FRP REINFORCEMENT FOR DURABLE CONCRETE STRUCTURES

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Abstract:

Although the use of fibre reinforced polymer (FRP) reinforcement for the strengthening and rehabilitation of existing structures has been well accepted within the construction industry, their adoption as internal reinforcement in new buildings has been considerably slower. The reason for the limited use of FRPs as internal reinforcement can be attributed to their initial higher material costs and unique mechanical properties, the perception of which generally overwhelms contractors and designers who rather deal with the more familiar steel reinforcement. In addition, the lack of a mature set of design rules also contributs to the delay the more widespread adoption of this new technology.

FRP reinforcement can offer improved structural performance along with superior durability characteristics. In addition, composites can be engineered to have exceptional resistance to environmental factors such as freeze-thaw cycles, chemical attack and temperature variations.

This paper will draw on the ongoing work of fib TG 9.3 on FRP Reinforcement for Concrete Structures, and present some of the important aspects of structural behaviour and philosophy focusing primarily on flexural capacity, deflections, cracking, shear and bond.

1. INTRODUCTION

For more than a century steel bars have been used as reinforcement in RC structures to compensate for the low tensile strength and toughness of concrete. However, when concrete structures are exposed to moist or chemically aggressive environments, steel reinforcement is susceptible to corrosion. Corrosion of the reinforcement can lead to premature deterioration of the mechanical performance of the structure and subsequent failure.

The need to find durable and cost effective solutions to the problem of corrosion in RC structures is one of the main reasons for the increasing interest in the use of advanced composite materials as internal reinforcement in concrete. Composites can be engineered to be highly corrosion resistant in specific environments such as freeze-thaw cycles, chemical attack, and temperature variations and can increase the design life of new concrete structures.

The lack of formal design standards is a significant barrier for the extensive use of FRPs in construction. The first draft design standards were published in Japan [1,2,3] followed by design recommendations in Europe by the EUROCRETE project [4], Canada [5] and United States by the ACI [6]. The ACI recommendations produced by ACI committee 440 have been upgraded several times and many European Countries published codes or recommendations for FRP reinforcement including strengthening. The *fib* (International Federation for Structural Concrete) established Task Group 9.3 in 1996, with the aim of developing design guidelines for the design of concrete structures, reinforced, prestressed or strengthened with advanced composites. A recent collaboration between *fib* TG9.3 and the Marie Curie RTN Network En-Core [7] resulted in the publication of Bulletin 40 [8], a state-of-the-art report on the use of FRPs as internal reinforcement for concrete structures.

This paper presents the general philosophy and the various important design considerations for FRP reinforcement as dealt with by the fib TG 9.3. The paper concentrates on aspects of structural behaviour such as flexure, shear, serviceability limit states and bond.

2. MATERIAL CHARACTERISTICS

FRPs are anisotropic materials characterized by a perfectly elastic behaviour up to failure and, in the direction of the fibres, can develop higher tensile strength than conventional steel reinforcement. The elastic modulus of FRP materials used in construction generally varies between 20% (for glass fibres) to 75% (for carbon fibres) of that of steel. As a result, FRPs can lead to RC structures with a very different behaviour from conventional RC. Figure 1 shows the generic mechanical properties of FRP reinforcement according to the type of fibres used in their manufacture [9].

FRP materials, in general, have a low compressive strength, due to the low buckling strength of the individual fibres. However, this is usually not a major concern since, in the majority of civil engineering applications, FRP is predominantly used to resist tensile forces. Due to the particular mechanical properties of FRPs, and especially their lack of ductility, FRP RC structures are normally governed by brittle modes of failure. Based on these considerations, both construction techniques and design philosophy need to be carefully reassessed [9].

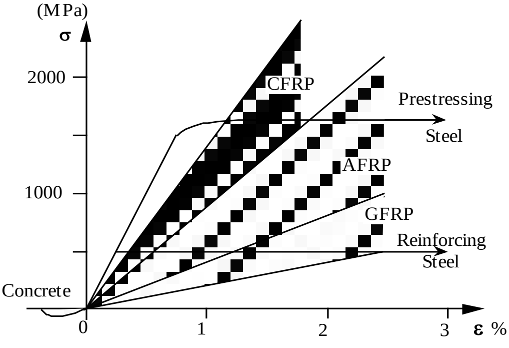

Figure 1: Stress-strain characteristics for concrete and reinforcing materials [9]

3. FLEXURE

It is well accepted that the basic principles of section analysis also apply in FRP RC [10]. For flexural resistance, the amount of required reinforcement depends on the stiffness and strength of the composite material. The FRP strength to stiffness ratio is an order of magnitude greater than that of steel and this has a significant impact on the distribution of stresses along the section. When considering a balanced section, as usually desired in steel RC design, the neutral axis depth for the equivalent FRP RC section is relatively small, as shown in Figure 2. For such a section this implies that a larger proportion of the cross-section is subjected to tensile stress and that the compressive zone is subjected to a greater strain gradient. Hence, for a similar cross-section as that used for steel RC, much larger deflections and crack widths are to be expected. Furthermore, anchoring of the FRP rebars becomes more difficult due to the high strains developed in the tensile reinforcement [11].

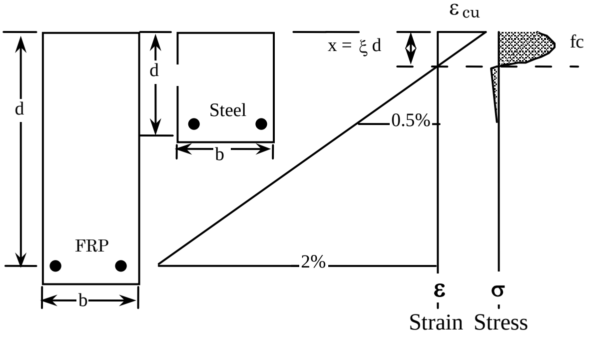

Figure 2: Strain distribution for a GFRP RC section [11]

If all the other modes of failure are avoided, flexural capacity is limited either by crushing of the concrete in compression or rupture of the FRP reinforcement in tension. Although both modes are brittle and undesirable, the approach currently adopted is to accept that FRP RC sections will be over-reinforced and that the ultimate failure will be by concrete crushing rather than by reinforcement failure.
The tensile rupture of FRP reinforcement depends on its type, but also on its bond characteristics. High bond demand around the crack can lead to bond slip, and that would result in violation of the plane-sections assumption and lead to higher deformations. In addition, high surface shear stresses will have a knock down effect on the FRP strength, leading to lower strength compared to the uniaxial material strength. Previous work at the University of Sheffield [10,12] has looked at the issue of a suitable design philosophy and has arrived at a new approach which will be discussed later on in this paper.

4. SHEAR

Shear behaviour of RC members is a complex phenomenon that relies on the development of internal carrying mechanisms the nature of which is still not well understood. Nevertheless, it has been recognised that the shear resistance of RC elements is determined mainly by the contribution offered by the un-cracked compression zone, aggregate interlock, dowel action and, when provided, shear reinforcement. The development of all of these basic mechanisms, however, depends not only on the characteristics of the concrete, but also on the mechanical properties of the reinforcing material and the interaction between concrete and reinforcement. The larger strains that are induced in the reinforcement of FRP RC elements in general result in larger deflections and wider cracks, and thus affect the development of shear resisting mechanisms. The absence of plastic behaviour in the reinforcement always leads to brittle types of failure and not much dowel strength is expected from the more flexible FPR materials. Furthermore, due to the anisotropic properties of FRP reinforcement, FRP links cannot develop their full tensile potential and, as a result, FRP RC elements can fail in shear due to the premature fracture of the shear reinforcement at their bent portions.
Despite the differences underlined above, the typical shear modes of failure that can occur in an FRP reinforced concrete element, most commonly diagonal tension failure and shear compression failure, initiate and develop in a similar manner to those of conventionally reinforced concrete members. As a result, most of the researchers working in this field have been trying to address the shear problem in a similar way as for steel reinforced concrete elements and have proposed the use of modification factors for inclusion in existing predictive code equations [13].

5. SERVICEABILITY LIMIT STATES (SLS)

There are no fundamental reasons why the principles behind the verification of SLS for FRP RC elements should not be similar to those already established in the codes of practice for steel RC elements. However, the actual limits could differ to account for differences in both short and long-term material properties. This will be discussed in the following sections.

5.1. Stresses in Materials

The stresses in materials should remain near their elastic limits to avoid long-term deterioration. At this level, stresses can be evaluated using elastic section analysis. Concrete compressive stresses could be limited to the levels indicated by Eurocode-2 [14], with a maximum 60% of the characteristic strength, but that may result in uneconomic sections; more work is recommended in this respect.

As far as the reinforcement is concerned, the limitation in FRP stress is more complex and important than for steel due to cracking of the resin and stress corrosion (of glass fibres). The *fib* document [8] presents the levels of stress given by other standards, but has not prescribed new values.

5.2. Deflections

Under similar conditions, in terms of concrete, loading, member dimensions and area of reinforcement, FRP RC members would develop larger deformations than steel reinforced members. This is mainly due to the lower modulus of elasticity of the FRP rebars, but is also influenced to a certain extent by the differences in bond characteristics.

As mentioned above, FRP rebars have high tensile strengths and stress-strain behaviour that is linear up to failure. This leads, under pure bending and beyond the crack formation phase, to almost linear moment-curvature and load-deflection relationships up to failure. Despite this brittle behaviour, FRP elements are capable of achieving large deformations that are comparable to those of steel RC elements.

The allowable overall deflection depends on the importance of a given structural member, the type of action and the type of structure being considered. To satisfy the SLS of deflection, codes of practice for steel RC specify a minimum element thickness by limiting the ratio of the element's effective span to its effective depth. Alternatively, deflections can be calculated and checked to be less than predefined limits that are normally taken as a certain percentage of the effective span of the member. Eurocode-2 [14], for instance, typically limits the design deflections to either span/250 or span/500. Though the span/depth limits are still valid, the span-to-depth ratios need to be redefined.

There are two main approaches to determining deflections of FRP RC. The first one involves modifying the ACI equations, which are based on the second moment of area of cracked and uncracked sections, as originally proposed by Branson [15]. Though there are numerous modifications and bond correction factors, these empirical modifications lack a fundamental base and are in general limited in their application. The second approach is used by Eurocode-2 [14] (and Model Code 90), which appears to be more fundamental and to be almost directly applicable to FRP RC. There are several recommendations for minor modifications to these equations and the *fib* TG 9.3 will adopt one of them.

5.3. Cracks

Control of cracking in steel RC members is important for aesthetic purposes, for mitigating the risk of corrosion of steel rebars and for preventing water leakage. When FRP reinforcement is used, corrosion is not the main issue, however, crack widths have to be controlled to satisfy the requirements of appearance and specialised performance.

Cracking of RC elements is normally controlled by implementing simple reinforcement detailing rules. Alternatively, the maximum crack width can be calculated and checked not to exceed predefined limits. The predefined limits have been relaxed for FRP RC structures and are generally of around 0.5 mm [8]. Most equations proposed for crack prediction are empirical and of limited applicability whilst the Eurocode-2 [14] approach appears to work with minor modifications.

6. BOND

Bond between concrete and FRP reinforcing bars is the key to developing the composite action of FRP RC. To secure composite action, sufficient bond must be mobilised between reinforcement and concrete for the successful transfer of forces from one to the other. Bond interaction of deformed steel bars is different from that of FRP bars in many ways. In the case of the deformed steel bars the interaction arises primarily from the mechanical action of the bar lugs against concrete. Once the tensile stress of the concrete is exceeded this mechanical bond action leads to primary cracking extending to the surface. In addition, multiple secondary cracks can develop from the lugs along the length of the bar in-between the primary cracks. These secondary cracks normally are inclined to the primary and get trapped inside the concrete matrix without surfacing. In the case of FRP bars, with these lower elastic modulus and lower surface undulations, bond interaction has more of a frictional character. Bond failure in steel bars is by crushing of concrete in the vicinity of the lugs whereas in FRP it is largely caused by partial failure in the concrete and some surface damage on the FRP.

The bond splitting behaviour of FRP bars to concrete is expected to vary from that of conventional steel bars due to their lower modulus of elasticity, lower shear strength and stiffness in the longitudinal and transverse direction and the high normal strains expected before failure. However, despite the fact that a lower maximum bond strength is expected from FRPs, the more ductile nature of the bonding mechanism can lead to a better distribution of the bond stresses and, hence, lead to reduced anchorage lengths.

The basic development length of FRP bars is calculated in the various design guides by using modifications to the already elaborate existing equations through additional factors to account for the various FRP types. However, there is still a lot of debate amongst researchers as to the accuracy of these approaches.

7. DESIGN PHILOSOPHY

Conventional RC codes of practice assume that the predominant failure mode is always ductile due to yielding of the flexural reinforcement. However, this is not the case for FRP RC design guidelines, which assume that brittle flexural failure would be sustained due to either concrete crushing or rupture of the FRP reinforcement. In addition, existing codes of practice have fundamental structural safety uncertainties, which in conjunction with the change in the type of failure and other design issues relevant to FRP RC, have major implications for the structural design and safety of FRP RC elements. Neocleous et al work [12] revealed that the application of the current partial safety approach (limit-state design) does not lead to uniform safety levels and results in RC elements with larger amounts of reinforcement (or larger dead to live load ratios) being safer. In addition, the resistance-capacity margins between the flexural mode of failure and the other modes of failure are quite variable and the designer has no reliable means of assessing them. Hence, if there is flexural over-strength, codes of practice do not provide information on the failure mode that will actually occur first and at which load level. It was also shown that concrete crushing is the most probable type of flexural failure, as the ultimate tensile strength of FRP is rarely attained in normal concrete sections. Furthermore, the use of a partial safety factor for longitudinal reinforcement (γ_{FRP-L}) may not be essential for the design of FRP RC, as long as the flexural failure intended at design is due to concrete crushing.

Another issue arises from the assumption that the application of γ_{FRP-L} will always lead to the desired type of flexural failure. This is not always valid, especially for the large values of γ_{FRP-L}, which are normally expected to lead to flexural failure due to FRP rupture. However, it was highlighted that application of high safety factors would actually lead to concrete crushing and will not necessarily improve the safety of elements.

Additional issues that require further investigation arise when considering the long-term behaviour of FRP RC elements. The application of multiple strength-reduction factors, intended to account for the long-term effects of FRP reinforcement, may not lead to the mode of failure aimed at the short-term design and may often lead to uneconomical designs. It is therefore essential to develop appropriate design provisions that take into account the long-term behaviour of FRP reinforcement. One possible solution is to use the short-term properties for the limit state design and, subsequently, to verify that (at various time intervals) the applied stress is less than the FRP strength.

Based on the above findings, a new design and safety philosophy was developed for FRP RC. The basis of design is still limit-state design, but with the main aims being the attainment of a predefined failure-mode-hierarchy and the satisfaction of target safety levels. The proposed philosophy can be implemented through a framework that enables the determination of appropriate safety factors and forms part of an overall code development process [12]. This approach was adopted as it would help new materials to be adopted as they are developed, without the need for re-writing the design guide each time. Hence, the engineer or code committee will be able to select whether concrete crushing, bond failure or shear failure is to be the predominant mode of failure for design purposes (but also allow the second failure mode to be determined). This approach will always ensure the correct safety level in a structure without undue conservatism in the second failure mode.

8. CONCLUSIONS

Although FRP materials have fundamentally different mechanical characteristics than steel, the design of FRP RC elements can be based on the same fundamental principles as far as flexural design, shear design, cracking and deflections are concerned. However, a different philosophy of design is needed which addresses the issue of safety at a more fundamental level.

Despite the extraordinary progress made to date in the use of these advanced composite materials, many aspects of their structural behaviour had to be addressed in detail before their full potential can be exploited in new construction. First generation design recommendations incorporating the use of FRPs in RC structures are already available, but a huge international effort is taking place which will soon produce more advanced guidelines.

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