribution of the resin to the axial strength is negligible, and when the fibres fail, the bar fails.

So we have lost the essential property of the bar that we rely on to shed load from our tendon when the concrete cracks. The strain across the crack remains infinite, at least in theory, and the bar cannot yield to even out the strain in the bar. So what is the strain in the bar?

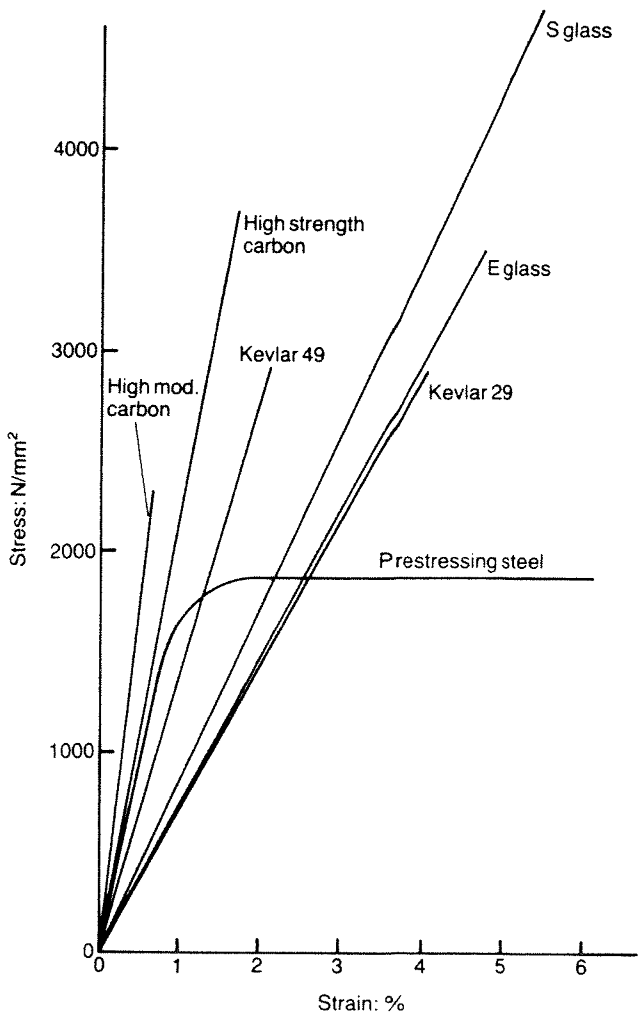

Figure 6. Stress strain curves for glass, aramid and carbon fibres proposed for use as reinforcement or prestressing. (taken from reference 5.)

The answer is that it is almost impossible to know. The strain in the concrete on the face of the crack must be zero, while the strain in the FRP at the crack ought to be infinite. A short distance away from the crack, the strains in the two materials must be the same. There is thus a discontinuity (at point A in Figure 7), which can only be resolved if there is a bond failure crack propagating along the interface. But how far does it go? In steel, the requirement that the steel stress remains below yield allows us to have some idea of what is going on, but no such mechanism exists for FRP.

The controlling factors will thus be surface treatment of the bar, the quality of the concrete, and the degree of compaction and consolidation in the concrete around the bar. These are notoriously difficult matters to control, and almost impossible to test in real structures. Furthermore, the condition of the surface of the bar is almost entirely a function of the resin, and this will be very sensitive to creep. We would expect the bar strains to differ between a beam subjected to transient loads, and one subjected to permanent loads.

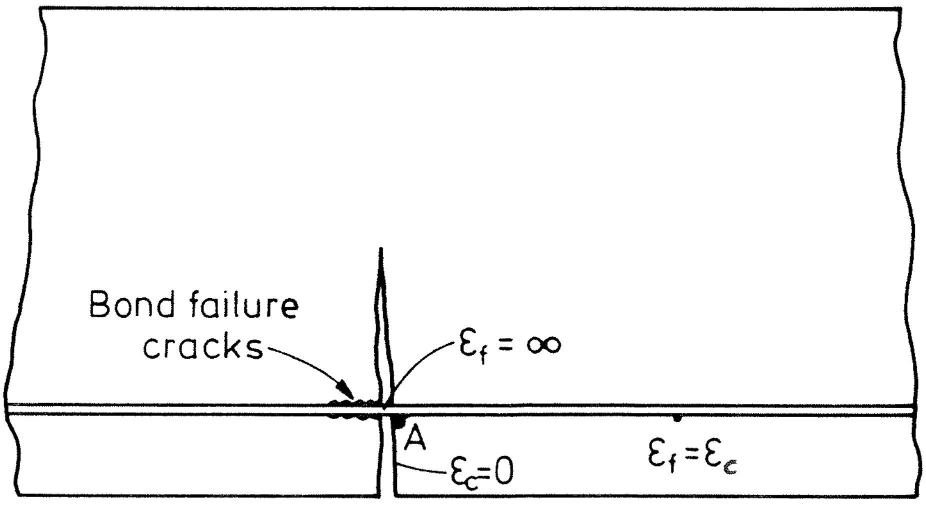

Figure 7. Strains in the vicinity of a crack.

And yet, we shall have to estimate the strain, and hence the stress, in these bars, since we shall have to predict the load capacity that the composite (FRP/concrete) beam will have. Equilibrium considerations will help to a certain extent, but only in so far as we can estimate the lever arm, and we do that at the moment by assuming that the strain distribution is linear.

If the strain distribution is non-linear, it is quite possible for the centre of compression to lie in a different place from that assumed. If it is closer to the reinforcement than originally assumed, then the lever arm would be lower, and the reinforcement stress higher.

There is a further problem with variability of the bars, in instances where there are several reinforcing bars. This is analogous to the bundle theory problems in ropes (6). The various reinforcing bars will have (slightly) different strengths. Clearly the weakest will fail first; if it is steel, it can yield – straining at approximately constant stress – without shedding load. But if it is FRP, it will snap, so that the remaining bars will have to carry its share of the load. This phenomenon usually means that the beam will fail when the weakest bar fails.

Under-reinforcement versus Over-reinforcement

We take for granted the fact that we under-reinforce beams with steel reinforcement, to ensure that the steel yields before the concrete crushes, thus giving us a more "ductile" response. But snapping of FRP bars will be rather final; we shall need to ensure, either that the concrete fails first (i.e. over-reinforce), or overdesign so that the bars stay well below their strain capacity.

This then leads to consideration of the strain capacity problem. Most new materials have strain capacities that are up to an order of magnitude higher than those of reinforcing steels (typically 0.015 or above). They are also expensive, so we want to make good use of them. This means taking them close to their strain capacities. But if we do, the strain diagram will be as shown in Figure 8, once more returning to very high curvatures, with a high neutral axis, and a very under-reinforced beam.

At this point in the argument, it becomes sensible to think in terms of increasing the strain capacity of the concrete in compression. It has long been known that providing a lot of confining reinforcement increases both the strength, and more particularly the strain capacity, of concrete; up to 0.015 or 0.02 can be achieved (7). There has never been much economic sense in making use of this with steel reinforcement, but with high straining FRP, it becomes a more sensible proposition.

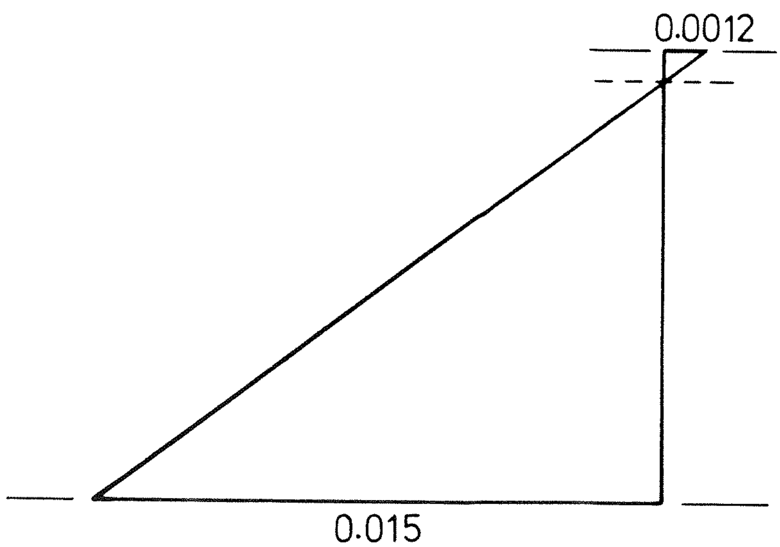

Figure 8. Strain distribution for FRP reinforced beam.

Research Needed

There is very little data available on the behaviour of beams reinforced or prestressed with FRP, and what there is concentrates simply on the load capacity. What is important for future writers of codes of practice, and users of such materials, is information about the *mode* of failure, and the behaviour of the tendon in cracked concrete. This will have to be obtained by careful measurements of concrete strains, studies of crack patterns, and back-analysis of beams to determine reinforcement stresses.

BEAMS WITH UNBONDED REINFORCEMENT

Another possibility is to build the beam with unbonded reinforcement. What happens in this case? For simplicity, it will be assumed that the reinforcement is attached to the concrete at the ends of the beam, either by prestressing anchorages or by bond in reinforcement. The strain in the reinforcement is not now affected by local strains in the concrete, but must respond to changes in length of the whole beam. The reinforcement strain must be the strain of the concrete adjacent to the reinforcement, averaged over the whole length of the beam.

If the beam is subjected to a normal working load, there may be cracking in some regions, but not to any great extent. The strain distribution in the concrete will be similar to that shown in Figure 9, and the average value (as in the reinforcement), will be of the order of 50% to 100% of the maximum value, depending on the shape of the moment diagram.

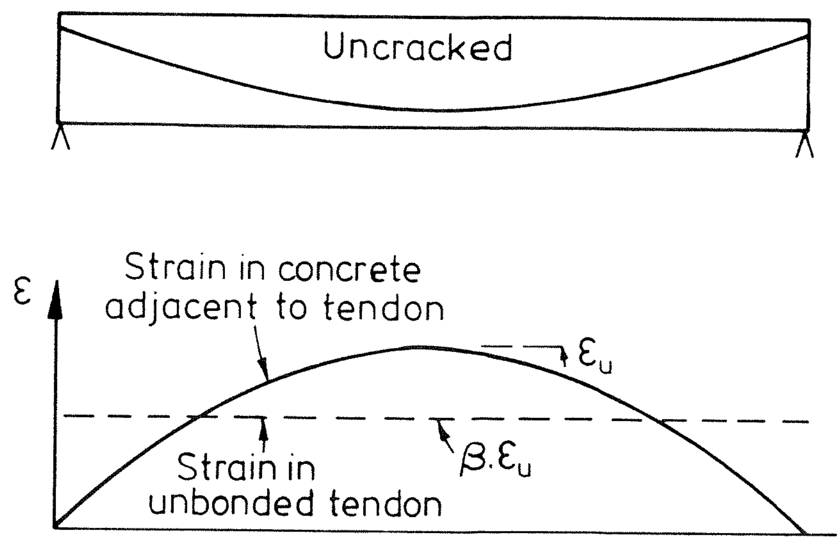

Figure 9. Strains in an uncracked beam with an unbonded tendon.

When the beam is subjected to its ultimate load, one (or more) hinges will form, leading to very high concrete strains in that region. But the reinforcement will not see that peak strain, since it will continue to be subjected to the average value. This will probably be between 10% and 25% of the maximum value (8), once again depending on the details of the loading arrangement (Figure 10).

These values will be affected by contact between the reinforcement and the concrete, such as at the deflectors in prestressed construction, where some sizeable friction effects may arise. Nevertheless, the reinforcement strain will remain much lower than the concrete strain.

If the beam is reinforced, the strains mentioned above will be the *actual* strains in the reinforcement, but if the beam is prestressed, they will be the *additional* strain, over and above that induced by the prestress.

Nothing in the above argument relies on the fact that the reinforcement yields, so the same logic applies if we are talking about steel or FRP. The argument against unbonded construction has always been the loss of corrosion protection, and the loss of the additional stress in the steel. Neither of these will be problems with FRP.

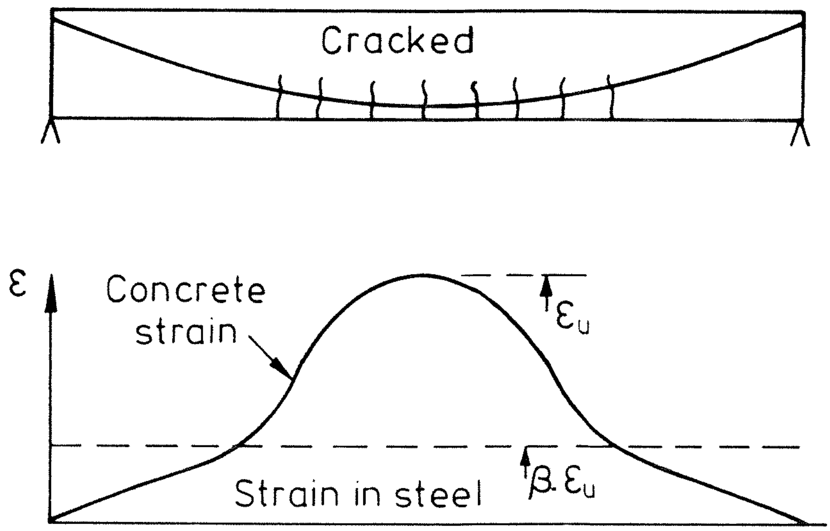

Figure 10. Strains in a cracked beam with an unbonded tendon.

PREDICTIONS

The following predictions are made on the basis of the above arguments. It is assumed that tests and prototype structures will be made in many different configurations. Most will work, and a few will fail, but eventually those that make effective use of the materials will become adopted by the industry.

Beams Reinforced with FRP

Beams reinforced with FRP are unlikely to be adopted widely on strength grounds alone. It will be difficult to get the reinforcement strain sufficiently high without causing unacceptable curvature in the beam, and very high strains in the concrete.

FRP reinforced beams will find application where their resistance to corrosion is of prime importance. In that case, it is unlikely that making maximum use of the material's strength will be important, and stiffness is likely to govern, so the danger of over-stressing the FRP, and hence snapping the bars, will not arise.

Beams Prestressed with FRP

When we consider prestressed construction, the situation is different. We can take up as much of the strain capacity in the tendon as we want by pre-straining it. Economics will dictate that we tension it as highly as possible. The stiffness of the beam at the working load will not be a problem; that comes almost entirely from the uncracked concrete. The use of confining links in the compression zone of the concrete will be economically attractive, so we will be left with the idea that, at failure, we will want to induce high strains in the concrete next to the tendon.

This leads inevitably to the concept that we will want our tendon to be unbonded. This will avoid pushing the tendon strain up so high that we snap the tendon. It will remove problems associated with the difficulty of providing controlled bond between tendon and concrete, and will give great freedom to the design engineer, both in form of the concrete cross-section, and in the layout of the prestress.

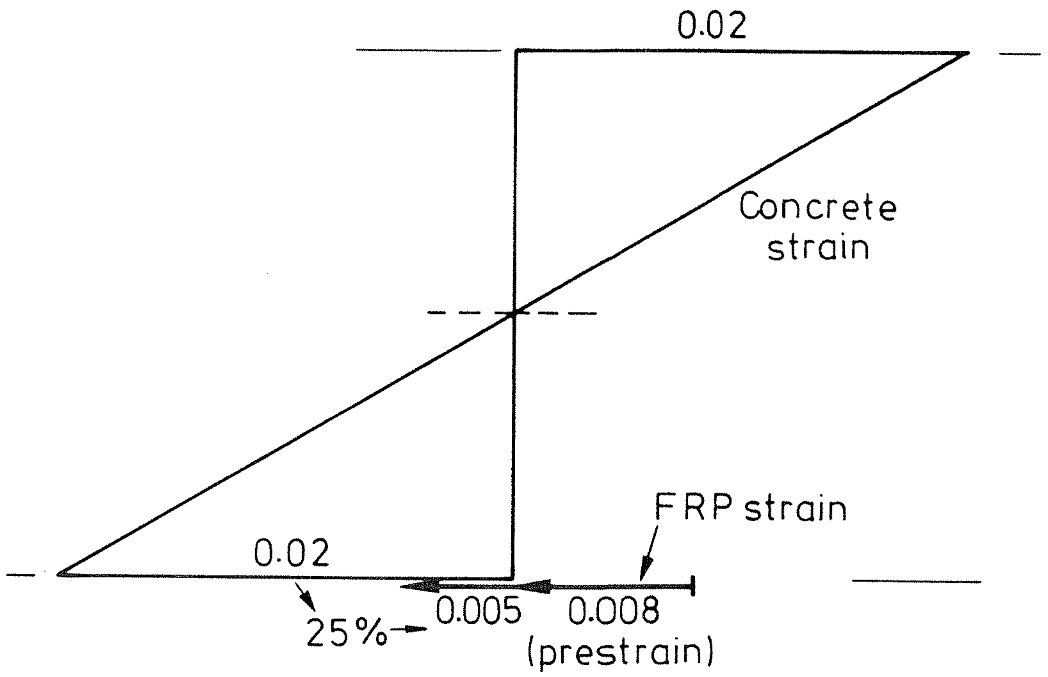

Figure 11. Strain distributions in a beam prestressed with an unbonded FRP tendon, with compression concrete confined to enhance strain capacity.

We can envisage a strain diagram, at failure of the beam, as shown in Figure 11. The concrete in compression is confined by hoop reinforcement (probably made from loops of FRP), that can sustain a strain of about 0.02. The tendon will have been prestrained to about 0.008, and if the neutral axis is at mid-depth, would pick up an additional strain of about 0.02, which would be very dangerous. But if unbonded, the extra strain induced in the tendon is likely to be no more than 25% of this, or about 0.005. This will give a total strain of 0.013, which is well within the strain capacity of a material such as a high modulus aramid like Kevlar 49.

CONCLUSION

New materials like glass, aramid and carbon fibres offer a great potential to structural engineers for the construction of non-corroding tendons and reinforcement. But these materials should not be seen as direct replacements for steel; they are materials in their own right, and we must consider how they should best be used.

It has been concluded that beams reinforced with FRP will be unable to use the full strain capacity of the bars, since the strains induced in the concrete will cause unacceptable curvatures. It is thus unlikely that there will be problems caused by bars snapping due to the lack of ductility.

For beams prestressed with FRP, the situation is different. Prestraining the tendons will make them very sensitive to additional strains induced by beam curvature. Making the tendon unbonded will offer real assurance that the tendon is not pushed off the top of its stress-strain curve, causing premature failure.

We need research about the way FRP acts compositely with concrete. We can be happy about the basic fibre properties themselves, but we must now pass on and see how these materials can best be used in practice

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