The Institution of Structural Engineers

AUGUST 1999

Interim guidance on the design of reinforced concrete structures using fibre composite reinforcement

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The Institution of Structural Engineers
11 UPPER BELGRAVE STREET, LONDON, SW1X 8BH

Constitution of Task Group

The draft of this report, which was prepared as part of the EUROCRETE Project, was reviewed, on behalf of the Institution, by a task group whose membership was:

S J Matthews, BSc(Eng), MSc, DIC, CEng, FIStructE, MICE (WSP Civils Limited), *Chairman*
C J Burgoyne, MA, MSc, PhD, CEng, MIStructE, MICE (University of Cambridge)
C K Jolly, MSc, PhD, CEng, FIStructE, MICE (University of Southampton)
J L Clarke, MA, PhD, CEng, MIStructE, MICE (The Concrete Society, *formerly with* Sir William Halcrow & Partners)

Secretary to the Task Group
S M Doran, BSc(Eng) AKC PhD CEng MICE FRSA (The Institution of Structural Engineers)

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Preface

Foreword

Fibre reinforced plastic (FRP) rods or grids, consisting of high modulus continuous fibres combined with an appropriate resin, have been developed for use as embedded reinforcement for concrete over the past 15 years or so and are now commercially available. While they have been used for a number of demonstration structures world-wide, design is not currently covered by appropriate Standards. The Institution of Structural Engineers considers that it should be at the forefront of new and emerging technology. It has, therefore, produced this document as interim guidance on the design of concrete structures reinforced with FRP. It is not intended to provide the definitive approach in any situation, as in all circumstances the party best placed to decide on the appropriate course of action will be the engineer undertaking the particular project.

The guidance given herein is not specific to any particular FRP material. This Guide is in the form of suggested changes to the British design Codes, which are based on information available in the public domain. The approaches adopted are in line with similar recommendations being developed elsewhere in the world, principally in Japan, the USA and Canada.

When considering the use of FRP materials, it must be borne in mind that their properties and characteristics differ significantly from those of conventional steel. Thus, those limit states that are critical for conventional steel reinforced structures may not be critical for those with FRP reinforcement. For the latter, other factors may dominate the design, as indicated in the text. In time, design approaches which more accurately reflect the behaviour of FRP reinforcement in concrete will be developed.

There are some situations in which the Guide does not recommend the use of FRP reinforcement, because the current materials are unsuitable or because of a lack of knowledge of their behaviour. Probably the most important is fire; FRP reinforcement is not currently recommended for situations in which fire is a major design consideration.

Where material properties are quoted in the Guide they should be taken as being for preliminary design purposes only; the final design should be based on actual properties of the materials. As there are currently no Standards covering the properties of FRP materials, the values to be used in the design will have to be determined by the manufacturer, to the satisfaction of the user, or by appropriate independent testing.

Status of the report

This Guide is intended for use by engineers familiar with the design of conventionally reinforced concrete structures in accordance with the current design Codes but who have little, or no, experience of the use of FRP rods or grids as embedded reinforcement. It does not cover the use of FRP as prestressing tendons nor does it cover the use of FRP material applied as reinforcement to the outer surface of the structure. Finally it does not cover the use of short chopped fibres incorporated into the concrete during mixing.

In the absence of an authoritative Code, this document is intended to provide safe design guidance. This is based on the information presently available, but it does not necessarily yet reflect tested good practice. As more experience becomes available, the guidance may be modified. As with any emerging technology, engineers are advised to proceed with caution and to advise their client if they are unsure of the technology.

Acknowledgments

Much of the initial work on the suggested modifications to the design Codes given in this Guide was carried out as part of EUROCRETE, an international collaborative research project under EUREKA, which was partly funded by the Department of Trade and Industry (DTI)/EPSRC Link Scheme, Structural Composites Programme.

The membership of the EUROCRETE consortium was:

Euro-Projects (LTTC) Ltd
Sir William Halcrow and Partners Ltd
Laing Technology Group Ltd
Sheffield University
A S W Construction Systems
Tech Build Composites (now Fibreforce Composites)
D S M Resins (now DSM.BASF Structural Resins)
Vetrotex
SINTEF
Statoil
Norsk Hydro

PART A

General introduction to FRP reinforcement

PART A General introduction to FRP reinforcement

A.1 Introduction

A.1.1 Background

Traditionally, concrete structures have been reinforced with steel bars or have been prestressed with steel wires or tendons. Generally, embedded steel is protected by the alkalinity of the concrete, resulting in a durable structure. However, for structures in highly aggressive environments, such as marine structures or bridges subjected to de-icing salts, the protection surrounding the steel is overcome and corrosion can take place. This may lead to cracking and spalling of the concrete and eventually the structure may become unserviceable or unsafe. Similarly, incomplete grouting of ducts has led to serious problems for post-tensioned structures.

There are many approaches to improving the durability of embedded steel. They have mainly been measures to improve the quality of the concrete, for example, by reducing its permeability. However they have had limited success in some applications and hence designers have turned to alternative reinforcement such as epoxy coated bars or, more recently, advanced composites, made of fibres embedded in a suitable resin, which are generally known as FRPs. These may be used as reinforcement or prestressing tendons for concrete structures.

There have been several major international conferences dealing with FRP materials, both as reinforcement and for prestressing concrete structures[1-5].

Currently, the most suitable fibres are glass, carbon or aramid. Each is a family of fibre types and not a particular one. They all have a high ultimate strength (in the region of 3000N/mm^2) and a stiffness which ranges from about 50kN/mm^2 for glass to over 200kN/mm^2 for some carbons. The fibres all have a linear elastic response up to ultimate load, with no significant yielding. The fibres are relatively difficult to handle and anchor into concrete and hence are generally combined with resins to form composite (FRP) rods or grids, as described later.

The main advantages of fibre composites are that they are lighter and stronger than steel and, with the correct resin and fibre combination, should prove to be more durable. However, there are possible disadvantages which include the lack of any yield at the ultimate load. This will be considered further in the section on design.

A.1.2 Resins

There is a wide choice of resins available, many of which, though not all, are suitable for forming composites. (The Draft Canadian Highways Bridge Design Code specifically prohibits the use of polyester resins for embedded FRP material[6]).The choice will depend on the required durability, the manufacturing process and the cost. Thermosetting resins are generally used but they have the major disadvantage that, once they have fully cured, the composites cannot be bent to form hooks, bends and similar shapes. One possible alternative is the use of suitable thermoplastic resins, which are now being developed. With these resins the composite components can be warmed and bent into the required shapes. On cooling the full properties of the resins are restored. However, there is likely to be distortion of the fibres in the region of the bend, which will lead to a reduction of the strength locally.

A.1.3 Manufacturing processes

The most widely-used manufacturing process is pultrusion. The fibres, which are supplied in the form of continuous rovings, are drawn off in a carefully controlled pattern through a resin bath which impregnates the fibre bundle. They are then pulled through a die which consolidates the fibre-resin combination and forms the required shape. The die is heated which sets and cures the resin allowing the completed

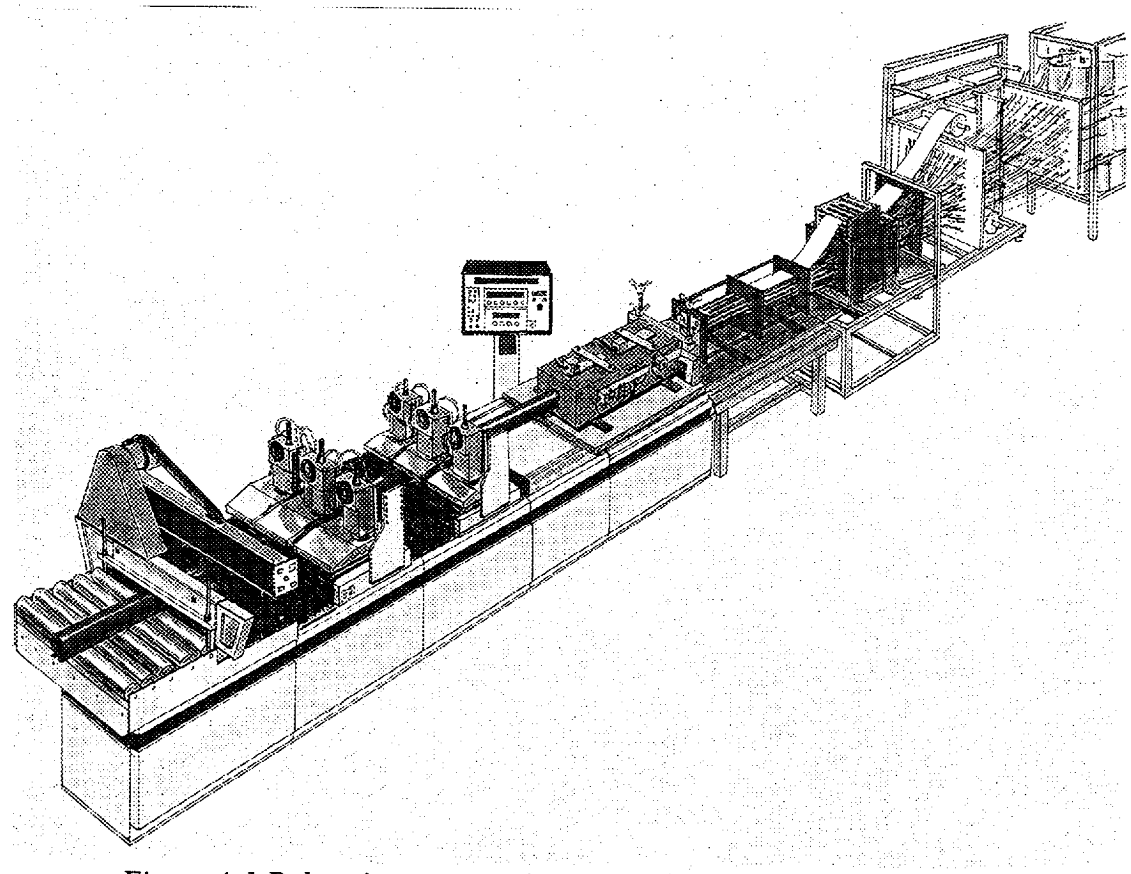

Figure A.1 Pultrusion process (courtesy Fibreforce Composites Ltd)

composite to be drawn off by suitable reciprocating clamps or a tension device. The process enables a high proportion of fibres to be incorporated in the cross-section and hence relatively high strengths and stiffnesses are achieved. However, the sections have a smooth surface, which provides insufficient bond if they are to be used as reinforcement in concrete. Hence, a secondary process, such as overwinding with additional fibres, is required to improve the bond.

As indicated earlier, thermosetting resins are generally used at present. Once formed these cannot be bent into the range of shapes currently used by the concrete industry, and hence different manufacturing processes are required to form specials. Filament winding, in which resin impregnated fibres are wound round a mandrel of the required shape, has been used to manufacture shear links. Other manufacturing processes, such as filament arranging, are being developed for more complex shapes.

Fibre composite two-dimensional reinforcement grids, and even three-dimensional grids, are made by a number of different patented processes.

A.1.4 Short-term properties

The physical properties of the composite will depend on the type and percentage of fibres used. Typically a pultruded composite would have about 65% of fibre by volume. Thus with glass the ultimate strength might be 1200N/mm^2 rising to 2000N/mm^2 for carbon. The elastic modulus will be about 40kN/mm^2 for glass fibre composites and may be 150kN/mm^2 for carbon fibre composites. As composites are not currently manufactured to a common Standard, their properties will vary from one manufacturer to another. Thus all design must be on the basis of the actual properties of the composites, as supplied by the appropriate manufacturer.

A.1.5 Long-term properties – creep rupture

Creep rupture, or stress rupture as it is often known, is the process by which a material with a permanent high load applied to it will creep to failure. (This will be particularly important for prestressed structures and for reinforced structures with a high permanent load.) There is theoretical justification for a relationship of the form $\sigma = a + b\ \log(t_b)$

where σ is the applied stress, t_b is the time to break and a and b are suitable constants. There is a reasonable amount of data to confirm these predictions for a few years under stress but there is still a degree of uncertainty about the exact form of the long-term response. For this reason relatively large factors of safety have been proposed to allow for the uncertainty, which will be revised as more test data become available.

The phenomenon of creep rupture does not appear to have any effect on the short-term strength. Thus a FRP element that has been loaded to a significant level for a period of time will retain its initial short term strength. Similarly the stiffness of FRP is largely unaffected by permanent load.

A.1.6 Long-term properties – chemical attack

The effect of corrosive actions on the fibre is complicated and varies from fibre to fibre. Some corrosive elements attack the internal bonds, breaking the long-chain molecules into short lengths, reducing the strength but not the stiffness. Other factors attack the fibre from the outside, physically removing some of the fibre, which will change both the strength and the effective stiffness.

The effect of the resin is to shield the fibres from chemical attack. The durability of the resin itself, and its permeability to aggressive substances, will affect the durability of the FRP bar. Because of the lack of research data, relatively large safety factors have to be applied to the measured short-term properties. Future testing will allow these factors to be modified and will indicate the most suitable combinations of resins and fibres to resist attack.

A.2 Review of applications

A.2.1 Bridges

The widest use of fibre composites in concrete structures has been for prestressing. This is probably because changing from steel to the new materials leads to little, if any, change in the design process. A number of bridges have been built worldwide, generally with conventional steel for the unstressed reinforcement.

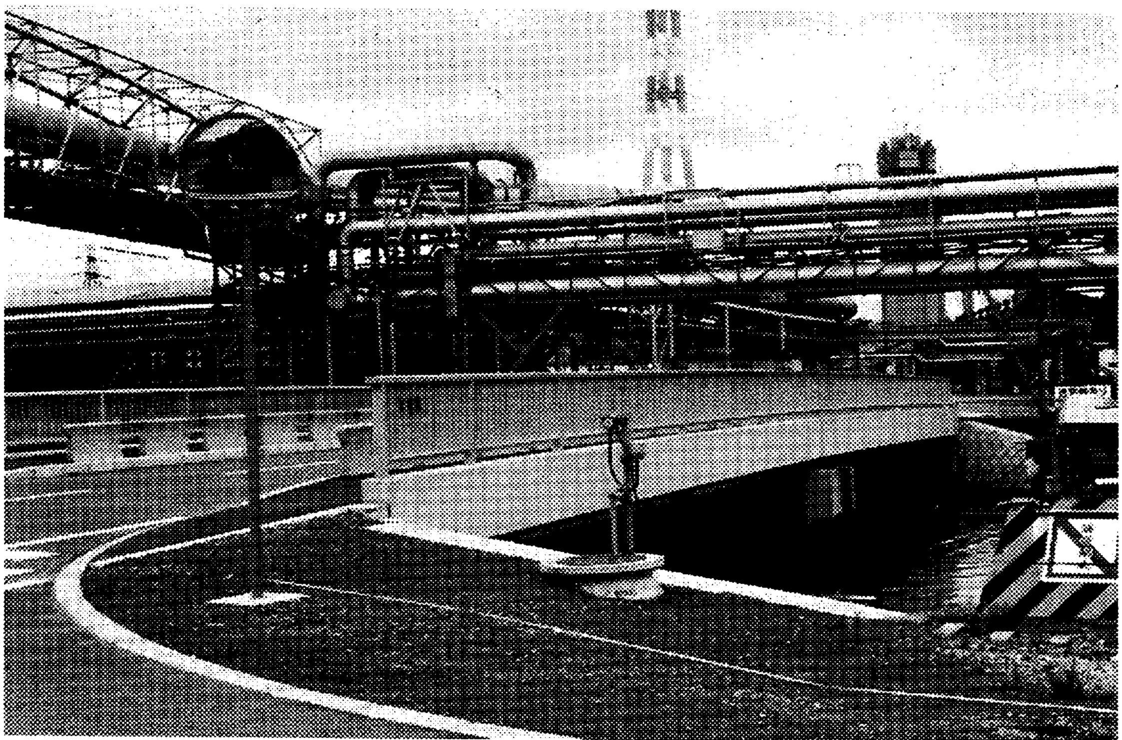

Figure A.2 Typical Japanese Bridge prestressed with carbon fibre tendons
(courtesy Mitsubishi Kasai Corporation, Japan)

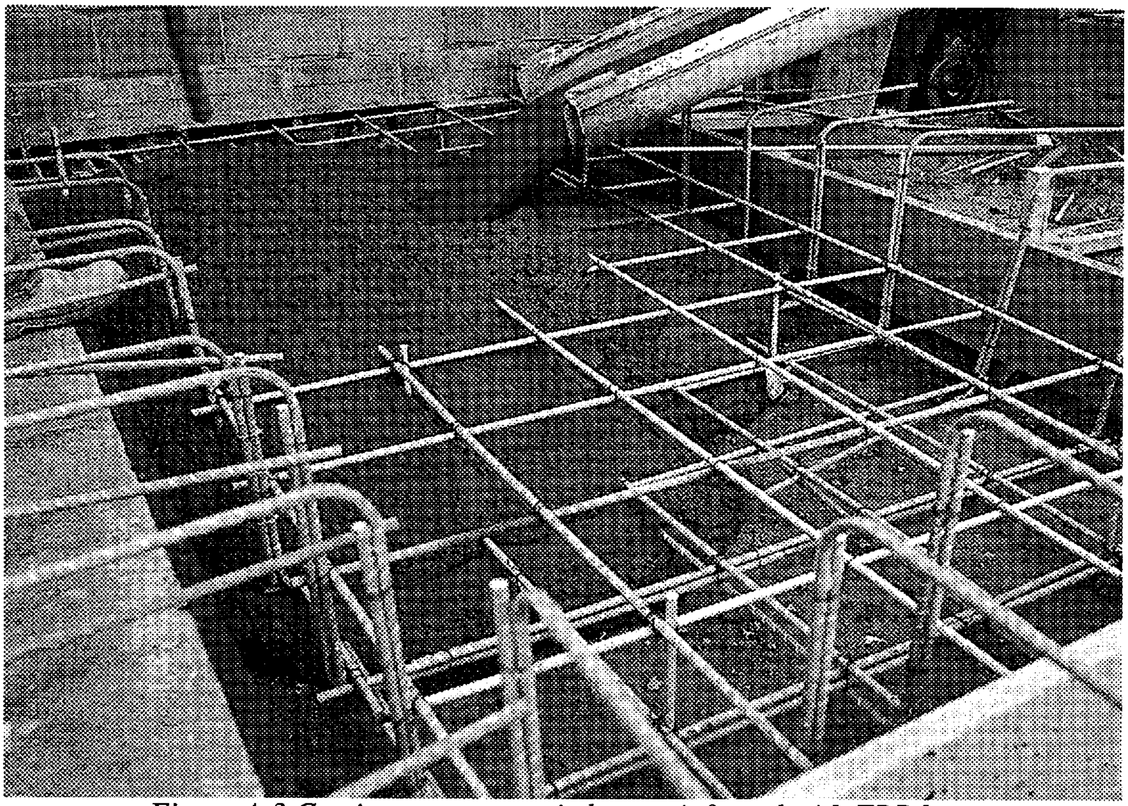

Figure A.3 Casting non-magnetic base reinforced with FRP bars
(courtesy Hughes Brothers, USA)

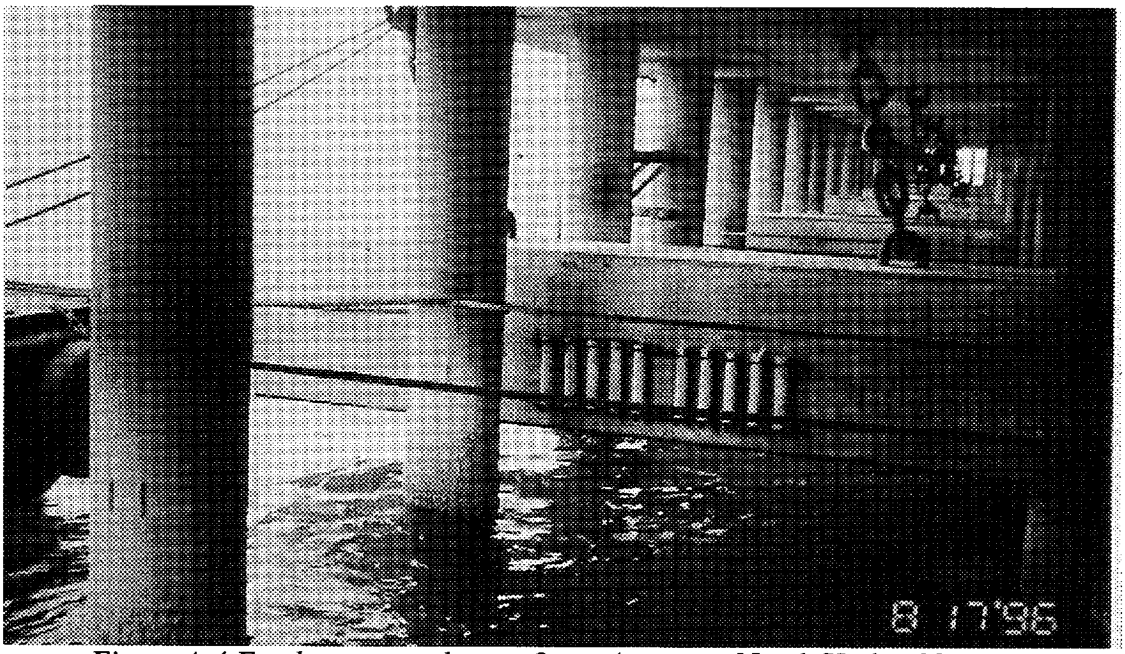

Figure A.4 Fender support beam, Qatar (courtesy Norsk Hydro, Norway)

In Germany and Austria a total of 5 road bridges and footbridges have been built prestressed with glass-fibre composite tendons. The first highway bridge, the Ulenbergstrasse Bridge in Dusseldorf, was opened to traffic in 1986 and has been monitored and load tested periodically since(7). In Japan the emphasis of development has been on carbon or aramid composites. At least 10 bridges have been built to date (1998)(8, 9). Carbon fibre has also been used in Germany for one bridge(10) and aramid fibre for a cantilevered roadway in Spain(11).

The first footbridge in Britain, and probably in Europe, using glass fibre composite reinforcement was built at Chalgrove in Oxfordshire in 1995(12). The Oppegard footbridge was built in 1997 on a golf course near Oslo(13). It has a span of about 10m and consists of twin arched beams, reinforced with glass FRP rods and stirrups, with horizontal prestressed ties containing aramid tendons.

In North America part of one bridge in South Dakota has been stressed with glass and carbon tendons(14) and a bridge in Calgary contains carbon fibre composite strands(15). A number of other bridges are currently being planned.

A.2.2 Other structures

Apart from bridges, fibre composite reinforcement has been used in a number of different applications. The most extensive has been for the reinforcement of sprayed linings for tunnels in Japan, using a grid of glass and/or carbon fibres in resin, which has been developed as a direct replacement for conventional welded steel fabric. In the USA glass fibre composite bars have been used in a number of applications such as chemical plants where high durability is required. In addition it has been used under sensitive electronic equipment where the stray electrical currents in steel reinforcement would be a problem. Descriptions are given in the proceedings of various conferences(1–4).

Other experimental structures which have been built or tested in the laboratory include post and panel fencing, cladding panels, precast elements for a retaining wall system and a replacement fender support beam for a jetty in the Middle East(13). A number of other applications are being actively considered.

A.3 Research

A.3.1 Brief overview of research

Throughout the world there are programmes of work developing the use of FRP in concrete structures. In Japan the emphasis has been chiefly on prestressing while in North America and in Europe, considerable effort is being put into the development of embedded FRP reinforcement for concrete. The main areas being investigated around the world are:

- selection of suitable resins and fibres
- development of appropriate manufacturing techniques
- investigations to determine the durability of FRP rods exposed to aggressive environments either directly or embedded in concrete
- determination of the structural behaviour through testing and analytical techniques
- economic and feasibility studies
- development of case studies of trial structures and components
- development of suitable design guidance

A.3.2 Tests on reinforced concrete members

Much of the early work was concerned with the bond between FRP reinforcement and concrete. Over the last 10 years or so there have been many programmes of tests on simple beams, considering both the flexural and the shear behaviour. Limited work has been carried out on frames, columns and slabs. Relevant published data have been used to formulate the modified design rules. They are reviewed in Part D, and are compared with the predictions of the modified rules.

Analytical work is being carried out by a number of research workers in parallel with the experimental work, considering such topics as ductility.

A.3.3 Durability

One very important aspect of the use of FRP rods embedded in concrete is their durability. Surprisingly, this was not considered to any great extent in any of the early research programmes worldwide. However, many resins and fibres degrade in the highly alkaline concrete environment. (As indicated earlier, the Draft Canadian Highways Bridge Design Code specifically prohibits the use of polyester resins for embedded FRP material(6)). Thus the manufacturers' claims of long life have still to be demonstrated.

Currently, trials are being carried out on the resins and fibres in isolation and in the form of the composite, both in a range of artificial aggressive environments and embedded in concrete, with a view to developing the necessary confidence in the long-term properties of the materials. The accelerated laboratory testing is being backed up by data from specimens on exposure sites.

A.3.4 Thermal effects

The coefficient of thermal expansion along a FRP bar is controlled by the fibres and will be significantly lower than that of concrete. Transversely, it will be due largely to the resin and will be significantly higher. Cracking of some elements prestressed with an aramid FRP has been attributed to this high transverse expansion.

A.3.5 Experimental structures

An important aspect in the development of new materials is the construction of demonstration structures, as outlined above. While they may not be economic, because the materials themselves are not yet fully understood and the design approaches are not fully developed, they give valuable experience of practical construction aspects and an indication of the long-term performance. They thus develop the necessary confidence in the new materials.

A.4 Design of reinforced concrete elements

A.4.1 Introduction

Current design codes consist of a mixture of basic principles and simple rules of thumb. The latter are based on one hundred or so years of experience of structures reinforced with steel. The designer is given no guidance as to how such requirements should be modified when changing to different reinforcement materials. Currently Codes are being developed in Japan(16, 17) and in North America(6, 18–19). Within EUROCRETE, modified design rules have been developed for the British design Codes, BS 8110 for buildings and BS 5400 for bridges. The detailed changes are given in Parts B and C of this document respectively.

In formulating new design methods two approaches are possible. The first is to develop completely new methods and the second is to adapt the existing ones. The former requires considerable experimental data and would thus tend to delay the introduction of new materials. Hence the approach that has been adopted when developing the proposals in the present document, in line with other development work around the world, has been to modify existing design guidance and to justify the changes by comparing predicted behaviour with the limited data available. The resulting designs may not be the most economic use of the new materials but they will, on the basis of present knowledge, result in safe structures.

In modifying the design guidance, only those concerned directly or indirectly with the reinforcement need to be changed. Those covering general principles and those concerned with the geometry of the structure remain unaltered. A detailed review of the data is contained in Part D of this document; the following sections give an outline of the changes that are required, each with a summary of the key points.

Summary: Significant changes required, as given in Part D

A.4.2 Analysis

Because FRP materials have a straight-line response to ultimate load, with no yielding, it is appropriate to use only elastic methods of analysis. Some experimental work has shown that there can be some limited pseudo-ductility at ultimate load due to cracking in the concrete, but for design purposes it should be assumed that no redistribution of the elastic bending moments and shear forces will take place.

Summary: Use elastic analysis, no redistribution

A.4.3 Partial safety factors

The properties of FRP materials embedded in concrete, both the effective strength and the effective stiffness, may change with time, due to the possibility of loss of material due to alkali attack. The amount of change will depend on the types of resin and fibre used in the composite. It is therefore necessary to apply appropriate factors of safety to the short-term values to take account of the changes. Because of the lack of knowledge of the changes with time, the partial safety factors have been set fairly high in this design Guide, considerably higher than those currently used for steel. (It should be noted that, in the design of conventional steel reinforcement, it is assumed that the concrete provides the necessary protection, no corrosion takes place and hence the properties of the steel are assumed to remain constant throughout the life of the structure). Once the long-term properties of particular materials are better understood, there should be considerable scope for reducing the factors, leading to more economic structures.

Summary: Modified partial safety factors required

A.4.4 Cover and durability

The design requirements intended to achieve durability of the steel reinforcement are not appropriate when using FRP reinforcement. Thus the quality of the concrete will be governed mainly by strength considerations. The cover requirements will be controlled by the aggregate size and the size of the reinforcing bar. FRP reinforcement is generally not recommended for structures for which fire is a significant design consideration. If it is used for such structures, the cover requirements will be dictated by the need to protect the FRP. Design crack widths will be controlled by aesthetic considerations and, possibly, watertightness of the structure.

Summary: Cover and crack width requirements not influenced by environment

A.4.5 Bending of beams

The basic principles of the behaviour of a beam made from elastic materials in bending, such as plane sections remaining plane, should not depend on the type of reinforcement material provided it has sufficient bond. The design equations and design charts given in the Codes are not appropriate when using FRP as they assume yielding of the reinforcement at ultimate. FRP reinforcement generally has a significantly lower strength in compression than in tension; the strength of any bars in compression should be ignored when calculating the bending capacity of the cross-section.

The effect of significant load reversals on the tensile strength of FRP bars is not well understood, though this is unlikely to be a major design consideration for many structures.

The lack of energy absorption, due to the absence of any yield, may make FRP reinforcement unsuitable for use in seismic applications. However, this aspect is outside the scope of the present document.

Because of the relatively low stiffness of FRP, it is likely that failure will occur by compression of the concrete and not by reaching the ultimate capacity of the tensile reinforcement.

Summary: Basic elastic principles of section analysis unaltered but design equations no longer applicable

A.4.6 Shear

Most design codes use an empirical approach to determine the shear capacity of beams. The total capacity is taken to be the sum of the capacity of the concrete cross-section and that of the shear reinforcement. While this is not a true representation of the behaviour, it has been proved to give an adequate margin of safety for conventional steel reinforcement. Initially, the equations have been modified for beams with non-ferrous reinforcement as follows:

- Shear capacity of the concrete cross-section: use the same design approach as for steel but for calculation purposes replace the area of non-ferrous reinforcement provided with an equivalent area of steel, on the basis of the modulus of elasticity, in the tables/equations in BS 8110 and BS 5400 (i.e. transform the area on the basis of the modular ratio)
- Shear reinforcement: use a limiting strain of 0.0025 (c.f. a limiting stress of 460N/mm^2 in BS 8110).

One important practical aspect that has been highlighted by a number of researchers is the strength of FRP shear reinforcement locally at corners. Depending on the manufacturing process used, the strength can be significantly less than that of the straight portions, which will be an important design consideration.

Summary: Modified design approach allowing for stiffness of FRP

A.4.7 Serviceability

Because of the lower stiffness, deflections and crack widths may become dominant design criteria. However, as indicated above, with no limitation on crack width required from the point of view of durability, aesthetics will be the only criterion. Thus the current rules could be relaxed considerably.

For a given load, cracks in a FRP reinforced beam will generally be wider than for an equivalent steel reinforced beam. However initial studies have shown that existing formulae may be used to predict crack widths. Similarly current approaches may be used to predict the deflections, with reasonable accuracy.

Summary: Crack widths and deflections calculated by current methods

A.4.8 Slabs

Very limited work has been reported on the punching behaviour of slabs reinforced with fibre composites. The results agree with those predicted by the modified shear design approach.

Summary: Modified approach as for beams

A.4.9 Columns

As indicated when discussing beams, the behaviour of composite rods in compression is significantly different from that in tension, with much lower failure loads. Thus the strength of bars in compression should be ignored and the column designed on the basis of the concrete area alone. The approach has been adopted in the draft Japanese Ministry of Construction Guidelines[6]. There would appear to be considerable scope for using hoop reinforcement in columns, providing containment to the concrete and hence increasing its loadcarrying capacity.

Summary: Ignore reinforcement in compression, otherwise basic approach unaltered

A.4.10 Bond

Current codes specify bond stresses for conventional steel reinforcement, giving values for smooth or ribbed bars. Fibre composite bars are likely to fail to achieve the requirements for ribbed bars and hence would be classified as smooth. This would lead to the requirement for unnecessarily long anchorage lengths. Testing is being carried out worldwide on a range of different materials, using different techniques. Currently there is no agreed standard test specimen though, because splitting failures on the concrete generally do not occur, a simple cube with the rod cast in centrally should be adequate.

Because there are no agreed Standards, it will be necessary to determine the ultimate bond stress for the particular material and then use this in design with an appropriate partial safety factor.

Summary: Bond capacity specific to material used

A.4.11 Fire

FRP reinforcement is generally not recommended for structures for which fire is a significant design consideration.

The effect of fire on FRP reinforced concrete structures has received very little attention. Most of the testing of composites has been concerned with the resin burning or emitting toxic fumes. When embedded in concrete the lack of free oxygen will inhibit burning, but the resin will soften.

The critical time will be when the resin on the surface of the bar reaches its 'glass transition temperature', which is the temperature at which the elastic modulus of the resin is significantly reduced. Depending on the type of resin this will be in the region of 100–150°C. At this point the resin will no longer be able to transfer stresses from the concrete to the fibres, i.e. the bond will fail. Locally this may result in increased crack widths and hence in increased deflections. However, provided the end regions of the reinforcing rods are kept cool and are hence still adequately anchored, the safety of the structure is unlikely to be significantly affected. Collapse will only occur when the temperature of the fibres themselves reaches the level at which they start to degrade, which will be in the region of 1000°C for glass and significantly higher for carbon.

This is an area that requires considerable research to determine the actual response of reinforced structures in fire.

Summary: Not currently recommended in fire

A.4.12 Detailing and construction

Little or no work has been done on the detailing of FRP reinforcement. Hence any guidance must be based on current rules for steel reinforcement.

Reinforcement cages should be assembled with non-metallic ties; the use of conventional wire ties may lead to damage of the FRP bars and defeats the objective of obtaining non-corroding reinforcement.

Though lapping of reinforcement should be satisfactory, the relatively low bond strength will probably lead to uneconomic overlaps. Hence there is a need for simple adhesive bonding systems, that can be used under site conditions if necessary. Ideally as much of the reinforcement cage as possible would be preassembled under factory conditions to ensure that any adhesive in the connection is fully cured.

As indicated earlier, one significant difference between the present generation of fibre composite reinforcement and conventional steel is that, once formed, the composites can not be bent to form shear links, etc. Techniques are being used to fabricate such shapes but, until thermoplastic resins have been developed to the stage when site bending is possible, they will only be available in a limited range of sizes. Thus the designer will be required to work not only to standard shape codes but also to fixed dimensions. This may not be as arduous a restriction as it would appear to be at first sight. In many structures, a little thought can lead to significant rationalisation of the reinforcement schedule.

Experience to date has suggested that there should be no significant problems during the assembly of the reinforcement cage. Any cut ends of bars, or damaged areas, should be sealed with a suitable resin, in accordance with the manufacturer's recommendations. During the casting of the concrete the reinforcement will have a tendency to float, because of its low density.

Care must be taken with the storage of FRP material prior to use, to avoid damage to the resin matrix. In particular, reinforcement should be protected from the effects of prolonged exposure to UV-light which can lead to the degradation of some resins.

Further guidance on construction aspects should be provided by the manufacturer; additional advice is contained in the Japan Society of Civil Engineers recommendations(7).

Summary: Generally, more care must be taken in handling and storage on site. Most FRPs can not be bent on site

A.4.13 Health and Safety

Loose fibres on the surface of the FRP bars may cause irritation and a few people may suffer an allergic reaction to the resin matrix. Hence, during assembly of the reinforcement cage sensible precautions should be taken, such as wearing gloves. At all times current Health and Safety Regulations, as indicated by the manufacturer of the FRP material, should be followed. When cutting the material, suitable dust extraction should be used.

Once the FRP material is embedded into the concrete there will be no problems as far as Health and Safety are concerned.

Summary: Follow manufacturer's recommendations and latest Health and Safety Regulations

A.5 Design of prestressed concrete elements

As indicated earlier, a number of demonstration structures have been built using FRP as prestressing tendons. The practical applications have been backed up by extensive laboratory testing, much of it concerned with the development of suitable anchorage systems. In principle there should be few changes required when designing with fibre composite tendons rather than steel ones. One significant factor that has to be taken into account, however, is the phenomenon of stress rupture, that is the fact that a tendon stressed to a certain level will creep to failure. This will limit the applied stress in FRP tendons to, perhaps, 50% of their ultimate strength.

Detailed modifications to the relevant design clauses for prestressed structures are not included in this document.

A.6 Precast and *in situ* and composite concrete construction

The development of fibre composite reinforcement has generally been intended as a direct replacement for traditional steel. Hence work has concentrated on bars embedded in the concrete, with associated modification to the existing design rules. This is unlikely to be the most efficient use of either the fibre composite or the concrete. The next stage will be the development of more appropriate structural approaches. Unlike steel, which has the same properties in all directions, the properties of composites can be tailored to suit the particular requirements, by varying the amount, distribution and type of fibre in any given direction.

An example might be fibre composite tubes used to provide both permanent formwork and also the main structural reinforcement for columns. Adequate confinement would lead to high load capacity and higher ultimate strain in the concrete. Similarly, confinement of the compression zone of beams would lead to higher strains at failure and hence better utilisation of the reinforcement in tension.

PART B

Suggested modifications to BS 8110: 1997 Structural use of concrete

PART 1

Code of Practice for design and construction

SECTION 1. GENERAL

1.0 INTRODUCTION

The suggested changes to the Code clauses given in this Part were originally prepared as part of the EUROCRETE Project and represent the views of the Participants in that Project. They may therefore be taken to represent what is considered to be good advice at the time of publication, based on currently available information. BS 8110, in common with other British Standards, states in the Foreword that 'Compliance with a British Standard does not of itself confer immunity from legal obligations'. It also states that it has been assumed that design is entrusted to chartered structural or civil engineers. Thus in drafting this document it has been assumed that it will be used by suitably qualified engineers, who will satisfy themselves that the suggested design approaches are reasonable in the relevant application. The advice given has no legal standing; the EUROCRETE Participants and The Institution of Structural Engineers accept no responsibility for the adequacy of the contents of the document nor for any omissions.

The document lists the changes that are suggested to the clauses in BS 8110 when designing *in situ* or precast structures reinforced with FRP reinforcement in the form of rods, grids or other shapes. The document has been prepared on the basis of published work worldwide. Where appropriate, amended or replacement clauses are given. Where no specific change is indicated, the clause remains as given in the original Code of Practice. Where justification for the revised clauses is necessary some additional information is included, printed in italics, along with references to the appropriate background information given in Part D of this document.

Suggested modifications to BS 8110

Text | **Comments**

The document does not cover the use of FRP prestressing tendons; for information on the design of prestressed concrete the reader should refer to relevant specialist literature.

1.1 SCOPE

Add the following paragraph after the existing first paragraph:

'Reinforcement should consist of embedded FRP (Fibre Reinforced Plastic) rods, or assemblies such as grids, composed of continuous glass, carbon or aramid fibres combined with a suitable resin to form a composite. The guidance is not appropriate for concrete elements externally reinforced with FRP, in the form of strips or other shapes'.

It is important that the material used is of an adequate quality; this requires the choice of appropriate resins and fibres, combined in an appropriate manufacturing process which has the necessary quality controls. The suggestions have been formulated on the understanding that the design will use material that has a track record of use in service that has demonstrated adequate durability, or one that has demonstrated its suitability by means of durability trials for which test data can be provided.

Unlike steel, there are currently no agreed standard properties for FRP materials which will depend on the type and quantity of fibre, the resin, additives, the processing route and other factors. Hence design must be based on properties provided by the manufacturer. The designer must satisfy himself that the stated properties are appropriate to the particular material being used.

1.2 REFERENCES

1.2.1 Normative references

No change.

1.2.2 Informative references

No change.

1.3 DEFINITIONS

1.3.1 General

No change.

1.3.2 Terms specific to flat slabs (see 3.7)

No change.

Suggested modifications to BS 8110

Text	Comments
1.3.3 Terms specific to perimeters (see 3.7.7)	
No change except replace 'steel' by 'reinforcement'.	
1.3.4 Terms specific to walls (see 3.9)	
No change.	
1.3.5 Terms relating to bearings for precast members (see 5.2.3)	
No change	
1.4 Symbols	
Add 'γ_{me} Partial safety factor for elastic modulus of materials'.	*As the effective elastic modulus of a fibre composite material embedded in concrete may change with time, because of the possible loss of material due to alkali attack, it is necessary to apply a partial safety factor to the short-term values. Further information can be found in D.2.2 and in Part A.*
Replace 'f_y' by 'f_r'.	*The subscript* y *stands for yield. As FRP reinforcement does not have any significant yield, the subscript* r*, which stands for rupture, would seem more appropriate.*

SECTION 2. DESIGN OBJECTIVES AND GENERAL RECOMMENDATIONS

2.1 BASIS OF DESIGN

2.1.1 Aim of design

No change.

2.1.2 Design method

No change.

2.1.3 Durability, workmanship and materials

Replace 'steel' by 'reinforcement'.

2.1.4 Design process

No change.

Suggested modifications to BS 8110

Text	Comments

2.2 STRUCTURAL DESIGN

2.2.1 General

No change.

2.2.2 Ultimate limit state (ULS)

No change.

2.2.3 Serviceability limit state (SLS)

No change.

2.2.4 Durability

Replace second paragraph by:

'The environmental conditions to which the concrete will be exposed should be defined at the design stage. The designer should be satisfied that the durability of the reinforcement is adequate, taking into account the environment to which it will be subjected. Consideration may also be given to the use of protective coatings to the concrete to enhance the durability of vulnerable parts of construction'.

It is likely that the alkaline environment of the concrete itself will be the most serious durability consideration for the reinforcement. This will be reflected in the partial safety factors applied to the short-term properties of the reinforcement, see 2.4.4.1. The designer should take into account any special circumstances that might affect the durability of the reinforcement and seek appropriate advice from the supplier of the material. Further information can be found in D.2.2.

2.2.5 Fatigue

No change.

2.2.6 Fire resistance

Delete 'Recommendations ... BS 8110: Part 2: 1985'. and replace by:

'The design should be based on either Method 2 or Method 3 as given in section four of BS 8110: Part 2: 1985, with the recommended modifications. The designer should seek guidance from the supplier of the FRP reinforcement as to the suitability of the material being used and the relevant properties'.

Method 2 is the direct application of the results of fire tests, which is the most appropriate bearing in mind the current knowledge of concrete elements reinforced with FRP in fire situations. Method 3, fire engineering, will give the basis for an analytical design. In general the governing factor is likely to be the temperature at which a significant area of the surface of the resin matrix starts to soften rather than the behaviour of the composite when exposed to fire. Further information can be found in D.2.5.

Suggested modifications to BS 8110

Text / Comments

2.2.7 Lightning

Delete whole clause.

It has been suggested that lightning can cause damage to carbon fibre composite material. If carbon fibre reinforcement is used, the designer should ensure that it is suitably isolated from any possible lightning strike.

2.3 INSPECTION OF CONSTRUCTION

No change.

2.4 LOADS AND MATERIAL PROPERTIES

2.4.1 Loads

No change.

2.4.2 Material properties

2.4.2.1 *Characteristic strengths of materials*

Rename clause 'Characteristic properties of materials'.

Replace 'yield or proof' by 'guaranteed'.

Replace 'f_y' by 'f_r'.

The characteristic strength of materials used as shear reinforcement should be determined at the start of the bent portion.

Add two new paragraphs:

'The characteristic elastic modulus of the FRP reinforcement means that value of the short-term elastic modulus below which not more than 5% of the test results would be expected to fall.

Characteristic material properties should be determined by the supplier of the FRP, in accordance with agreed standard test methods'.

Additional information on the properties that should be provided by the manufacturer are given in Section Seven.

2.4.2.2 *Partial safety factors for strength of materials,* γ_m

Rename clause 'Partial safety factors for properties of materials'

Redraft clause to read:

Suggested modifications to BS 8110

Text

'For the analysis of sections, the design property for a given material and limit state is derived from the characteristic property divided by γ_m or γ_{me}, where γ_m and γ_{me} are the appropriate partial safety factors for strength and modulus of elasticity respectively given in **2.4.4.1** and **2.4.6.2**. γ_m and γ_{me} take account of differences between actual and laboratory values, local weaknesses, long-term effects, and inaccuracies in assessment of the resistance of sections. They also take account of the importance of the limit state being considered'.

2.4.2.3 ***Stress-strain relationships***

Replace sub-clause (b) by:

'(b) for reinforcement, a straight line response to ultimate load, as shown in Figure 2.2a, with the slope equivalent to the characteristic elastic modulus divided by the relevant γ_{me} from Table 2.2b'.

Replace Figure 2.2 by new Figure 2.2a.

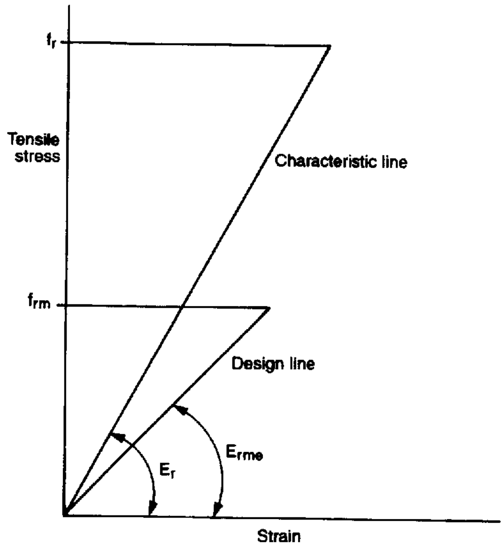

Figure 2.2a Stress-strain lines for FRP reinforcement

2.4.3 Values of loads for ultimate limit state (ULS)

No change.

Comments

The introduction of a partial safety factor applied to the modulus of elasticity is intended to take into account the fact that the effective stiffness may change with time, due to alkali attack on the composite. This contrasts with the elastic modulus of steel which, in BS 8110 is considered to be constant throughout the life of the structure. Further information can be found in D.2.2.

Suggested modifications to BS 8110

Text / **Comments**

2.4.4 Strengths of materials for the ultimate limit state

Rename clause 'Properties of materials for the ultimate limit state'.

2.4.4.1 *Design strengths*

Rename clause 'Design properties'.

In first paragraph replace 'Table 2.2'. by 'Table 2.2a'. Replace Table 2.2 by Table 2.2a, as shown below.

Table 2.2a Values of γ_m for the ultimate limit state

Material or behaviour	γ_m
E-glass reinforcement	3.6
Aramid reinforcement	2.2
Carbon reinforcement	1.8
Concrete in flexure or axial load	1.5
Shear strength without shear reinforcement	1.25
Bond strength	1.4
Others (e.g. bearing stress)	≥1.5

Note: Values in Tables 2.2a and 2.2b are for guidance and should only be used in the absence of data supplied by the manufacturer.

Add to the end of first paragraph:

'Where the response of the structure, or its resistance to loads, depends on the elastic modulus of the FRP reinforcement, the characteristic values should be divided by the appropriate γ_{me} value from Table 2.2b'.

As both the strength and the modulus of elasticity of the FRP material will change with time, it is necessary to apply a partial safety factor to both properties. The philosophy is that design will be carried out on the basis of the design properties at the end of the design life rather than at the beginning. In general, only one partial safety factor will be applied, either γ_m or γ_{me} as appropriate; the choice will depend on which factor will give a lower design strength. Further explanation is given in the notes to the relevant clauses. Further information can be found in D.2.2.

Add new Table 2.2b, as shown below.

Table 2.2b Values of γ_{me} for the ultimate limit state

Fibre type	γ_{me}
E-glass	1.8
Aramid	1.1
Carbon	1.1

Note: Values in Tables 2.2a and 2.2b are for guidance and should only be used in the absence of data supplied by the manufacturer.

Suggested modifications to BS 8110

Text	Comments
In final paragraph delete A more detailed method ... Part 2:1985'. (Remainder of paragraph unaltered.)	*The approach in Part 2 takes the 'worst credible values' which are then divided by a materials factor of 1.05. While this is an appropriate approach for properties that do not change with time, not enough is known about the long-term behaviour of FRP materials for it to be valid in this situation.*

2.4.4.2 *Effects of exceptional loads or localised damage*

Replace 'steel' by 'reinforcement'.

Add new clause:

'2.4.4.3 *Partial safety factors for short-term early age loading*

Text	Comments
The partial safety factors in 2.4.4.1 are to cover uncertainties in both the short- and long-term properties. For situations in which the reinforcement is only subjected to short-term loads early in the life of the structure, such as early thermal effects or handling stresses, the factor applied to the strength may be reduced to 1.25 for all materials and the factor applied to the elastic modulus taken as 1.1 for all materials'.	*It should be emphasised that this reduction in partial safety factors only applies to loadings such as those due to early thermal stresses or those due to handling and transport of precast units. It does not apply to elements subjected only to short-term loads which occur at some considerable time after casting; in this situation the higher safety factors must be used.*

2.4.5 Design loads for serviceability limit state

No change.

2.4.6 Material properties for serviceability limit state

No change.

2.4.7 Material properties for durability

Delete sub-clauses (a)(1) and (b)(1).

Add new final paragraph:

Text	Comments
'Unlike steel reinforcement, durability of FRP reinforcement in a particular environment depends on the selection of the appropriate materials and not on the properties of the surrounding concrete. The FRP reinforcement material used should be adequately durable for the particular application'.	*The durability of the reinforcement should be ensured by the manufacturer on the basis of adequate tests carried out in accordance with agreed test methods. Appropriate materials partial safety factors, such as those given in Tables 2.2a and 2.2b, should take account of the change in properties with time. Further*

Text | **Comments**

information can be found in D.2.2.

2.5 ANALYSIS

2.5.1 General

No change.

2.5.2 Analysis of structure

Delete second paragraph and replace by:

'Under design ultimate loads, envelopes of forces and moments should be determined by linear elastic analysis of the structure. No redistribution of forces may be taken into account in the analysis. Where appropriate, the effects of buckling should be considered using the methods described in section three'.

Non-metallic reinforcement does not have any significant yield at ultimate load and hence there will be only very limited plastic rotation at any given section. Thus it is unlikely that there will be any significant redistribution of moments. This is an area that has only been briefly studied experimentally. Further information can be found in D.2.7.

2.5.3 Analysis of sections for the ultimate limit state

Amend first sentence to read ' ... from the design properties of the materials as given in 2.4.4.1 or 2.4.4.3 and Figures 2.1 and 2.2a as appropriate'.

Delete 'In the case ... and BS 5896'.

2.5.4 Analysis of sections for serviceability limit state

In first paragraph, replace 'steel' by 'reinforcement'.

Replace first sentence of final paragraph by:

'The value for the elastic modulus of the reinforcement should be as supplied by the manufacturer'.

The elastic modulus will depend not only on the type of fibre used in the composite but also on the amount of fibre present in the cross-section. Hence the value used in design must be obtained from the manufacturer, or determined by some other means.

2.6 DESIGNS BASED ON TESTS

No change.

The use of prototype testing to develop new structural elements is particularly appropriate when using FRP reinforcement as the tests can overcome some of the uncertainties that exist in certain aspects of the design, as

Text | **Comments**

indicated elsewhere in this document. Physical testing should always be carried out in parallel with analytical studies so that the role of the reinforcement in the behaviour at service loads and at failure can be understood. This will enable appropriate safety factors to be applied to the observed behaviour, taking into account the likely changes in the properties of the reinforcement with time.

SECTION 3. DESIGN AND DETAILING: REINFORCED CONCRETE

3.1 DESIGN BASIS AND STRENGTH OF MATERIALS

3.1.1 General

No change.

3.1.2 Basis of design of reinforced concrete

No change.

3.1.3 Alternative methods (serviceability limit state)

No change.

3.1.4 Robustness

No change.

3.1.5 Durability of structural concrete

3.1.5.1 *General*

Redraft first and second paragraphs to read:

'A durable concrete element is one that is designed and constructed to perform satisfactorily in the working environment for the lifetime of the structure.

To achieve this it is necessary to consider many interrelated factors at the various stages in the design and construction process. Thus the structural form and the cover to the reinforcement are considered at the design

Text	Comments
stage and this involves consideration of the requirements for fire resistance (see **3.3.6**)'.	
Delete third paragraph. In the list of factors influencing durability delete (b), (c) and (f).	
3.1.5.2 *Design for durability*	
3.1.5.2.1 *Design and detailing of the structure*	
Delete third paragraph.	
3.1.5.2.2 *Depth of concrete cover and concrete quality*	
Delete.	
3.1.5.2.3 *Other properties*	
Delete ', or its interaction with steel reinforcement,'	
3.1.5.2.4 *Unreinforced concrete*	
No change.	
3.1.6 Loads	
No change.	
3.1.7 Strength of materials	
3.1.7.1 *General*	
No change.	
3.1.7.2 *Durability*	
No change.	
3.1.7.3 *Age allowance for concrete*	
No change.	
3.1.7.4 *Characteristic strengths of reinforcement*	
Delete all text and Table 3.1 and replace by 'The characteristic strength of the reinforcement should be supplied by the	*The strength will depend not only on the type of fibre used in the composite but also on the amount of fibre present in the cross-section. Hence the value used in design must be*

Suggested modifications to BS 8110

Text	Comments
manufacturer'.	*obtained from the manufacturer, or determined by some other means.*
3.2 STRUCTURES AND STRUCTURAL FRAMES	
3.2.1.2.1 *Simplification into sub-frames*	
Delete '(but see ... moments)' and replace by 'without redistribution of moments'.	*No redistribution should be used; see comments on 2.5.2.*
3.2.2 Redistribution of moments	
Delete whole clause and replace by 'No redistribution of the moments obtained by means of an elastic analysis may be carried out'.	*No redistribution should be used; see comments on 2.5.2.*
3.3 CONCRETE COVER TO REINFORCEMENT	
3.3.1 Nominal cover	
3.3.1.1 ***General***	
Delete 'steel'.	
Delete sub-clause (b).	*The durability of fibre composite material is not a function of the cover provided, see 2.4.7.*
In sub-clause (c) replace 'steel' by 'reinforcement'.	
Add new final sentence 'For structures for which fire is not a significant design consideration, only (a) and (d) will need to be taken into account'.	
3.3.1.2 ***Bar size***	
Replace 'steel' by 'reinforcement'.	
3.3.1.3 ***Nominal maximum size of aggregate***	
No change.	
3.3.1.4 ***Concrete cast against uneven surfaces***	
Delete 'beyond the values given in Table 3.3'.	

Suggested modifications to BS 8110

Text | **Comments**

3.3.3 Cover against corrosion

Delete whole clause and delete Table 3.3.

3.3.4 Exposure conditions

Delete 'listed in Table 3.3'.

3.3.5 Method of specifying concrete for durability

3.3.5.1 *Mix proportions*

Delete all text and replace by 'The mix proportions should be appropriate to the concrete grade, determined on the basis of the required strength'.

3.3.5.2 *Permitted reduction in cement content*

Delete whole clause.

3.3.5.3 *Permitted reduction in concrete grades*

Delete whole clause.

3.3.5.4 *Adjustment to cement contents for different sized aggregates*

Delete whole clause and delete Table 3.3.

3.3.6 Cover as fire protection

Delete whole clause and delete Table 3.4 and Figure 3.2. Replace text by:

'Generally, FRP reinforcement should not be used in structures for which fire is a significant design consideration, unless the design is based on the results of fire tests. Where fire test data are not available, the cover necessary to provide the required fire protection should be determined on the basis of an analysis of the heating within the member, as outlined in Method 3 in section four of BS 8110: Part 2: 1985, with the suggested modifications.

The principles of the design of structures

There are two stages in the degradation of the FRP reinforcement when a concrete member is subjected to fire. The first is when the temperature at a significant area of the surface of the bar reaches the 'glass transition temperature'. At this point the resin matrix will soften and hence the bond with the concrete in this region will be lost. This will limit the stress in the reinforcement that may be used for design purposes. Provided the ends of the bar are in cool regions, and hence have not lost their bond capacity, the member will behave in a manner similar to one with

Text | **Comments**

subjected to fire are that the reinforcement should be continuous, i.e. without any laps or other forms of connection, from one end of the member to the other. Steps should be taken to ensure that the ends of the member remain below the safe working temperature of the resin matrix during a fire. The cover to the reinforcement must then be sufficient to maintain the surface of the bar below the temperature at which the fibres start to degrade significantly. This data should be provided by the manufacturer.

Where the ends of the bars are not kept cool, the temperature at the surface of the bar should be limited to the glass transition temperature. This data should be provided by the manufacturer'.

unbonded prestressing. Thus it will not collapse, though deflections and crack widths will obviously be increased. The limiting condition will be when a significant portion of the cross-section of the bar itself is heated to the level at which the fibres themselves lose significant strength, again limiting the stress that may be used in design.

If the ends are not anchored, the element becomes effectively unreinforced and will fail once the glass transition temperature is reached at the surface of the bar.

Design for fire is an area that requires further development, backed up by experimental work. Further information can be found in D.2.5

3.3.7 Control of cover

No change.

3.4 BEAMS

3.4.1 General

No change.

3.4.2 Continuous beams

No change.

3.4.3 Uniformly-loaded continuous beams with approximately equal spans: moments and shears

No change.

3.4.4 Design resistance of beams

3.4.4.1 *Analysis of sections*

Modify sub-clause (d) to read:

'(d) The stresses in the reinforcement in tension are derived from the stress-strain curve in Figure 2.2a with the appropriate γ_{me} from Table 2.2b. The stress should not exceed f_r/γ_m, with γ_m being obtained from Table 2.2a. The stress in any reinforcement in

It is important to note that two different partial safety factors are required, one on the elastic modulus to determine the tensile stress and the other to check that the design strength of the material is not exceeded.

Fibre composite materials have a lower

Text	Comments
compression should be ignored'.	*strength in compression than in tension. It has therefore been decided to ignore the contribution of bars in compression, both for beams in bending and also for columns. This approach, which is in line with other proposed design codes for fibre composite reinforcement, is conservative.* *Apart from this, the approach for the design of sections is unaltered. However, it will be necessary to calculate the resistance moment from first principles rather than from the use of charts or simplified equations, until such time as design aids have been developed that are appropriate to the material being used.* *Because of the relatively low stiffness of fibre composites, and hence the high strains in the material, it is likely that failure may well be by crushing of the concrete rather than rupture of the reinforcement.* *Further information can be found in D.3.1.* *The effect of stress reversals of a significant magnitude on the tensile strength of FRP bars is not well understood. However, this is only likely to be of importance for seismic loading, which is outside the scope of this document.*
3.4.4.2 *Design charts*	
Delete whole clause.	*Being for steel reinforcement, which has an elastic-plastic response, the charts are not applicable to fibre-composites.*
3.4.4.3 *Symbols*	
Delete β_b and its definition.	
3.4.4.4 *Design formulae for rectangular beams*	
Delete whole clause.	*These formulae are based on the assumption that the reinforcement behaves plastically at ultimate load and hence are not appropriate to fibre composites.*

Suggested modifications to BS 8110

Text

3.4.4.5 ***Design formulae for flanged beams where the neutral axis falls below the flange***

Delete whole clause and delete Table 3.6.

3.4.5 Design shear resistance of beams

3.4.5.1 ***Symbols***

Replace 'A_{sb}' by 'A_{rb}' and replace 'A_{sv}' by 'A_{rv}'.

Add 'E_r elastic modulus of tensile reinforcement, not to be taken as more than 200,000N/mm^2

E_{rv} elastic modulus of shear reinforcement, not to be taken as more than 200,000N/mm^2.'

Replace 'f_{yv}' by 'f_{rv}'.

Delete '(not to be taken as more than 460N/mm^2)'.

Replace '(see Table 3.9)' by '(see equation 3a)'.

3.4.5.2 ***Shear stress in beams***

No change.

3.4.5.3 ***Shear reinforcement: form, area and stress***

Replace final sentence by 'Stress in any bar should not exceed f_{rv} divided by the appropriate γ_m from Table 2.2a.'

Amend Table 3.7 as follows to form Table 3.7a:

Replace '0.95 f_{yv}' by '0.0025 E_{rv}/γ_{me}' twice.

Replace 'steel' by 'reinforcement'.

Add footnote as follows:

'NOTE 4. 0.0025 E_{rv}/γ_{me} should not be greater than f_{rv}/γ_m'.

Comments

These formulae are based on the assumption that the reinforcement behaves plastically at ultimate load and hence are not appropriate to fibre composites.

The current approach for shear design adds the contribution from the shear reinforcement to that of the concrete cross-section. This assumption appears to give an adequate margin of safety. However, the assumption will only remain valid when using FRP shear reinforcement if any shear crack that forms is adequately controlled. Hence the strain in the shear reinforcement has to be limited. A maximum strain of 0.0025 has been selected; this is comparable to the limiting strain implied by BS 8110 which says that the characteristic strength of steel shear reinforcement should not be taken as greater than 460N/mm^2, a strain of 0.0023. The same philosophy is used in the Draft Canadian Bridge Code which limits

Suggested modifications to BS 8110

Text	Comments
	the strain in FRP shear reinforcement to 0.0020. *The capacity of the shear reinforcement is determined by multiplying the strain by the elastic modulus of the material. The partial safety factor used is thus that appropriate to the modulus and not the strength of the material.* *Further information can be found in D.3.2.*
3.4.5.4 ***Concrete shear stress*** Replace first paragraph by:	
'Values for the design concrete shear stress, v_c, should be derived from equation 3a as follows: $v_c = 0.27(100A_{reff}/(b_vd))^{0.33}(400/d)^{0.25}f_{cu}^{0.33}/1.25$ ... equation 3a where $A_{reff} = A_rE_r/200{,}000\gamma_{me}$ E_r = short-term elastic modulus of the reinforcement in N/mm^2, not to be taken as greater than 200,000N/mm^2 γ_{me} = partial safety factor for elastic modulus from Table 2.2b $100A_{reff}/b_vd$ should not be taken as greater than 3 $400/d$ should not be taken as less than 1'. In paragraph two replace 'A_s' by 'A_r' and replace 'Table' by 'equation'. In paragraph three replace 'steel' by 'reinforcement' four times. Delete Table 3.8.	*It has been assumed that the shear resistance of the concrete is a function of the effective area of tension reinforcement, determined on the basis of the elastic modulus of the fibre composite in comparison to that of steel. This assumption, which is in line with that in other draft suggestions, has been justified by comparing predicted and measured shear strengths of a large number of published tests. This empirical approach takes into account any changes in the behaviour that may be due to the lower transverse (dowel) strength of the material.* *As the shear strength is a function of the elastic modulus of the FRP it is necessary to apply a partial safety factor to the short-term value.* *Further information can be found in D.3.2.*
3.4.5.6 ***Shear resistance of bent-up bars***	
In equation 4 replace 'A_{sb}' by 'A_{rb}' and replace '$0.95f_{yv}$' by '$0.0025E_{rv}/\gamma_{me}$'. In last sentence of clause replace 'steel' by 'reinforcement'.	*The same approach as for vertical shear reinforcement has been used when modifying this clause.*

Suggested modifications to BS 8110

Text	Comments

3.4.5.7 *Anchorage and bearing of bent-up bars*

No change.

3.4.5.8 *Enhanced shear strength of sections close to supports*

No change.

3.4.5.9 *Shear reinforcement for sections close to supports*

Replace '$0.95f_{yv}$' by '$0.0025E_{rv}/\gamma_{me}$' twice in equation 5.

The same approach as for vertical shear reinforcement has been used when modifying this clause.

3.4.5.10 *Enhanced shear strength near supports (simplified method)*

Replace 'Table 3.7.' by 'Table 3.7a.'

3.4.5.11 *Bottom loaded beams*

No change

3.4.5.12 *Shear and axial load*

Replace 'from Table 3.8.' by 'from **3.4.5.4**.'

3.4.5.13 *Torsion*

No change.

3.4.6 Deflection of beams

3.4.6.1 *General*

Delete 'but in all normal cases ... and 3.11.'

Delete Tables 3.9, 3.10 and 3.11.

Where deflections are a design consideration, they should be calculated in accordance with the recommendations of Part 2 of BS 8110, with the suggested modifications. It should be noted that the deflections for FRP reinforced members are likely to be greater than those for equivalent steel reinforced members. However, comparisons with published data suggest that the calculation method for steel reinforced members may equally well be used for FRP reinforced ones.

Simple design methods using span/effective depth ratios for beams with FRP reinforcement

Text	Comments
	have yet to be developed. Further information can be found in D.3.7.

3.4.6.2 ***Symbols***

Delete whole clause.

3.4.6.3 ***Span/effective depth ratio for a rectangular or flanged beam***

Delete whole clause.

3.4.6.4 ***Long spans***

Delete whole clause.

3.4.6.5 ***Modification of span/depth ratios for tension reinforcement***

Delete whole clause.

3.4.6.6 ***Modification of span/depth ratios for compression reinforcement***

Delete whole clause.

3.4.6.7 ***Deflection due to creep and shrinkage***

Delete whole clause.

3.4.7 Crack control in beams

No change

3.5 SOLID SLABS SUPPORTED BY BEAMS OR WALLS

3.5.1 Design

No change.

The approach adopted in these clauses assumes a limited amount of redistribution. Although it has been suggested earlier that it is inappropriate to carry out any redistribution, it has been assumed that the redundancy in slab design justifies the limited amount used here. This is an area that requires further work.

3.5.2 Moments and forces

No change.

The approach used for the design of shear reinforcement in slabs is the same as for beams, see comments in 3.4.5. Further

Suggested modifications to BS 8110

Text | **Comments**

information can be found in D.3.4.

3.5.3 Solid slabs spanning in two directions at right angles: uniformly distributed loads

3.5.3.6 *Restrained slab with unequal conditions at adjacent panels*

Replace 'steel' by 'reinforcement' twice.

3.5.4 Resistance moment of solid slabs

No change.

3.5.5 Shear resistance of solid slabs

3.5.5.1 *Symbols*

Replace 'A_{sv}' by 'A_{rv}' and replace 'A_{sb}' by 'A_{rb}'.
Add 'E_{rv} short-term elastic modulus of shear reinforcement in N/mm^2, not to be taken as more than 200,000N/mm^2'.

Replace 'f_{yv}' by 'f_{rv}'.

Delete 'which should not be taken as greater than 460N/mm^2.'

Replace 'obtained from Table 3.9.' by 'derived from equation 3a'.

3.5.5.3 *Shear reinforcement*

In Table 3.16 replace '$0.95f_{yv}$' by '$0.0025E_{rv}/\gamma_{me}$' three times, replace 'A_{sv}' by 'A_{rv}' and replace 'A_{sb}' by 'A_{rb}'.

Add to Table 3.16 'NOTE 3. $0.0025E_{rv}/\gamma_{me}$ should not be greater than f_{rv}/γ_m'.

3.5.6 Shear in solid slabs under concentrated loads

No change.

3.5.7 Deflection

Delete whole clause.

Where deflections are a design consideration, they should be calculated in accordance with the recommendations of Part 2 of BS 8110, with the suggested modifications. It should be

Suggested modifications to BS 8110

Text	Comments
	noted that the deflections for FRP reinforced members are likely to be greater than those for equivalent steel reinforced members. As for beams, simple 'deemed to satisfy' rules need to be developed for slabs.
3.6 RIBBED SLABS (WITH SOLID OR HOLLOW BLOCKS OR VOIDS)	
3.6.4.7 ***Area of shear reinforcement in ribbed hollow block or voided slabs***	
Replace 'obtained from Table 3.8.' by 'derived from equation 3a'.	
3.6.5.2 ***Rib width of voided slabs or slabs of box or I-section units***	
Delete clause.	
3.6.6.2 ***Reinforcement in topping for ribbed or hollow block slabs***	
Replace 'wires' by 'the elements of the mesh'.	
3.6.6.3 ***Links in ribs***	
Delete final paragraph.	
3.7 FLAT SLABS	
3.7.1.1 ***Symbols***	
Add 'E_{rv} short-term elastic modulus of shear reinforcement in N/mm^2, not to be taken as more than 200,000N/mm^2'.	
Delete 'f_{yv}'.	
3.7.7.4 ***Shear capacity of a failure zone without shear reinforcement***	
Replace 'obtained from Table 3.8.' by 'derived from equation 3a.'	
3.7.7.5 ***Provision of shear reinforcement***	
In Equations 29a and 29b replace '$0.95f_{yv}$' by '$0.0025E_{rv}/\gamma_{me}$'.	*The approach used for the design of shear reinforcement in slabs is the same as for beams, see comments in 3.4.5. Further information can be found in D.3.4.*

Suggested modifications to BS 8110

Text | **Comments**

Add 'E_{rv} is the elastic modulus of the shear reinforcement, in N/mm^2, but not greater than 200,000N/mm^2'.

Delete 'f_{yv} is the characteristic ... in N/mm^2)'.

3.8 COLUMNS

3.8.1 General

No change.

3.8.2 Moments and forces in columns

No change.

3.8.3 Deflection induced moments in solid slender columns

3.8.3.1 *Design*

Below Equation 33 delete '+ $0.95 f_y A_{sc}$'.

3.8.4 Design of column section for ULS

3.8.4.1 *Analysis of sections*

Add to end of paragraph 'The strength of any FRP reinforcing bars in compression should be ignored'.

As indicated earlier, fibre composite materials have a lower strength in compression than in tension. It has therefore been decided to ignore the contribution of bars in compression, both for columns and also for beams in bending. This approach, which is in line with other proposed design codes for fibre composite reinforcement, is conservative. Further information can be found in D.3.5.

3.8.4.2 *Design charts for symmetrically-reinforced columns*

Delete whole clause.

3.8.4.3 *Nominal eccentricity of short columns resisting moments and axial loads*

In Equation 38 delete '+ $0.8 A_{sc} f_y$'.

Suggested modifications to BS 8110

Text	Comments

3.8.4.4 ***Short braced columns supporting an approximately symmetrical arrangement of beams***

In Equation 39 delete '+ $0.7A_{sc}f_y$'.

3.8.4.5 ***Biaxial bending***

No change.

3.8.4.6 ***Shear in columns***

No change

3.9 WALLS

3.9.1 Symbols

Delete 'f_y characteristic strength of compression reinforcement'.

3.9.2 Structural stability

No change.

3.9.3 Design of reinforced walls

3.9.3.1 ***Axial forces***

No change.

3.9.3.2 ***Effective height***

No change.

3.9.3.3 ***Design transverse moments***

No change.

3.9.3.4 ***In-plane moments***

No change.

3.9.3.5 ***Arrangement of reinforcement for reinforced walls in tension***

No change.

Suggested modifications to BS 8110

Text / Comments

3.9.3.6 *Stocky reinforced walls*

3.9.3.6.1 *Stocky braced reinforced walls supporting approximately symmetrical arrangements of slabs*

In Equation 42 delete '+ $0.7A_{sc}f_y$'.

3.9.3.6.2 *Walls resisting transverse moments and uniformly distributed axial forces*

No change.

3.9.3.6.3 *Walls resisting in-plane moments and axial forces*

No change.

3.9.3.6.4 *Walls with axial forces and significant transverse and in-plane moments*

No change.

3.9.3.7 *Slender reinforced walls*

No change.

3.9.3.8 *Deflection*

No change.

3.9.4 Design of plain walls

No change apart from to **3.9.4.19** as indicated below.

3.9.4.19 *Cracking of concrete*

Replace 'Wherever provided ... 0.30% of the concrete cross-sectional area.' by 'Wherever provided, the quantity of reinforcement should be, in each direction, at least 0.30% of the concrete cross-sectional area'.

The ability of reinforcement to control cracking due to thermal movements is a function of both its strength and also its bond characteristics. The effectiveness of FRP reinforcement has not been studied experimentally. Hence the area appropriate to grade 250 steel (smooth) has been chosen as being a conservative approach.

Suggested modifications to BS 8110

Text	Comments

3.10 STAIRCASES

3.10.1 General

No change.

3.10.2 Design of staircases

3.10.2.1 ***Strength, deflection and crack control***

No change.

3.10.2.2 ***Permissible span/effective depth ratio for staircases without stringer beams***

Delete whole clause.

3.11 BASES

3.11.1 Symbols

Replace 'see Table 3.8.' by 'derived from equation 3a'.

3.11.2 Assumptions in the design of pad footings and pile caps

No change.

3.11.3 Design of pad footings

No change.

3.11.4 Design of pile caps

No change.

3.12 CONSIDERATIONS AFFECTING DESIGN DETAILS

3.12.1 Permissible deviations

No change.

3.12.2 Joints

No change.

Suggested modifications to BS 8110

Text	Comments

3.12.3 Ties

3.12.3.2 *Proportioning of ties*

Replace ' ... acting at its characteristic strength ...' by ' ... acting at its design strength ...'

The introduction of a partial safety factor will take into account the long-term changes in the properties of the material.

3.12.4 Reinforcement

No change.

3.12.5 Minimum areas of reinforcement in members

3.12.5.1 *General*

No change.

3.12.5.2 *Symbols*

Replace 'f_y' by 'f_r', 'A_s' by 'A_r' and 'A_{st}' by 'A_{rt}'.

Replace 'steel' by 'reinforcement'.

Delete 'A_{sc} area of steel in compression.'

3.12.5.3 *Minimum percentages of reinforcement*

Amend Table 3.25 to form Table 3.25a as follows:

Replace 'A_{st}' by 'A_{rt}'. Delete column headed f_y = 250N/mm^2 completely. Also delete heading f_y = 460N/mm^2 from right-hand column. (Thus right-hand column percentages apply to all FRP reinforcement.) Finally, delete all reference to compression reinforcement.

The principle is that the minimum area of reinforcement provided should be capable of carrying the force carried by the concrete in tension prior to cracking. Hence the amount required is a function of the strength of the reinforcement. To limit the strains in the FRP reinforcement, and hence the widths of any cracks that form, the minimum percentages have been determined on the basis of limiting the strain to 0.0025, as for shear reinforcement. For convenience, the percentages for 460N/mm^2 have been retained though these are strictly equivalent to a strain of 0.0023. Further information can be found in D.3.8.

3.12.5.4 *Minimum size of bars in side faces of beams to control cracking*

Replace '$s_b b/f_y$' by '$s_b b \gamma_m / f_r$'.

The partial safety factor has been added to take account of long-term effects; this is an area that requires further experimental study.

Suggested modifications to BS 8110

Text	Comments

3.12.6 Maximum areas of reinforcement in members

No change.

The maximum area is controlled by the practical requirements for casting and compacting the concrete and is not a function of the type of reinforcement used.

3.12.7 Containment of compression reinforcement

No change.

Although bars in compression are not taken into account in the design process they will still carry some load and hence lateral reinforcement is required to control any tendency to buckle. For want of experimental evidence, the rules applicable to 460 N/mm^2 have been retained in the following sections, as elsewhere in these suggested changes.

This is a topic that requires further study.

3.12.7.1 *Links for containment of beam or column compression reinforcement*

No change.

3.12.7.2 *Arrangement of links for containment of beam or column compression reinforcement*

No change.

3.12.7.3 *Containment of compression reinforcement around periphery of circular column*

No change.

3.12.7.4 *Diameter of horizontal bars for support of small amounts of compression reinforcement in walls*

In first paragraph replace 'at least the following percentages ... 0.25% of concrete area' by 'the percentage of horizontal reinforcement should be at least 0.25% of the concrete area'.

Suggested modifications to BS 8110

Text	Comments
3.12.7.5 ***Links for containment of large amounts of compression reinforcement in walls***	
No change.	
3.12.8 Bond, anchorage, bearing, laps, joints and bends in bars	
3.12.8.1 ***Avoidance of bond failure due to ultimate loads***	
No change.	
3.12.8.2 ***Anchorage bond stress***	
No change.	
3.12.8.3 ***Design anchorage bond stress***	
No change.	
3.12.8.4 ***Values for design ultimate anchorage bond stress***	
Delete whole clause and Table 3.26 and replace by:	*The values in Table 2.2a take account of changes in the bond characteristics with time.*
'Values for the characteristic ultimate anchorage bond stress should be determined by the supplier of the material in accordance with approved test methods. Design ultimate values should be obtained by dividing the characteristic values by a partial safety factor, γ_m of 1.4 or by the appropriate value in Table 2.2a, which ever is the greater.'	*It is not clear whether the bond stress is a function of the concrete strength or not; experimental evidence is contradictory. Further information can be found in D.2.4.*
3.12.8.5 ***Design ultimate anchorage bond stress for fabric***	
Delete whole clause and clause heading.	*The values in Table 2.2a take account of changes in the bond characteristics with time.*
Replace by:	
'3.12.8.5 Design ultimate anchorage bond stresses for FRP grid material	
The value for the design ultimate anchorage bond stress should be in accordance with the manufacturer's recommendations, divided by a partial safety factor, γ_m of 1.4 or by the appropriate value in Table 2.2a, which ever is	

Text | Comments

the greater.

Alternative methods of anchoring fabric, such as overlapping by a specified amount should be in accordance with the manufacturer's recommendations.'

3.12.8.6 ***Anchorage of links***

Delete final paragraph.

3.12.8.7 ***Anchorage of welded fabric used as links***

Delete whole clause and clause heading.

3.12.8.8 ***Anchorage of column starter bars in bases or pile caps***

No change.

3.12.8.9 ***Laps and joints***

Replace 'welded' by 'adhesively bonded'.

Delete footnote.

Add 'The strength and durability of any adhesively bonded or mechanical connection should be demonstrated by the manufacturer. All such connections should be made in accordance with the manufacturer's recommendations.'

3.12.8.10 ***Joints where imposed loading is predominantly cyclical***

Delete whole clause and replace by 'The adequacy of connections subjected to cyclic loadings should be demonstrated by the manufacturer to the satisfaction of the designer.'

3.12.8.11 ***Minimum laps***

No change.

3.12.8.12 ***Laps in beams and columns with limited cover***

No change.

Suspect modifications to BS 8110

Text	Comments

3.12.8.13 ***Design of tension laps***

Delete final paragraph and delete Table 3.27.

3.12.8.14 ***Maximum amount of reinforcement in a layer including tension laps***

No change.

3.12.8.15 ***Design of compression laps***

Delete whole clause.

3.12.8.16 ***Butt joints***

3.12.8.16.1 *Bars in compression*

Delete whole clause.

3.12.8.16.2 *Bars in tension*

Delete whole clause and replace by:

'The only acceptable form of butt joint for a bar in tension comprises an adhesively bonded or mechanical coupler. The tensile capacity of the assembly should be demonstrated by the manufacturer, to the satisfaction of the designer, taking into account any effects due to long-term loading'.

3.12.8.17 ***Welded joints in bars***

Delete whole clause.

3.12.8.18 ***Strength of welds***

Delete whole clause.

3.12.8.19 ***Design strength of filler materials in lap-joint welds***

Delete whole clause.

3.12.8.20 ***Design of welded lap joints***

Delete whole clause.

Suggested modifications to BS 8110

Text	Comments
3.12.8.21 ***Limitation of length of weld in laps***	
Delete whole clause.	
3.8.12.22 ***Hooks and bends***	
Delete 'and should comply with BS 4466.'	
3.8.12.23 ***Effective anchorage length of a hook or bend***	
No change.	*The anchorage provided by hooks and bends is an area that requires experimental investigation.*
3.12.8.24 ***Minimum radius of bends***	
Delete 'than twice the ... the bar, nor less'.	*FRP bends can be fabricated to tight radii by the manufacturer, but there will still be a requirement to check that the permissible bearing stresses in the concrete are not exceeded.*
3.12.8.25 ***Design bearing stress inside bends***	
No change.	
3.12.9 Curtailment and anchorage of bars	
3.12.9.1 ***General***	
Replace '(0.87f_y)' by 'f_r/γ_m'.	
3.12.10 Curtailment of reinforcement	
Replace 'steel' by 'reinforcement' in title and throughout the text.	
3.12.11 Spacing of reinforcement	
3.12.11.2.3 *Clear horizontal distance between bars in tension*	
Delete 'the value given in Table 3.28 ... may be adjusted.' and replace by '160mm.' Delete Table 3.28.	*The clear distance is a function of the strain in the reinforcement under service conditions. As elsewhere in these suggested changes, such as the behaviour of shear reinforcement, the strain has been approximately limited to that of 460N/mm² steel bars. Hence the spacing used in this clause is that from Table 3.28, with 0%*

Text	Comments
	redistribution, appropriate to 460N/mm^2.
3.12.11.2.4 *Clear distance between bars in tension*	
Delete whole clause.	
3.12.11.2.5 *Clear distance between the corner of beam and nearest longitudinal bar in tension*	
Delete 'half the clear distance given in Table 3.28.' and replace by '80mm.'	*See comments to* ***3.12.11.2.3.***
3.12.11.2.7 *Slabs*	
Delete all of second paragraph i.e. 'In addition ... for lesser amounts.' and replace by 'In addition, unless crack widths are checked by direct calculation, bar spacings should be limited to 160mm for slabs where the reinforcement percentage exceeds 1 % or 160mm divided by the reinforcement percentage for lesser amounts.'	*See comments to* ***3.12.11.2.3.***
3.12.11.2.8 *Slabs where amount of redistribution is unknown*	
Delete whole clause.	
3.12.11.2.9 *Spacing of shrinkage reinforcement*	
No change.	

SECTION 4. DESIGN AND DETAILING: PRESTRESSED CONCRETE

The use of FRP tendons for prestressing is not covered by this document.

SECTION 5. DESIGN AND DETAILING: PRECAST AND COMPOSITE CONSTRUCTION

5.1 DESIGN BASIS AND STABILITY PROVISIONS

No change.

Suspected modifications to BS 8110

Text	Comments

5.2 PRECAST CONCRETE CONSTRUCTION

5.2.7.2.2 *Reinforcement anchorage*

Amend sub-clause (a) to read '(a) connecting, by means of a mechanical or adhesively bonded coupler, to a transverse bar of equal strength; ...'

5.2.8.3 ***Position of tension reinforcement***

Replace 'welding' by 'connecting, by means of a mechanical or adhesively bonded coupler,'

5.2.8.4 ***Design shear resistance***

Replace 'given in Table 3.8.' by 'obtained from equation 3a.'

5.3 STRUCTURAL CONNECTIONS BETWEEN PRECAST UNITS

5.3.4 Continuity of reinforcement

5.3.4.5 ***Sleeving***

Replace sub-clause (a) by '(a) resin-filled sleeves or other forms of coupler capable of transmitting both tensile and compressive forces;'

5.3.4.6 ***Threading of reinforcement***

Delete sub-clause (b).

Add a final paragraph as follows:

'The load capacity of any connector should be demonstrated by the supplier, to the satisfaction of the designer, taking into account any long-term effects'.

5.3.4.8 ***Welding of bars***

Delete whole clause.

5.3.7 Joints transmitting shear

In sub-clause (c) delete 'steel'.

Text	Comments
In sub-clause (d) replace '$0.95f_y$' by '$0.0025E_r/\gamma_{me}$'.	
Add 'E_r is the elastic modulus of the reinforcement; $0.0025E_r/\gamma_{me}$ should not exceed f_r/γ_m'.	
5.4 COMPOSITE CONCRETE CONSTRUCTION	
5.4.7.3 *Nominal links*	
Replace '0.15%' by '$0.15(200/E_r)$%'	*The minimum amount of reinforcement passing through the interface that will control the behaviour will be a function of the modulus of elasticity. Further information can be found in D.4.2.*
5.4.7.4 *Links in excess of minimum*	
Replace 'steel' by 'reinforcement'.	*Further information can be found in D.4.2.*
In Equation 62 replace '$0.95f_y$' by '$0.0025E_r/\gamma_{me}$'.	
Add '$0.0025E_r/\gamma_{me}$ should not exceed f_r/γ_m'.	

SECTION 6. CONCRETE MATERIALS, SPECIFICATION AND CONSTRUCTION

Text	Comments
6.1 MATERIALS AND SPECIFICATION	
No change.	*The Code makes reference to BS 5328, the standard for Concrete. No attempt has been made at this stage to make any changes to this Standard which may lead to the adoption of concrete mixes that are more suitable for use with FRP reinforcement than with conventional steel.*
6.2 CONCRETE CONSTRUCTION	
No change.	

Suggested modifications to BS 8110

Text | **Comments**

SECTION 7. SPECIFICATION AND WORKMANSHIP: REINFORCEMENT

7.1 GENERAL

Delete existing text and replace by:

'FRP reinforcement comprising resin-fibre composite rods, or similar materials, should be procured from sources approved by the designer and should comply with appropriate trade standards and with any schedule specified by the designer. The manufacturer should provide the designer with the following information on the FRP material:

- fibre type
- resin type
- thermal response of composite (e.g. glass transition temperature)
- nominal diameter or dimensions
- characteristic tensile strength
- characteristic modulus of elasticity (tensile)
- characteristic bond stress (when embedded in 40N/mm^2 concrete)

In addition the manufacturer should provide evidence that the long-term properties of the FRP material are such that the design assumptions, such as partial safety factors, used in this document are reasonable.'

Add new clause:

'7.1a HANDLING AND STORAGE
The contractor should take advice from the manufacturer regarding any particular requirements for handling or storage.

FRP reinforcement should be suitably supported and secured during transport to site to ensure that there is no damage or distortion during delivery. Care should be taken during off-loading and handling to avoid mechanical damage, bending or heating of the material. All FRP reinforcement should be stored clear of the ground and protected from the weather. The material should not be exposed to a direct heat source.

Suggested modifications to BS 8110

Text

Comments

Contact with any chemicals should be avoided, including substances commonly found on construction sites, such as mould release oils.

7.2 CUTTING AND BENDING

Delete all existing text and replace by:

'FRP reinforcement should be scheduled in accordance with specified dimensional tolerances and shapes appropriate to the material used.

FRP reinforcement should be provided manufactured to the shapes and dimensions specified in the contract. Unless permitted by the manufacturer, there will be no opportunity to correct supplied dimensions of FRP reinforcement other than by cutting and connecting by approved means, see **7.4**. Any cutting of FRP reinforcement should be in accordance with guidance from the manufacturer. Any cut ends should be immediately protected by a suitable resin coat or other approved methods, unless the manufacturer can demonstrate that such protection is unnecessary.'

The manufacturer should provide a method statement covering all aspects of handling FRP materials on site. This should include an indication of the minimum radius to which the material may be bent without causing damage. The only materials that can be bent on site to a tight radius, to form shear links for example, will be thermoplastic composites; bends with these materials may only be formed on site strictly in accordance with the manufacturer's method statement.

7.3 FIXING

Delete 'steel' in second paragraph twice. In third paragraph, replace 'lead to corrosion of' by 'damage'.

The manufacturer should provide guidance on approved methods for fixing the FRP material.

Redraft fifth paragraph to read:

'Non-structural connections for the positioning of reinforcement should be made with plastic ties, clips or other tying devices or by means of mechanical or adhesively bonded connections, see **7.6**. Care should be taken to ensure that projecting ends of ties or clips do not encroach into the cover concrete.'

Delete final sentence of clause: 'The importance of cover ... the hardened concrete.'

7.4 SURFACE CONDITION

Delete 'loose rust, loose mill scale,'.

Suggested modifications to BS 8110

Text | **Comments**

Replace 'steel' by 'reinforcement'.

Delete final sentence: 'Normal handling prior ... scale from reinforcement.'

7.5 LAPS AND JOINTS

No change.

7.6 WELDING

Delete complete clause and replace by:

'7.6 CONNECTIONS

The only acceptable form of structural (load-carrying) connection between FRP bars or other forms of reinforcement is by means of couplers, which may be either adhesively bonded or mechanically connected. The load capacity of the coupler, either in tension or compression, should be demonstrated by the manufacturer, with due allowance being made for long-term effects.

Some limited development work has been carried out on couplers, but there are currently no systems that will carry acceptable loads.

All connections should be made in accordance with the manufacturer's specification, by suitably trained staff. Particular attention should be paid to any requirements for surface preparation and for the curing of the adhesive. Where possible, all connections should be made off site, ideally by the reinforcement supplier. Where it is essential that connections are made on site, the work should only be carried out by suitably trained staff, and steps should be taken to provide a suitable environment for curing to take place.'

SECTION 8. SPECIFICATION AND WORKMANSHIP: PRESTRESSING TENDONS

The use of FRP prestressing tendons is not covered by this document.

Suggested modifications to BS 8110

Text | **Comments**

BS 8110: PART 2: 1985

CODE OF PRACTICE FOR SPECIAL CIRCUMSTANCES

SECTION ONE. GENERAL

1.1 SCOPE

No change.

Because of the limited experience of the use of FRP reinforcement, it may be appropriate to use the guidance in Part 2 more often than it would be with conventional steel reinforcement.

1.2 DEFINITIONS

No change.

Autoclaving may not be appropriate when using FRP reinforcement, see Section Six.

1.3 SYMBOLS

Replace 'f_y' by 'f_r'.

SECTION TWO. METHODS OF ANALYSIS FOR THE ULTIMATE LIMIT STATES

2.1 GENERAL

No change.

2.2 DESIGN LOADS AND STRENGTHS

No change, apart from minor alteration below.

2.2.1.3 *Element design phase*

Modify (a) to read 'the strength and stiffness of the material in the structure, taking into account any changes in properties with time;'

2.3 NON-LINEAR METHODS

No change.

2.4 TORSIONAL RESISTANCE OF BEAMS

No change, apart from the following:

Suggested modifications to BS 8110

Text	Comments

2.4.2 Symbols

Replace 'f_{yv}' by 'f_{rv}'

2.4.7 Area of torsional reinforcement

Replace '$0.87f_{yv}$' by '$0.0025E_{rv}/\gamma_{me}$'

Replace 'f_y' by '$0.0025E_r$'

Replace 'f_y and f_{yv} should not be taken as greater than 460 N/mm^2 ' by ' $0.0025E_{rv}/\gamma_{me}$ should not be taken as greater than f_{rv}/γ_m and $0.0025E_r/\gamma_{me}$ should not be taken as greater than f_r/γ_m'

There is very limited information on the behaviour of beams in torsion reinforced with FRP. Hence, the alterations to these clauses are in line with those made to the clauses dealing with the shear reinforcement for beams and slabs, see Sections 3.4.5 and 3.5.5 in Part 1. Limited further information on beams in shear can be found in D.3.3.

2.5 EFFECTIVE COLUMN HEIGHT

No change.

2.6 ROBUSTNESS

No change.

SECTION THREE. SERVICEABILITY CALCULATIONS

3.1 GENERAL

No change.

3.2 SERVICEABILITY LIMIT STATES

No change apart from that shown below.

3.2.4.2 *Corrosion*

Delete whole clause.

3.3 LOADS

No change.

3.4 ANALYSIS OF STRUCTURE FOR SERVICEABILITY LIMIT STATES

No change.

Suggested modifications to BS 8110

Text

3.5 MATERIAL PROPERTIES FOR THE CALCULATION OF CURVATURE AND STRESSES

Add new paragraph as follows:

'The modulus of elasticity of the reinforcement should be appropriate to the age of loading at which the curvatures and stresses are being calculated.'

3.6 CALCULATION OF CURVATURES

In sub-clause (a)(1) replace ' ... may be taken as 200kN/mm^2.' by ' ... should be taken as E_r/γ_{me} , where γ_{me} has the appropriate value from Table 2.2b in the suggested changes to BS 8110: Part 1: 1985.'

In sub-clause (a)(4) change 'steel' to 'reinforcement'.

Redraft sub-clause (b) to read:

'The concrete and the reinforcement are both considered to be fully elastic in tension and compression. The elastic modulus of the reinforcement should be taken as E_r/γ_{me}, where γ_{me} has the appropriate value from Table 2.2b in the suggested changes to BS 8110: Part 1: 1985, and the elastic modulus of the concrete ...'

Replace 'E_s' by 'E_r' in Equation 7 throughout sub-clause.

3.7 CALCULATION OF DEFLECTION

No change apart from in penultimate paragraph delete 'Deflections of slabs are therefore probably best dealt with by using the ratios of span to effective depth.'

3.8 CALCULATION OF CRACK WIDTH

3.8.1 General

No change.

Comments

The elastic modulus of the reinforcement will reduce with time. The short term value, which should be provided by the Manufacturer, should be divided by the partial safety factors in Table 2.2b of the suggested alterations to BS 8110: Part 1.

When calculating instantaneous deflections at early ages, the suggestions in the new 2.4.4.3 in Part 1, that the materials safety factors may be reduced, will apply.

As indicated in Part 1, span to effective depth ratios have yet to be developed for FRP reinforcement.

Further information can be found in D.3.7.

Suggested modifications to BS 8110

Text	Comments

3.8.2 Symbols

Replace 'A_s' by 'A_r' and 'E_s' by 'E_r'

3.8.3 Assessment of crack widths

At start of clause, delete 'Provided the strain in the tension reinforcement is limited to $0.8f_y/E_s$,'

Further information can be found in D.3.6.

In second paragraph replace 'steel' by 'reinforcement' twice.

Replace 'E_sA_s' by 'E_rA_r/γ_{me}' in Equation 13.

3.8.4 Early thermal cracking

No change.

SECTION FOUR. FIRE RESISTANCE

4.1 GENERAL

4.1.1 Methods

Delete sub-clause (a) Method 1. Tabulated data.

There is not yet sufficient information on the behaviour of FRP reinforced concrete elements in fire for tabulated values to be developed.

4.1.7 Protection against spalling

In sub-clause (d) replace 'steel' by 'reinforcement'.

Delete second paragraph.

4.2 FACTORS TO BE CONSIDERED IN DETERMINING FIRE RESISTANCE

No change.

4.3 TABULATED DATA (METHOD 1)

Delete whole clause and delete Tables 4.2, 4.3, 4.4 and 4.5.

There is not yet sufficient information on the behaviour of FRP reinforced concrete elements in fire for tabulated values to be developed.

Suggested modifications to BS 8110

Text	Comments

4.4 FIRE TEST (METHOD 2)

No change.

4.5 FIRE ENGINEERING CALCULATIONS (METHOD 3)

4.5.1 General

No change.

4.5.2 Principles of design

Redraft first paragraph to read:

Further information can be found in D.2.5.

'The principles employed in the calculation of the fire resistance of structural concrete elements are based on the insulating properties of concrete and on the strengths of concrete and FRP reinforcement at high temperatures.'

Delete second paragraph.

4.5.3 Application to structural elements

Redraft clause to read:

'The design approach in this method relates to structural elements in flexure, e.g. beams and floors, where failure of the element in a fire is governed by the main tensile reinforcement.

Design of compression elements, e.g. columns and walls, should be based on experience from fire tests (see method 2).'

4.5.4 Material properties for design

No change.

4.5.5 Design curve for concrete

No change.

4.5.6 Design curve for steel

Rename clause 'Design curve for reinforcement'.

The glass transition temperature will be the point at which the bond between the bar and the concrete will be effectively lost.

Delete whole clause and replace by:

Suggested modifications to BS 8110

Text

'Design curves for the reduction in the strength and stiffness of the reinforcement should be provided by the manufacturer. The manufacturer should also provide a value for the glass transition temperature for the resin matrix in the surface layer of the reinforcement.'

Delete Figure 4.5.

4.5.7 Design

Replace 'steel 1.0.' by 'reinforcement values in Table 2.2a in Part 1 of BS 8110 as modified by this document, divided by 1.15.'

Delete final sentence 'For other methods ... report [5].'

SECTION FIVE. ADDITIONAL CONSIDERATIONS IN THE USE OF LIGHTWEIGHT AGGREGATE CONCRETE

5.1 GENERAL

No change apart from replace 'A_s' by 'A_r'.

5.2 COVER FOR DURABILITY AND FIRE RESISTANCE

Delete whole clause and delete Tables 5.1 and 5.2.

5.3 CHARACTERISTIC STRENGTH OF CONCRETE

No change.

5.4 SHEAR RESISTANCE

Redraft clause to read:

'The shear resistance and shear reinforcement requirements for lightweight aggregate concrete members should be established in accordance with the suggested amendments to BS 8110: Part 1 except that the design concrete shear stress v_c should be taken as 0.8 times the values determined from 3.4.5 for beams or 3.5.5 for slabs.'

Comments

There is no reported data on the behaviour of lightweight aggregate concrete beams with FRP reinforcement. However, it has been assumed that it is reasonable to adopt the same changes as for normal weight aggregate beams. Further information can be found in D.3.2.

Text	Comments
Delete Table 5.3.	
5.5 TORSIONAL RESISTANCE OF BEAMS	
No change.	
5.6 DEFLECTIONS	
Delete 'Alternatively, for normal structures ... multiplied by 0.85.'	
5.7 COLUMNS	
No change.	
5.8 WALLS	
No change.	
5.9 ANCHORAGE BOND AND LAPS	
No change.	
5.10 BEARING STRESS INSIDE BENDS	
No change.	
SECTION SIX. AUTOCLAVED AERATED CONCRETE	
No change.	*There is no published record of FRP reinforcement being used in autoclaved aerated concrete. Care should be taken that the temperature rise is not so great that the properties of the reinforcing bars are affected.*

Suggested modifications to BS 8110

Text | **Comments**

SECTION SEVEN. ELASTIC DEFORMATION, CREEP, DRYING SHRINKAGE AND THERMAL STRAINS OF CONCRETE

No change except as follows:

7.4 DRYING SHRINKAGE

Delete paragraphs six and seven i.e. 'An estimate of the shrinkage ... made to section three of this Part.'

Because FRP reinforcement has a lower stiffness than steel, it will have a lesser effect on the shrinkage. One approach might be to work to an equivalent area of steel, based on its stiffness relative to steel, which is the approach used in shear design, and use this in the expression given in the shrinkage clause.

SECTION EIGHT. MOVEMENT JOINTS

No change.

SECTION NINE. APPRAISAL AND TESTING OF STRUCTURES AND COMPONENTS DURING CONSTRUCTION

No change.

When interpreting the results from any testing, allowance must be made for the changes in the properties of the FRP reinforcement with time, by using the appropriate safety factors given in these suggested changes.

PART C

Suggested modifications to BS 5400: Part 4: 1990 Code of Practice for design of concrete bridges

INTRODUCTION

The suggested changes to the Code clauses given in this Part were originally prepared as part of the EUROCRETE Project and represent the views of the Participants in that Project. They may therefore be taken to represent what is considered to be good advice at the time of publication, based on currently available information. BS 5400: Part 4, in common with other British Standards, states in the Foreword that 'Compliance with a British Standard does not of itself confer immunity from legal obligations'. It also states that it has been assumed that design is entrusted to chartered structural or civil engineers. Thus in drafting this document it has been assumed that it will be used by suitably qualified engineers, who will satisfy themselves that the suggested design approaches are reasonable in the relevant application. The advice given has no legal standing; the EUROCRETE Participants and The Institution of Structural Engineers accept no responsibility for the adequacy of the contents of the document nor for any omissions.

The document lists the suggested changes that should be made to the clauses in BS 5400: Part 4 when designing *in situ* or precast structures reinforced with FRP reinforcement in the form of rods, grids or other shapes. The document has been prepared on the basis of extensive work carried out in the EUROCRETE Project and on the basis of published work worldwide. Where appropriate, amended or replacement clauses are given. Where no specific change is indicated, the clause remains as given in the original Code of Practice. Where justification for the revised clauses is necessary some additional information is included, printed in italics, along with references to the appropriate background information given in Part D of this document.

The document does not cover the use of FRP prestressing tendons; for information on the design of prestressed concrete the reader should refer to relevant specialist literature.

Suggested modifications to BS 5400: Part 4

TEXT / **COMMENTS**

1 SCOPE

Add the following paragraph after the existing first paragraph:

'Reinforcement should consist of embedded FRP (Fibre Reinforced Plastic) rods, or assemblies such as grids, composed of continuous glass, carbon or aramid fibres combined with a suitable resin to form a composite. The guidance is not appropriate for concrete elements externally reinforced with FRP, in the form of strips or other shapes'.

It is important that the material used is of an adequate quality; this requires the choice of appropriate resins and fibres, combined in an appropriate manufacturing process which has the necessary quality controls. The suggested changes have been formulated on the understanding that the design will use material that has a track record of use in service that has demonstrated adequate durability, or one that has demonstrated its suitability by means of durability trials for which test data can be provided.

Unlike steel, there are currently no agreed standard properties for FRP materials which will depend on the type and quantity of fibre, the resin, additives, the processing route and other factors. Hence design must be based on the properties provided by the manufacturer. The designer must satisfy himself that the stated properties are appropriate to the particular material being used.

2 DEFINITIONS AND SYMBOLS

2.1 Definitions

2.1.1 *General*

No change.

2.1.2 *Partial load factors*

No change.

2.1.3 *Materials*

2.1.3.1 *Characteristic strength*

Redraft clause to read '...of concrete, f_{cu}, or the failure strength of reinforcement, f_r, below which ..'.

The characteristic strength of material used as shear reinforcement should be determined at the start of the bent portion.

2.1.3.2 *Characteristic stress*

No change.

2.2 Symbols

Replace subscript 'y' by 'r' throughout.

The subscript y *stands for yield. As FRP*

Suggested modifications to BS 5400: Part 4

Text	Comments
Add 'γ_{me} Partial safety factor for elastic modulus of materials'.	*reinforcement does not have any significant yield, the subscript* r, *which stands for rupture, would seem more appropriate.* *As the effective elastic modulus of a fibre composite material embedded in concrete may change with time, because of possible loss of material due to alkali attack, it is necessary to apply a partial safety factor to the short-term values. Further information can be found in D.2.2 and in Part A.*
3 LIMIT STATE PHILOSOPHY	
3.1 General	
No change.	
3.2 Serviceability limit state (SLS)	
Replace 'In particular ... **4.1.1.1**' by 'In particular crack widths should be limited so as not to impair the performance or appearance of the structure'.	*The durability of the FRP reinforcement will not be significantly affected by crack widths and hence the requirements relating crack width to the environment are no longer appropriate.*
3.3 Ultimate limit state	
No change.	
4 DESIGN: GENERAL	
4.1 Limit state requirements	
4.1.1 ***Serviceability limit states***	
4.1.1.1 *Cracking*	
In first paragraph delete 'and the conditions of exposure'.	
In definition of class 3 delete 'but with the crack widths ...Table 1'.	
Table 1 Design crack widths. Delete whole Table.	*The durability of the FRP reinforcement will not be significantly affected by crack widths and hence the requirements relating crack width to the environment are no longer appropriate.*
4.1.1.2 *Vibration*	
No change.	

Suggested modifications to BS 5400: Part 4

TEXT

4.1.1.3 *Stress limitations*

Replace 'steel' by 'reinforcement'.
Delete sub-clause (d).

Table 2 Stress limitations for the serviceability limit state

Delete 'Compression' and replace 'f_y' by 'f_r'

4.1.2 Ultimate limit states

No change.

4.1.3 Other considerations

4.1.3.1 *Deflections*

No change.

4.1.3.2 *Fatigue*

No change.

4.1.3.3 *Durability*

Delete whole clause and replace:

'The environmental conditions to which the concrete will be exposed should be defined at the design stage. The designer should be satisfied that the durability of the reinforcement is adequate, taking into account the environment to which it will be subjected. Consideration may also be given to the use of protective coatings to the concrete to enhance the durability of vulnerable parts of construction.

Unlike steel reinforcement, durability of FRP reinforcement in a particular environment depends on the selection of the appropriate materials and not on the properties of the surrounding concrete. The FRP reinforcement material used should be adequately durable

COMMENTS

It is likely that a lower level of stress would be appropriate for FRP material in tension. Material in compression is assumed to carry no load and hence that limitation has been removed.

Plastic methods of analysis and redistribution of moments are not appropriate to FRP reinforcement which has an elastic response to failure, with no yielding. Further information can be found in D.2.7.

The durability of the reinforcement should be ensured by the manufacturer on the basis of adequate tests carried out in accordance with agreed test methods. Appropriate materials partial safety factors, such as those given in Tables 3a and 3b, should take account of the change in properties with time.

It is likely that the alkaline environment of the concrete itself will be the most serious durability consideration for the reinforcement. This will be reflected in the partial safety factors applied to the short-term properties of the reinforcement. The designer should take into account any special circumstances that might affect the durability of the reinforcement and seek appropriate advice from the supplier

Suggested modifications to BS 5400: Part 4

Text

for the particular application'.

4.2 Loads, load combinations and partial factors γ_{fL} and γ_{f3}

No change.

4.3 Properties of materials

4.3.1 *General*

Replace 'f_y' by 'f_r'

4.3.2 *Material properties*

4.3.2.1 *Concrete*

No change.

4.3.2.2 *Reinforcement and prestressing steel*

Delete 'and prestressing steel' from title.

Add two new paragraphs at the beginning of the clause:

'The characteristic elastic modulus of the FRP reinforcement means that value of the short-term elastic modulus below which not more than 5% of the test results would be expected to fall.

Characteristic material properties should be determined by the manufacturer of the FRP, in accordance with agreed standard test methods'.

Replace sub-clause (a) by:

'(a) for reinforcement, a straight line response to ultimate load, as shown in Figure 2a, with the slope equivalent to the characteristic elastic modulus divided by the relevant γ_{me} from Table 3b'.

4.3.3 *Values of* γ_m

4.3.3.1 *General*

No change

Comments

of the material. Further information can be found in D.2.2.

Additional information on the properties that should be provided by the manufacturer is indicated elsewhere.

Suggested modifications to BS 5400: Part 4

TEXT | **COMMENTS**

4.3.3.2 *Serviceability limit state*

No change.

4.3.3.3 *Ultimate limit state*

Delete 'and prestressed concrete' and replace 'are' by 'is'. Delete 'and 1.15 ...tendons'.

Add second paragraph as follows:

'For the analysis of sections, the design property for a given material and limit state is derived from the characteristic property divided by γ_m or γ_{me}, where γ_m and γ_{me} are the appropriate partial safety factors for strength and modulus of elasticity given in Tables 3a and 3b respectively. γ_m and γ_{me} take account of differences between actual and laboratory values, local weaknesses, long-term effects, and inaccuracies in assessment of the resistance of sections. They also take account of the importance of the limit state being considered.

Where the response of the structure, or its resistance to loads, depends on the elastic modulus of the FRP reinforcement, the characteristic values should be divided by the appropriate γ_{me} value from Table 3b'.

Both the effective strength and the effective modulus of elasticity of FRP material embedded in concrete may change with time due to alkali attack. This contrasts with the properties of steel which are considered by BS 5400 to be constant throughout the life of the structure. Hence, it is necessary to apply a partial safety factor to both properties. The philosophy is that design will be carried out on the basis of the design properties at the end of the design life rather than at the beginning. In general, only one partial safety factor will be applied, either γ_m or γ_{me} as appropriate; the choice will depend on which factor will give a lower design strength. Further explanation is given in the notes to the relevant clauses and also in D.2.2.

Add new Tables 3a and 3b, as shown below.

Table 3a Values of γ_m for the ultimate limit state

Material or behaviour	γ_m
E-glass reinforcement	3.6
Aramid reinforcement	2.2
Carbon reinforcement	1.8
Concrete in flexure or axial load	1.5
Shear strength without shear reinforcement	1.25
Bond strength	1.4
Others (e.g. bearing stress)	≥1.5

Table 3b Values of γ_{me} for the ultimate limit state

Fibre type	γ_{me}
E-glass	1.8
Aramid	1.1
Carbon	1.1

Note: Values in Tables 3a and 3b are for guidance and should only be used in the absence of data supplied by the manufacturer.

Suggested modifications to BS 5400: Part 4

Text / Comments

Add new clause:

'4.3.3.3a *Partial safety factors for short-term loading*

The partial safety factors in 4.3.3.3 are to cover uncertainties in both the short- and long-term properties. For situations in which the reinforcement is only subjected to short-term loads early in the life of the structure, such as early thermal effects or handling stresses, the factor applied to the strength may be reduced to 1.25 for all materials and the factor applied to the elastic modulus taken as 1.1 for all materials'.

It should be emphasised that this reduction in partial safety factors only applies to loadings such as those due to early thermal stresses or those due to handling and transport of precast units. It does not apply to elements subjected only to short-term loads which occur at some considerable time after casting; in this situation the higher safety factors must be used.

Replace Figure 2 by new Figure 2a,

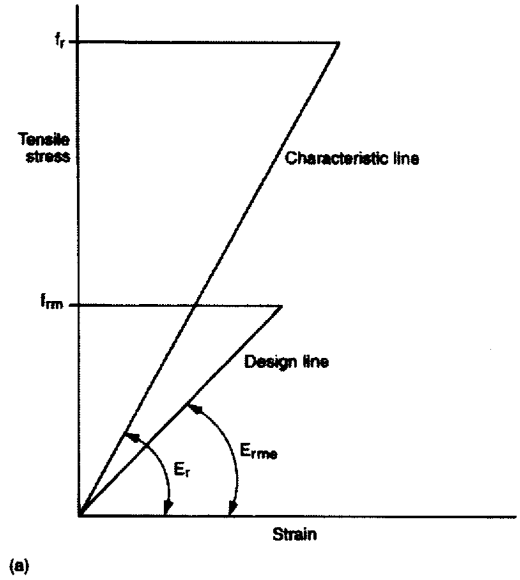

Figure 2a Stress-strain lines for FRP reinforcement

4.3.3.4 *Fatigue*

No change.

The effect of significant stress reversals on the fatigue behaviour of FRP material is not well understood. It is not recommenced for use in such circumstances.

4.4 Analysis of structure

4.4.1 General

No change.

Suggested modifications to BS 5400: Part 4

TEXT	COMMENTS

4.4.2 *Analysis for serviceability limit state*

No change.

4.4.3 *Analysis for ultimate limit state*

4.4.3.1 *General*

Replace 'Elastic methods may ..'. by 'Elastic methods should ..'.

Delete 'However, plastic methods ... bridge authority'.

4.4.3.2 *Methods of analysis and their requirements*

Delete 'alternatively, yield line ... wheel loads'.

4.5 Analysis of section

No change.

4.6 Deflection

No change.

4.7 Fatigue

Delete whole clause and replace by:

'The effect of repeated live loading on the reinforcing bars should be considered. The stress ranges in the reinforcing bars should be kept below the endurance limit for the material being used. Appropriate information on the fatigue resistance should be provided by the manufacturer.

4.8 Combined global and local effects

No change.

5 DESIGN AND DETAILING: REINFORCED CONCRETE

5.1 General

5.1.1 *Introduction*

No change.

Suggested modifications to BS 5400: Part 4

Text	Comments
5.1.2 ***Limit state design of reinforced concrete***	
5.1.2.1 *Basis of design*	
Delete third paragraph	*Plastic methods of analysis and redistribution are not appropriate when using FRP reinforcement. Further information can be found in D.2.7.*
5.1.2.2 *Durability*	
Delete whole clause.	*The durability of FRP reinforcement is not influenced by the cover.*
5.1.2.3 *Other limit states and considerations*	
No change	
5.1.3 Loads	
No change.	
5.1.4 Strength of materials	
5.1.4.1 *Definition of strengths*	
No change.	
5.1.4.2 *Characteristic strength of concrete*	
No change.	
5.1.4.3 *Characteristic strength of reinforcement*	
Delete all text and Table 6 and replace by 'The characteristic strength of the reinforcement should be supplied by the manufacturer'.	*There are currently no standard specifications for FRP bars. The strength will depend not only on the type of fibre used in the composite but also on the amount of fibre present in the cross-section. Hence the value used in design must be obtained from the manufacturer, or determined by some other means.*
5.2 Structures and structural frames	
5.2.1 ***Analysis of structures***	
No change.	

Suggested modifications to BS 5400: Part 4

TEXT

5.2.2 *Redistribution of moments*

Delete whole clause and replace by 'No redistribution of the moments obtained by means of an elastic analysis may be carried out'.

5.3 Beams

5.3.1 *General*

No change.

5.3.2 *Resistance moment of beams*

5.3.2.1 *Analysis of sections*

Modify sub-clause (d) to read:

'(d) The stresses in the reinforcement in tension are derived from the stress-strain curve in Figure 2a with the appropriate γ_{me} from Table 3b. The stress should not exceed f_r/γ_m, with γ_m being obtained from Table 3a. The stress in any reinforcement in compression should be ignored.

'Modify sub-clause (h) to read:

'(h) The stresses in the reinforcement in tension are derived from the stress-strain curve in Figure 2a with the appropriate γ_{me} from Table 3b. The stress should not exceed f_r/γ_m, with γ_m being obtained from Table 3a. The stress in any reinforcement in compression should be ignored'.

COMMENTS

No redistribution should be used as FRP reinforcement has an elastic response to failure, with no yielding. Hence plastic methods are not appropriate. Further information can be found in D.2.7.

It is important to note that two different partial safety factors are required, one on the elastic modulus to determine the tensile stress and the other to check that the design strength of the material is not exceeded.

Fibre composite materials have a lower strength in compression than in tension. It has therefore been decided to ignore the contribution of bars in compression, both for beams in bending and also for columns. This approach, which is in line with other proposed design codes for fibre composite reinforcement, is conservative.

Apart from this, the approach for the design of sections is unaltered. However, it will be necessary to calculate the resistance moment from first principles rather than from the use of charts or simplified equations, until such time as design aids have been developed that are appropriate to the material being used.

Because of the relatively low stiffness of fibre composites, and hence the high strains in the material, it is likely that failure may well be by crushing of the concrete rather that rupture of the reinforcement.

Further information can be found in D.3.1.

The effect of stress reversals of a significant magnitude on the tensile strength of FRP bars is not well understood. However, this is only

Suggested modifications to BS 5400: Part 4

Text | **Comments**

likely to be of importance for seismic loading, which is outside the scope of this document.

5.3.2.2 *Design charts*

Delete whole clause.

Being for steel reinforcement, which has an elastic-plastic response, the charts are not applicable to fibre composites.

5.3.2.3 *Design formulae*

Delete whole clause.

These formulae are based on the assumption that the reinforcement behaves plastically at ultimate load and hence are not appropriate to fibre composites.

5.3.3 ***Shear resistance of beams***

5.3.3.1 *Shear stress*

No change.

5.3.3.2 *Shear reinforcement*

Replace final sentence by 'Stress in any bar should not exceed f_{rv} divided by the appropriate γ_m from Table 3a'.

Amend Table 7 as follows to form Table 7a:

Replace '$0.87f_{yv}$' by '$0.0025E_{rv}/\gamma_{me}$' twice.

Below Table 7:

Replace 'see Table 8' by 'see below'

Replace 'f_{rv} is ... than 460N/mm^2'. by 'E_{rv} is the elastic modulus of shear reinforcement, not to be taken as more than 200,000N/mm^2'.

Add, '$0.0025E_{rv}/\gamma_{me}$ should not be greater than f_{rv}/γ_m'

The current approach for shear design adds the contribution from the shear reinforcement to that of the concrete cross-section. This assumption appears to give an adequate margin of safety. However, the assumption will only remain valid when using FRP shear reinforcement if any shear crack that forms is adequately controlled. Hence the strain in the shear reinforcement has to be limited. A maximum strain of 0.0025 has been selected; this is comparable to the limiting strain implied by BS 5400 which says that the characteristic strength of steel shear reinforcement should not be taken as greater than 460N/mm^2, a strain of 0.0023. The same philosophy is used in the Draft Canadian Bridge Code which limits the strain in FRP shear reinforcement to 0.0020.

The capacity of the shear reinforcement is determined by multiplying the strain by the elastic modulus of the material. The partial safety factor used is thus that appropriate to the modulus and not the strength of the material. Further information can be found in D.3.2.

Delete Table 8

Suggested modifications to BS 5400: Part 4

TEXT

Redraft next paragraph to read:

'Values for the design concrete shear stress, v_c, should be derived from the following expression

$$v_c = 0.27(100A_{reff}/(b_v d))^{0.33} f_{cu}^{0.33}/1.25 \quad \text{... equation 8a}$$

where: $A_{reff} = A_r E_r/200{,}000\gamma_{me}$
E_r = short-term elastic modulus of the reinforcement in N/mm^2, not to be taken as greater than 200,000N/mm^2
γ_{me} = partial safety factor for elastic modulus from Table 3a
$100A_{reff}/b_v d$ should not be taken as greater than 3
f_{cu} should not exceed 40'.

In next paragraph replace 'A_s' by 'A_r' and replace 'Table 8 is' by 'equation 8a is'.

In next paragraph replace 'A_s' by 'A_r'.

5.3.3.3 *Enhanced shear strength of sections close to supports*

No change.

5.3.3.4 *Bottom loaded beams*

No change.

5.3.4 ***Torsion***

5.3.4.1 *General*

No change.

5.3.4.2 *Torsionless systems*

No change.

5.3.4.3 *Stresses and reinforcement*

No change.

5.3.4.4 *Treatment of various cross sections*

No change, apart from the following alterations.

COMMENTS

It has been assumed that the shear resistance of the concrete is a function of the effective area of tension reinforcement, determined on the basis of the elastic modulus of the fibre composite in comparison to that of steel. This assumption, which is in line with that in other draft suggestions, has been justified by comparing predicted and measured shear strengths of a large number of published tests. This empirical approach takes into account any changes in the behaviour that may be due to the lower transverse (dowel) strength of the material. Further information can be found in D.3.2.

As the shear strength is a function of the elastic modulus of the FRP it is necessary to apply a partial safety factor to the short-term value.

The alterations to these clauses are in line with those made to the clauses dealing with the shear reinforcement for beams and slabs.

Suggested modifications to BS 5400: Part 4

Text	Comments
In equations 10 and 10(a), replace '0.87f_{yv}' by '0.0025E_{rv}/γ_{me}'	*Limited further information can be found in D.3.3.*
In equation 11, and in definitions below, replace 'f_{yv}' by 'f_{rv}' and 'f_{yL}' by 'f_{rL}'	
Replace 'f_{yv} and f_{yL} should not be taken as greater than 460N/mm^2'. by '0.0025E_{rv}/γ_{me} should not be taken as greater than f_{rv}/γ_m'.	
5.3.4.5 *Detailing*	
Replace '0.87f_{yL}' by '0.0025E_{rL}/γ_{me}'	
5.3.5 *Longitudinal shear*	
No change.	
5.3.6 *Deflection in beams*	
No change.	
5.3.7 *Crack control in beams*	
No change.	
5.4 Slabs	
5.4.1 *Moments and shear forces in slabs*	
Delete 'alternatively, Johansen's ...are met'.	*Plastic methods of analysis are not appropriate when using FRP reinforcement.*
5.4.2 *Resistance moments of slabs*	
No change.	
5.4.3 *Resistance to in-plane forces*	
No change.	
5.4.4 *Shear resistance of slabs*	
5.4.4.1 *Shear stress in solid slabs: general*	
No change.	
5.4.4.2 *Shear stress in solid slabs under concentrated loads (including wheel loads)*	
In fourth paragraph, replace 'Table 8'. by 'the	*The approach used for the design of shear*

TEXT | **COMMENTS**

calculation of the shear capacity'.

reinforcement in slabs is the same as for beams. Additional information can be found in D.3.4.

In equation 13, and in the definitions below, replace 'A_{sv}' by 'A_{rv}'.

Replace '$0.87f_{yv}$' by '$0.0025E_{rv}/\gamma_{me}$'.

Add 'E_{rv} is the short-term elastic modulus of shear reinforcement in N/mm^2, not to be taken as more than 200,000N/mm^2'.
Replace 'f_{yv}' by 'f_{rv}'.

Delete 'which should not be taken as greater than 460N/mm^2'.

Add '$0.0025E_{rv}/\gamma_{me}$ should not be greater than f_{rv}/γ_m'.

5.4.4.3 *Shear in voided slabs*

No change.

5.4.5 ***Deflections of slabs***

No change.

5.4.6 ***Crack control in slabs***

No change.

5.5 Columns

5.5.1 ***General***

No change.

5.5.2 ***Moments and forces in columns***

No change.

5.5.3 ***Short columns subject to axial load and bending about the minor axis***

5.5.3.1 *General*

No change.

5.5.3.2 *Analysis of sections*

In (d) replace 'Figure 2 with γ_m = 1.15'. by 'Figure 2a with the appropriate γ_{me} from Table

As indicated earlier, fibre composite materials have a lower strength in compression than in

Suggested modifications to BS 5400: Part 4

Text	Comments
3b'.	*tension. It has therefore been decided to ignore the contribution of bars in compression, both for columns and also for beams in bending. This approach, which is in line with other proposed design codes for fibre composite reinforcement, is conservative. Further information can be found in D.3.5.*
Add new sub-section as follows; '(e) The strength of any FRP reinforcing bars in compression should be ignored'.	
5.5.3.3 *Design charts for rectangular and circular columns*	
Delete whole clause.	
5.5.3.4 *Design formulae for rectangular columns*	
Delete whole clause.	
5.5.3.5 *Simplified design formulae for rectangular columns*	
Replace 'f_y' by 'f_r/γ_m'	
5.5.4 Short columns subjected to axial load and either bending about the major axis or biaxial bending	
In equation 17 delete '+ $f_{yc}A_{sc}$'	
In definitions, replace 'and f_{yc} are' by 'is' and delete 'A_{sc} is .... reinforcement'.	
5.5.5 Slender columns	
No change.	*There have been no reported tests on slender columns reinforced with FRP material.*
5.5.6 Shear resistance of columns	
No change.	*There have been no reported tests on the shear behaviour of columns reinforced with FRP material.*
5.6 Reinforced concrete walls	
No change.	
5.7 Bases	
No change.	

Suggested modifications to BS 5400: Part 4

TEXT

5.8 Considerations affecting design details

5.8.1 ***Construction details***

No change.

5.8.2 ***Concrete cover to reinforcement***

Delete 'The cover to reinforcement ...durability (see Part 8)'.

Add new final sentence 'For structures for which fire is a significant design consideration, see **5.2.8a**'.

Add new clause as follows:

'**5.8.2a** ***Cover as fire protection***

Generally, FRP reinforcement should not be used in structures for which fire is a significant design consideration, unless the design is based on the results of fire tests. Where fire test data are not available, the cover necessary to provide the required fire protection should be determined on the basis of an analysis of the heating within the member.

The principles of the design of structures subjected to fire are that the reinforcement should be continuous, i.e. without any laps or other forms of connection, from one end of the member to the other. Steps should be taken to ensure that the ends of the member remain below the safe working temperature of the resin matrix during a fire. The cover to the reinforcement must then be sufficient to maintain the surface of the bar below the temperature at which the fibres start to degrade significantly. This data should be provided by the manufacturer.

Where the ends of the bars are not kept cool, the temperature at the surface of the bar should be limited to the glass transition temperature. This data should be provided by the manufacturer'.

COMMENTS

The durability of fibre composite material is not a function of the cover provided.

There are two stages in the degradation of the FRP reinforcement when a concrete member is subjected to fire. The first is when the temperature at a significant area of the surface of the bar reaches the 'glass transition temperature'. At this point the resin matrix will soften and hence the bond with the concrete in this region will be lost. This will limit the stress in the reinforcement that may be used for design purposes. Provided the ends of the bar are in cool regions, and hence have not lost their bond capacity, the member will behave in a manner similar to one with unbonded prestressing. Thus it will not collapse, though deflections and crack widths will obviously be increased. The limiting condition will be when a significant portion of the cross-section of the bar itself is heated to the level at which the fibres themselves lose significant strength, again limiting the stress that may be used in design.

If the ends are not anchored, the element becomes effectively unreinforced and will fail once the glass transition temperature is reached at the surface of the bar.

Design for fire is an area that requires further development, backed up by experimental work. Further information can be found in D.2.4.

Suggested modifications to BS 5400: Part 4

Text	Comments
5.8.3 ***Reinforcement: general considerations***	
No change.	
5.8.4 ***Minimum areas of reinforcement in members***	
5.8.4.1 *Minimum area of main reinforcement*	
In first paragraph delete 'when using grade 460 ... is used' In second paragraph replace 'f_y' by '$0.0025E_r$' and replace 'f_y is the characteristic strength' by 'E_r is the elastic modulus'	*The principle is that the minimum area of reinforcement provided should be capable of carrying the force carried by the concrete in tension prior to cracking. Hence the amount required is a function of the strength of the reinforcement. To limit the strains in the FRP reinforcement, and hence the widths of any cracks that form, the minimum percentages have been determined on the basis of limiting the strain to 0.0025, as for shear reinforcement. For convenience, the percentages for 460N/mm^2 have been retained though these are strictly equivalent to a strain of 0.0023. Further information can be found in D.3.8.*
5.8.4.2 *Minimum area of secondary reinforcement*	
Delete 'or 0.15% of $b_t d$ when grade 250 reinforcement is used' and similarly in second paragraph.	
5.8.4.3 *Minimum area of links*	
No change.	
5.8.5 ***Maximum areas of reinforcement in members***	
No change.	*The maximum area is controlled by the practical requirements for casting and compacting the concrete and is not a function of the type of reinforcement used.*
5.8.6 ***Bond, anchorage and bearing***	
5.8.6.1 *Geometrical classification of deformed bars*	
Delete whole clause.	*Information on the bond properties of the bars should be provided by the manufacturer.*

Suggested modifications to BS 5400: Part 4

TEXT	COMMENTS
	Additional information can be found in D.3.11.
5.8.6.2 *Local bond*	
Replace 'the appropriate value obtained from Table 14' by '80% of the value in **5.8.6.3**'	
Delete 'need not be considered ... in **5.8.6.3**'	
Table 14 Ultimate local bond stresses	
Delete whole Table.	
5.8.6.3 *Anchorage bond*	
Add the following at the start of the clause:	*The values in Table 3a take account of changes in the bond characteristics with time.*
'Values for the characteristic ultimate anchorage bond stress should be determined by the supplier of the material in accordance with approved test methods. Design ultimate values should be obtained by dividing the characteristic values by a partial safety factor, γ_m of 1.4 or by the appropriate value in Table 3a, which ever is the greater'.	*It is not clear whether the bond stress is a function of the concrete strength or not; experimental evidence is contradictory. Additional information can be found in D.3.11.*
At end of clause replace 'value obtained from Table 15'. by 'design value'.	
Table 15 Ultimate anchorage bond stresses	
Delete whole Table.	
5.8.6.4 *Effective perimeter of a bar or group of bars*	
No change	
5.8.6.5 *Anchorage of links*	
No change.	
5.8.6.6 *Laps and joints*	
Delete sub-clauses (b), (d) and (e).	
Add 'The strength and durability of any adhesively bonded or mechanical connection should be demonstrated by the manufacturer. All such connections should be made in accordance with the manufacturer's recommendations'.	*Some limited development work has been carried out on couplers, but there are currently no systems that will carry acceptable loads.*

Suggested modifications to BS 5400: Part 4

Text	Comments
5.8.6.7 *Lap lengths*	
No change.	
5.8.6.8 *Hooks and bends*	
Delete 'Hooks and bends should be in accordance with BS 4466'. Redraft third paragraph to read 'The radius of any bend should be sufficient to ensure ... the value given in **5.8.6.9**'.	*FRP bends can be fabricated to tight radii by the manufacturer, but there will still be a requirement to check that the permissible bearing stresses in the concrete are not exceeded.*
5.8.6.9 *Bearing stress inside bends*	
No change.	
5.8.7 ***Curtailment and anchorage of reinforcement***	
Replace 'f_y' by 'f_r/γ_m'	
5.8.8 ***Spacing of reinforcement***	
5.8.8.1 *Minimum distance between bars*	
No change.	
5.8.8.2 *Maximum distance between bars in tension*	
Redraft definition of c_{nom} to read '... outermost reinforcement; where the cover shown on the drawing is greater than the nominal value, the latter value may be used'.	
5.8.9 ***Shrinkage and temperature reinforcement***	
In the expression, replace 'k_r' by '0.005' and delete the definition of k_r.	*It has been assumed that the provision of reinforcement to control cracking in these circumstances is largely a function of the strength of the material and hence the factor appropriate to grade 460 steel has been adopted rather than that for 250 steel. However, this is an area that has not been considered experimentally.*
5.8.10 ***Arrangement of reinforcement in skew slabs***	
No change.	

Suggested modifications to BS 5400: Part 4

TEXT	COMMENTS

5.9 Additional considerations in the use of lightweight aggregate concrete

5.9.1 ***General***

No change.

5.9.2 ***Durability***

Delete whole clause.

5.9.3 ***Characteristic strength***

No change.

5.9.4 ***Shear resistance of beams***

Delete 'except that Table 17 should be used in place of Table 8'. and replace by 'except that the values for v_c should be taken as 80% of the value for the equivalent grade of normal weight concrete'.

Table 17. Ultimate shear stress, v_c, in lightweight aggregate concrete beams without shear reinforcement

Delete whole Table

5.9.5 ***Shear resistance of slabs***

Delete 'except that Table 17 should be used in place of Table 8'. and replace by 'except that the values for v_c should be taken as 80% of the value for the equivalent grade of normal weight concrete'.

5.9.6 ***Deflection of beams***

No change.

5.9.7 ***Shear resistance of slabs***

No change.

5.9.8 ***Deflection of slabs***

No change.

Suggested modifications to BS 5400: Part 4

Text	Comments

5.9.9 *Columns*

No change.

5.9.10 *Local bond, anchorage bond and laps*

Redraft end of clause to read '... should not exceed 80% of the values determined from **5.8.6.2** and **5.8.6.3** respectively'.

5.9.11 *Bearing stress inside bends*

No change.

6 DESIGN AND DETAILING: PRESTRESSED CONCRETE

The use of FRP tendons for prestressing is not covered by this document.

The main design changes will be concerned with the non-ferrous tendon itself, such as frictional losses, relaxation, etc. which should be provided by the manufacturer. The tendons are likely to be unbonded and hence the current rules for determining the flexural and shear capacities cannot be readily adapted. The designer should seek specialist advice elsewhere.

Because of the limited experience of the behaviour of prestressed elements with FRP used for both the tendons and the unstressed reinforcement, design by testing would appear to be particularly suitable.

7 DESIGN AND DETAILING: PRECAST, COMPOSITE AND PLAIN CONCRETE CONSTRUCTION

7.1 General

7.1.1 *Introduction*

Add the following; 'The requirements of this clause are not applicable to construction consisting of slabs cast onto FRP members, such as beams, to form a composite structure'.

7.1.2 *Limit state design*

No change.

Suggested modifications to BS 5400: Part 4

TEXT	COMMENTS

7.2 Precast concrete construction

7.2.1 *Framed structures and continuous beams*

No change.

7.2.2 *Other precast members*

No change.

7.2.3 *Supports for precast members*

No change.

7.2.4 *Joints between precast members*

7.2.4.1 *General*

No change.

7.2.4.2 *Halving joint*

Replace '$0.87f_{yv}$' by '$0.0025E_r/\gamma_{me}$'

In definitions replace 'f_{yv} ...strength' by 'E_r is the elastic modulus'

7.3 Structural connections between units

7.3.1 *General*

No change.

7.3.2 *Continuity of reinforcement*

7.3.2.1 *General*

Delete ' (b) butt welding' and delete '(d) threading of bars'

7.3.2.2 *Sleeving*

No change.

7.3.2.3 *Threading*

Delete whole clause.

Threading will not generally be an appropriate method for joining FRP bars because of the damage to the fibres and the consequent loss of strength.

Suggested modifications to BS 5400: Part 4

Text	Comments
7.3.2.4 *Welding of bars*	
Delete whole clause.	
7.4 Composite concrete construction	
7.4.1 *General*	
No change.	
7.4.2 *Ultimate limit state*	
7.4.2.1 *General*	
No change.	
7.4.2.2 *Vertical shear*	
No change.	
7.4.2.3 *Longitudinal shear*	
Below Figure 8 replace '$0.7A_e f_y$' by '$0.8A_e f_r/\gamma_m$' and replace 'f_y' by 'f_r'	
7.4.3 *Serviceability limit state*	
No change.	
7.5 Plain concrete walls and abutments	
No change.	
APPENDIX A METHODS OF COMPLIANCE WITH SERVICEABILITY CRITERIA BY DIRECT CALCULATION	
A.1 Analysis of structure for serviceability limit states	
No change	
A.2 Calculation of deflections	
A.2.1 *General*	
No change.	
A.2.2 *Calculation of curvatures*	
Redraft sub-clause (a) to read 'The modulus of	*The elastic modulus of the reinforcement will*

TEXT

the reinforcement in tension or in compression ... should be taken as E_r/γ_{me}, where γ_{me} has the appropriate value from Table 3b'.

In sub-clause (a)(4) change 'steel' to 'reinforcement'.

Add new paragraph as follows:

'The modulus of elasticity of the reinforcement should be appropriate to the age of loading at which the curvatures and stresses are being calculated'.

Redraft sub-clause (b) to read:

'The concrete and the reinforcement are both considered to be fully elastic in tension and compression. The elastic modulus of the reinforcement should be taken as E_r/γ_{me}, where γ_{me} has the appropriate value from Table 3b, and the elastic modulus of the concrete...'.

A.2.3 *Calculation of deflection from curvatures*

No change.

APPENDIX B ELASTIC DEFORMATION OF CONCRETE

No change.

APPENDIX C SHRINKAGE AND CREEP

C.1 General

No change.

C.2 Creep

No change.

C.3 Shrinkage

Replace 'steel' by 'reinforcement'.

C.4 Reinforced concrete

Replace 'steel' by 'reinforcement'.

COMMENTS

reduce with time. The short-term value, which should be provided by the manufacturer, should be divided by the partial safety factors in Table 3b.

When calculating instantaneous deflections at early ages, the suggestions in the new 4.3.3, that the materials safety factors may be reduced, will apply.

As the stresses and strains are a function of

Text | **Comments**

α_e, the modular ratio between the reinforcement and the concrete, the approach should be equally valid when using FRP reinforcement, taking the appropriate value for the elastic modulus of the material.

C.5 Prestressed concrete

This document does not cover the use of FRP for prestressing.

APPENDIX D COVER AND SPACING OF CURVED TENDONS IN DUCTS FOR PRESTRESSED CONCRETE

This document does not cover the use of FRP for prestressing tendons.

PART D

Background to modified clauses

Part D Background to modified clauses

D.1 Introduction

Draft guidelines for the design of structures reinforced with FRP material have been published by the Japanese Ministry of Construction[16, 17] and, for bridges, by the Canadian Standards Association[6, 18]. In addition, a State-of-the-Art Report has been prepared by the American Concrete Institute[19].

Modified design rules have been developed for the following design Codes:

- BS 8110: *Structural use of concrete,* Part 1, *Code of Practice for design and construction.*, Part 2, *Code of Practice for special circumstances*
- BS 5400: *Steel, concrete and composite bridges,* Part 4, *Code of Practice for design of concrete bridges.*

These are given in Parts B and C of this document respectively. This Part outlines the background to the changes to the Codes and reviews relevant test information. It should be pointed out the information comes from a range of different research organisations, working with various types of material. In modifying the clauses it was felt that it was appropriate to change only those that are concerned directly, or indirectly with the reinforcement materials. Thus no changes have been made to those covering general principles such as limit state design, nor to those dealing with the geometry of the structure, such as the slenderness of columns. Similarly, though there may be valid reasons for lowering the specification for the concrete when using non-ferrous materials, such as the reduced requirement for corrosion protection, clauses have been left unaltered to maintain adequate performance in other areas such as abrasion and freeze–thaw resistances.

In some cases partial safety factors are not included in the original Code clauses dealing with reinforcement, the factors being included in the coefficients in the design expressions. This is a valid approach when using steel, which has a single partial safety factor irrespective of its grade, but is not appropriate when using a range of non-ferrous materials. Thus, in general, partial safety factors will have to be inserted in all expressions and the coefficients modified accordingly.

Obviously, no changes have been made to the partial safety factors to be applied to the loads.

Section D.2 considers general aspects of design while Sections D.3, D.4 and D.5 consider the detailed requirements. In the latter sections appropriate test evidence is reviewed and methods for amending the Codes are outlined. Where appropriate, test data are compared with the predicted values from the amended BS 8110 clauses; comparisons with the BS 5400: Part 4 amended Codes will be similar.

It is assumed that Part D will be read by suitably qualified engineers who will satisfy themselves that the suggested design approaches are reasonable in the relevant application.

D.2 General design considerations

D.2.1 Materials

In line with the approach adopted currently by the Codes a characteristic value should be used for the property under consideration, i.e. not more than 1 result in 20 falling below that value. This would, for an infinite population of results, be the mean value less 1.64 standard deviations.

The draft Canadian Bridge Code[6, 18] proposes a 95% confidence level for the strength of the reinforcement material. However, for the elastic modulus they propose taking the mean value. The document outlines test methods for determining the characteristic strength on the basis of a limited number of test samples.

D.2.2 Durability and partial safety factors

Many researchers appear to rely on manufacturers' over-optimistic claims and assume that fibre composites will be durable. However, the ACI State-of-the-Art-Report[19] identifies alkaline attack as one that 'may pose serious problems with regard to the durability of glass FRP bars'. It is also known that alkalis attack some aramid fibres and some resins. In fact the key consideration must be the durability of the composite and not necessarily of the fibre and resin in isolation. This is recognised by the draft Canadian Bridge Code[6, 18] which limits the types of applications in which certain fibre composites may be used.

Composites are known to deteriorate with time, particularly in an alkaline environment. In fact, the conditions aimed for in conventional design, namely a high alkaline environment to provide the greatest protection to the steel, will be the most detrimental to composites. Thus the partial safety factors applied to composites must reflect not only the variability of the material as produced but also the anticipated worst long-term behaviour.

There is very little independently validated long-term data for composite performance in the concrete environment. Any data that may be published will be relevant to the particular material tested. Hence, the partial safety factors to be applied to the short-term properties proposed in the following section may well err on the side of caution. They should be considered as initial values which may be modified according to materials developments and the provision of validated test data. It is particularly important to note that durability will be significantly affected by the ambient temperature. If the temperature is significantly higher than normal, the factors will have to be increased to allow for the increased rate of degradation.

The proposed factors would appear very unfavourable when compared with the value of 1.05 currently used in BS 8110 for steel, which is only intended to cover the variability of the material as produced and to make some allowance for errors in its location in the member. All the properties of the steel, and its interaction with the concrete, are assumed to be constant throughout the life of the structure. The durability of the reinforcement is, in theory at least, ensured by the provision of the correct minimum cover of concrete of a quality appropriate to the environment in which the structure is located. No allowance is made in conventional reinforced concrete design for the rusting of steel and the subsequent loss of safety. If this were done, rather than relying on the concrete to provide the necessary protection, a more realistic steel partial safety factor might be 1.5 to 2.0.

For those aspects of design that involve the stiffness of the FRP rather than its strength (e.g. shear of beams), a new partial safety factor, γ_{me}, has been introduced. There are based on 50% of the factors derived from durability testing multiplied by the 1.1 required for inaccuracies in placement.

The above partial safety factors are intended to cover uncertainties in the short-term properties and also the long-term behaviour. For situations in which the reinforcement is only subjected to short-term loads early in the life of the structure, such as early thermal and shrinkage effects, the factor applied to the strength may be reduced to 1.25 for all materials and the factor applied to the stiffness taken as 1.1 for all materials.

D.2.3 Crack widths and crack control

It is generally considered that it is necessary to control cracks in reinforced concrete from aesthetic considerations and from the point of view of durability. In addition, crack widths in water-retaining structures will have to be controlled to eliminate leakage. It is well known that, in the presence of a supply of water, narrow cracks will tend to heal although wider cracks will tend to remain open. In recent years experimental work has shown that the durability aspect of crack control is of less importance, though BS 5400: Part 4 still includes clauses linking the design crack width to the environment.

While the surface crack width may influence the time to the initiation of corrosion in steel reinforcement, it is the quality of the cover that dictates the rate of corrosion. This is reflected in the requirement linking the quality of the concrete and the cover to the reinforcement to the severity of the environment. However, there is still a general requirement in BS 8110 to limit crack widths to an acceptable level of 0.3mm, which is generally achieved by detailing rules which limit bar sizes and spacing.

The draft Canadian Standard(6) suggests crack widths of 0.5mm for exterior surfaces and 0.7mm for interior surfaces when using FRP reinforcement.

In the light of the above, it has been assumed when amending the clauses that crack widths will not affect the durability of non-ferrous reinforcement. However, there will still remain the question of aesthetics, which is more difficult to quantify. It would appear that a limit of 0.3mm would seem to be reasonable for surfaces that are close to the observer though it could be relaxed for surfaces that are further away. In extreme situations, such as inside surfaces of box structures, there need be no limit to the crack width. However, any strain limitations applied to the reinforcement would in effect limit the crack width.

The method of calculating the crack width is considered in Section D.3.6.

D.2.4 Bond

An adequate level of bond is required between the reinforcement and the concrete to transmit forces from one to the other. Particular areas of concern are the anchorage at the ends of bars and at laps where loads transfer from one bar to an adjacent one.

The amount of bond achieved varies considerably, depending on the type of FRP reinforcement. However, it is important to note that a very high bond strength can lead to unacceptably high local stresses in the reinforcement. This situation does not occur with steel reinforcement, which will yield locally, relieving the peak stresses. This is an area that is being studied at present.

For steel the Codes give ultimate bond stresses, in tension or compression, for plain bars and for one or two types of deformed bars. Other aspects of design assume an adequate level of bond, for example the bending of beams where the strain in the tension reinforcement is assumed to be the same as that in the surrounding concrete. In other words, it is assumed that there is no slip. Similarly when assessing the shear of beams it is assumed that the main bars are anchored.

With non-ferrous bars the capacity of any deformations on the surface are likely to be controlled by the shear capacity of the resin matrix(20). Even with those bars that consist of a pultruded core overwound with a fibre reinforced rib to simulate a steel bar, the shear capacity of the resin will control the bond capacity, as there is no continuity between the core and rib fibres. Thus a pull-through failure would be expected, with the plane of weakness being through the base of the deformations or ribs.

For a particular bar it will be necessary to determine an 'acceptable' bond strength from pull-out tests. Then two partial safety factors will have to be applied to allow for:

- variations in the manufacturing process
- changes with time due to attack of the resin by the alkaline concrete environment.

When the reinforcement is only required for early thermal movements and handling stresses no allowance need be made for long-term effects. Information on the variations in the manufacturing process can only be obtained from pull-out tests carried out at regular intervals as part of the quality control process.

The design value for bond will be the characteristic value obtained from pull-out tests, divided by the product of the two partial safety factors.

D.2.5 Fire

The behaviour of non-metallic reinforcement in concrete is an area of obvious concern which needs to be considered both analytically and experimentally. Heating of the bar will lead to softening of the resin matrix (at the 'glass transition temperature' which will depend on the type of resin being used) which will result in a loss of composite action with the fibres. Thus it will be necessary to limit the temperature rise at the surface of the bar to the glass transition temperature, until experimental evidence is available. If the ends of the bar are adequately anchored and are sufficiently protected from the effects of the fire it may be possible to allow higher temperatures at the surface of the bar. The requirement would be that the core of the bar should be below the glass transition temperature and the core should still be capable of carrying the loads. Under fire conditions, the specified partial safety factors applied to both the loads and the material properties will be lower than under normal situations.

There is only limited data on the behaviour of non-metallic composites at elevated temperatures. Some work on composite bars heated in air has been carried out by Kumahara *et al*(21) and by Chaallal and Benmokrane(22). Widely differing results were obtained, reflecting the different resins and fibres.

Fire tests have been carried out by Tanano *et al*(23) who heated beams, which were then cooled to ambient and load tested in bending. All showed a significant drop in strength.

Some fire testing on lightweight cladding panels has been reported by Fujisaki *et al* (24) for 30 minutes in accordance with the Japanese Industrial Standard. The insulation provided by the concrete was such that the temperature of the reinforcement was about 300°C. On the assumption that the reinforcement would still maintain 60% of its strength the panel was assumed to be satisfactory.

Part 2 of BS 8110 gives detailed guidance on design for fire, which considers the flow of heat into the concrete and takes into account the deterioration of the strength of both the concrete and the steel at elevated temperatures. This should be an appropriate approach for determining the behaviour of FRP material at high temperatures. As indicated above, any design would be on the basis of two different criteria, namely:

- the resin at the surface of the reinforcing bar is raised to such a temperature that it softens, leading to a significant loss of bond
- the bar itself is heated to such a temperature that the fibres start to lose their properties to a significant extent.

The choice of the appropriate criterion will depend on the anchorage conditions for the bars.

This is an area that requires considerable additional work.

D.2.6 Analysis of structures

All the non-metallic materials considered show a straight line response to ultimate, with no significant yielding. Tests on concrete beams reinforced with the materials have produced load deflection curves with a limited plateau at ultimate, demonstrating a small amount of pseudo-plastic behaviour. However, little work has been carried out on the rotation capacity of frames and slabs and hence of any redistribution of moments that might take place. Tezuka *et al*(25) tested continuous beams which showed that a maximum redistribution of moment of about 10% occurred.

In the light of this limited evidence it has been assumed that only elastic methods of analysis may be used, with no redistribution. This is in agreement with the draft

Canadian Standard(6). But, sections reinforced with FRP tend to have high curvatures at failure, caused by the large strain capacity of the material and cracking in the concrete. However, no plastic work is done on the fibres, since they are elastic up to failure. However, redistribution in a continuous member is a function of the rotation capacity, not the plasticity of the material. Work is needed in this area, which should lead to more economic design solutions.

No change has been proposed for the clauses dealing with the flexural stiffness constants for analysis at service and ultimate. Although crack widths will be wider with the non-metallic reinforcement, the uncracked region between the cracks will still play a major role in the distribution of moments and shears and hence the approaches used in the Codes should still be viable.

D.3 Design and detailing: reinforced concrete

D.3.1 Resistance moment of beams

Introduction

The basic principles of the behaviour of a beam in bending do not depend on the type of reinforcement material. Hence they need not be changed provided the reinforcement is adequately bonded. This has been confirmed by Kawaguchi(26) who recorded near-linear strain profiles through the depth at 95% of the ultimate load.

If a beam with a number of reinforcing bars is loaded to failure, one of the bars will reach its ultimate capacity before the others. With steel, the bar will yield, with additional load being carried by the adjacent bars. With FRP the bar will snap. Thus the strength of the member will not be governed by the sum of the strengths of the individual bars but by some complex relationship involving the variability of the bars. This effect is covered by bundle theory. Currently the effect will be covered by the relatively high factors of safety that are applied, but as factors are reduced it will become important that work is carried out in this area.

To reach the full capacity of the reinforcement it would probably be necessary to provide confinement to the compression zone, which would increase the failure strain of the concrete. One possible approach might be to incorporate chopped fibres into the concrete of the compression zone. With a sufficient percentage, the ultimate strain in the concrete would be significantly increased, though the strength may be reduced slightly.

Concern has been expressed by a number of authors because of the lack of ductility of FRP reinforcement, though it is not clear whether their real concern is that structures should absorb large amounts of energy prior to failure or that they should give adequate warning of failure. In practice the latter will be covered by cracking and deflection, which will be higher than for steel reinforced members because of the lower elastic modulus of the composite.

Alternative methods for determining ductility have been proposed; the current American Concrete Institute requirements are for a strain in the reinforcement, at failure of the member, of at least 0.005(19). However, this could impose a restriction on the use of high modulus FRP materials.

Review of test work

A large number of beams have been tested worldwide, with or without shear reinforcement(27–36). In addition, some slabs have been tested in bending(37).

Amendments to Code requirements

The basic principles in the Codes will be unchanged but the stress-strain curves for the reinforcement will have to be replaced. The strength of any reinforcement in compression should be ignored (see Section D.3.5). Where more than one layer of tension reinforcement is provided, the stress in the bars in each layer should be computed separately, unless it can be shown that failure will occur by compression of the concrete. Failure of the reinforcement will occur first in the outermost layer, i.e. the layer at the greatest strain, and this should be taken as being the ultimate capacity of the beam.

Design charts, such as those in Part 3 of BS 8110 are obviously not applicable to non-metallic reinforcement. Similarly the design equations, which assume steel yielding, are not valid.

The clauses in BS 5400: Part 4 have the additional requirement that the reinforcement should be yielding, or else the calculated moment should be 15% greater than the required value, to ensure adequate warning of failure. To meet this criterion the additional 15% capacity should be required whether failure is expected in the concrete or in the reinforcement, when designing to BS 5400: Part 4.

Comparisons with data

The results from the tests outlined above have been compared with the predicted values, using the BS 8110 approach. Table D.1 gives the average values of measured divided by predicted moment for the three basic types of reinforcement considered. This shows that the approach gives good results.

Figure D.1 shows the results in graphical form, demonstrating the good agreement between measured and predicted values throughout. Applying a partial safety factor to both the concrete and the reinforcement would give a safe design approach.

Table D.1 Average values of measured bending divided by predicted bending

Tensile reinforcement	Measured + predicted moment
Glass	1.02
Aramid	1.20
Carbon	1.03
Average	1.04

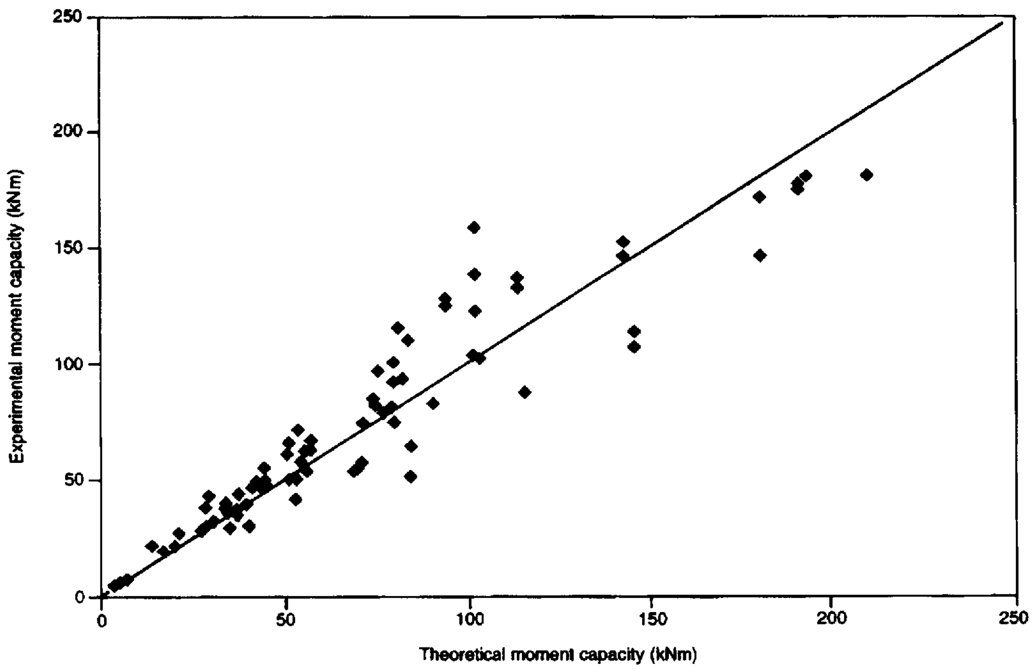

Figure D.1 Predicted moment capacity v. measured moment capacity

D.3.2 Shear of beams

Introduction

Both Codes assume that the shear resistance of the cross-section is the sum of the shear resistance of the concrete and the capacity of any shear reinforcement provided. Though not a true representation of the behaviour, the approach is simple to adopt and would appear to give adequate margins of safety. It is therefore logical to retain the same approach with non-metallic reinforcement, at least until more experimental evidence is available.

The ultimate shear stress on the concrete is a function of the characteristic strength of the concrete and the area of effectively anchored longitudinal tension reinforcement. The former will be unchanged but the latter requires modification. As only the area of the reinforcement is considered in the Codes, with no reference to its characteristic strength, it must be assumed that its stiffness is the governing parameter. Thus when using non-metallic reinforcement, with a lower elastic modulus than steel, the area should be modified by multiplying it by the ratio of the modulus to that of steel. This approach has been suggested by a number of other authors, such as Nagasaka *et al*(38).

The design of shear reinforcement is based on the assumption of a simple 45° truss. The required area of reinforcement is a function of the difference between the applied shear stress and the shear capacity of the concrete cross-section, divided by the design strength of the reinforcement. Thus, in theory, when using non-metallic reinforcement, it should only be necessary to modify the expression to incorporate the appropriate design strength. However, the design assumption that the concrete and shear reinforcement capacities can be added together is likely to be valid only if any shear crack that forms is adequately controlled. Hence the strain in the main reinforcement needs to be limited. This approach is used in the draft Canadian Bridge Code(6), which limits the strain in shear reinforcement to 0.002.

Review of test work

Beams with glass fibre main reinforcement but no shear reinforcement have been tested by Nawy and Neuwerth(29, 30). A number of investigators have tested beams with shear reinforcement, using a range of different materials(33, 38, 39). Maruyama and Zhao also considered the effect of size on the shear capacity of beams(40). They concluded that the behaviour was broadly in line with the approach in the British Codes.

Tests by Nagasaka *et al*(38), and by others, have shown that the strength of a link at the start of a bend is significantly lower than in the straight portion. The reduction can be as much as 50% of the full strength. This will significantly reduce the effectiveness of links.

The results of the above tests are compared with a proposed method for designing for shear outlined below.

Amendments to Code requirements

When determining the shear capacity of the concrete cross-section, the area of longitudinal tension reinforcement A_r is transformed into an effective area, A_e, for use in the existing expressions, by multiplying by the modular ratio:

$$A_e = A_r\,(E_r/200)$$

where E_r is the modulus of elasticity of the reinforcement in kN/mm^2, which should not exceed 200.

The partial safety factor of 1.25 applied to the shear stress in the concrete in the British Codes should be replaced by a factor of 1.5 applied to f_{cm} and γ_{ms} applied to A_sE_s.

Hence the BS 5400: Part 4 expression becomes:

$$v_c = 0.27(A_r E_r/\gamma_{ms} 2 b_w d)^{0.33}(f_{em}/1.5)^{0.33}$$

The BS 8110 expression can be rewritten into the same format as that in BS 5400 and would be amended similarly.

The strain in the shear reinforcement at ultimate should be limited to 0.0025 (which is slightly greater than the maximum strain implied by British Codes which limit the stress for steel to 460N/mm^2). Thus for design purposes the characteristic strength of the shear link material should be taken as:

$$0.0025\, E_{rv}$$

where E_{rv} is the modulus of elasticity which should be divided by the appropriate partial safety factor γ_{ms}. This design effective strength should not exceed the strength of the material measured near any bend nor should it exceed f_r/γ_{mu} where f_r is the characteristic strength of the material.

BS 8110 and BS 5400: Part 4 require a minimum amount of shear reinforcement equivalent to a shear stress of 0.4N/mm^2. It has been assumed that the minimum requirements are not a function of the type of shear reinforcement. Hence, links should be provided to carry the minimum shear stresses specified in the codes.

The check on the amount of longitudinal reinforcement required by BS 5400 is basically unaltered but the 0.87 should be replaced by $(1/\gamma_{mu})$ to make it more generally applicable.

It is assumed that the detailing requirements for the shear reinforcement will be unaltered as they are intended to ensure that the truss mechanism operates and are not a function of the material properties.

Care must be taken to ensure that the shear reinforcement is anchored in the compression zone of the beam, which will tend to be shallower than that in an equivalent steel reinforced beam. The depth of the compression zone should be determined from consideration of the bending moment applied to the section. If there is doubt that the shear reinforcement is fully anchored, its contribution to the shear capacity of the section should be ignored. In this case the design should ensure that the capacity of the concrete section alone is at least 0.4N/mm^2 greater than the applied shear stress.

Comparisons with data

The results from the tests outlined above have been compared with the predicted values determined using the modified approach described above. Partial safety factors have been taken as 1.0 throughout. In general the ratio of measured to predicted strength is above 1.0 indicating that the proposed approach is safe. Table D.2 shows overall averages, irrespective of author, for the various combinations of flexural and shear reinforcement.

The Table would suggest that the approach is reasonable for beams without shear reinforcement but that the restriction of a strain of 0.0025 in the shear reinforcement may be unnecessarily severe.

Figure D.2 shows all the data plotted against the values predicted by the amended BS 8110 approach. The scatter of results is larger than for beams in bending as would be expected. Thus the modified approach would appear to be safe with only a very few

Table D2 Average values of measured divided by predicted shear strengths

Shear reinforcement	Flexural reinforcement		
	Glass	Aramid	Carbon
None	1.25	1.95	1.10
Glass	1.46	1.87	1.98
Aramid	–	1.71	–
Carbon	–	1.36	1.51
Steel	0.93	–	–

points falling below the 1:1 line. These comparisons have been made with partial safety factors set to 1.0 and actual material strengths. For design purposes an overall partial safety factors of at least 1.25 would be applied to the expression for the capacity of the concrete and a somewhat larger factor to the shear link capacity.

Thus when compared with design values the experimental results would all lie safely above the 1:1 line.

Current approaches to shear in design Codes are based on the assumption of plasticity both in the steel and the concrete. Any compatibility problems between the steel and the concrete are overcome by the assumption that the steel is yielding. This will not be applicable to FRP materials. In the present proposal, the strain in the FRP shear reinforcement has been limited so that it can be assumed that the concrete is still effective. Further testing is required to form the basis for a more appropriate approach.

D.3.3 Torsion

Only one series of tests would have appeared to have been carried out on FRP reinforced beams in torsion. Yonekura *et al*[41] tested beams in pure torsion or combined bending and torsion. The authors identified premature failure of the torsional reinforcement at its corners, as already noted for shear reinforcement (see Section D.3.2). However they did not record the stress in the reinforcement at failure

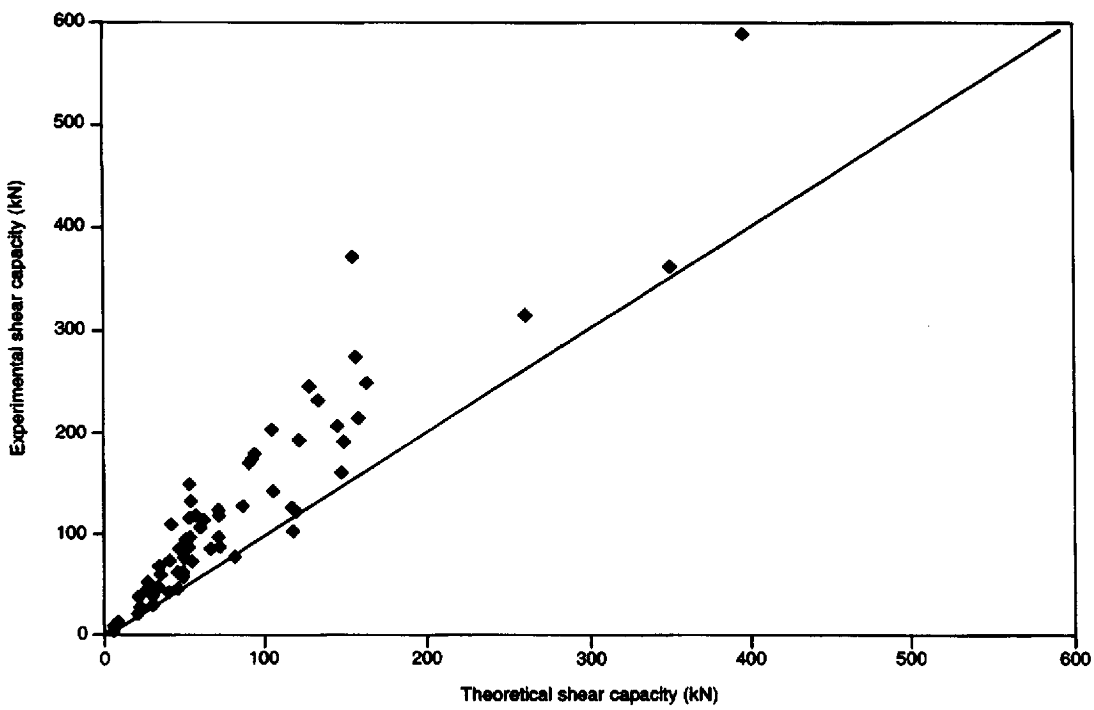

Figure D.2 Predicted shear capacity v. measured shear capacity hence the results cannot be compared with the method in the British Codes.

Until more data become available, it may be assumed that no change to the approach is required when using non-metallic materials. Minor editorial changes are required in the Code equations to introduce the appropriate γ factors in place of that used for steel.

D.3.4 Shear resistance of slabs

Very limited work has been carried out on slabs reinforced with FRP. Ahmad *et al*(42) tested slabs reinforced with a 3-dimensional carbon fibre grid while Matthys and Taerwe(43) used either a 2-dimensional grid or rods. This is obviously an area that requires further research.

The determination of the shear resistance of slabs in BS 8110 and BS 5400: Part 4 is based on that of beams and hence the assumptions detailed above for beams should be equally applicable.

If the comparisons with BS 8110 that were made in the paper by Ahmad *et al*(42) are modified to take account of the elastic modulus of the non-ferrous reinforcement material, the quoted ratios of measured:predicted ultimate strengths become on average 1.04. Similarly for Matthys and Taerwe the average is 1.57. Thus the approach would appear to be reasonable.

D.3.5 Columns

Introduction

The basic principles of the design of a reinforced cross-section should be unaltered. Hence the only adjustment to clauses dealing with the analysis of sections should be to redefine the stresses in the reinforcement and the appropriate partial safety factor. In compression the stresses will be limited to values lower than those permitted in tension.

As discussed under Section D.3.1, a design method that relates the strain capacity of the concrete in compression to the confinement provided by the main bars and the links needs to be developed to realise the full potential of the tensile reinforcement. This is an area that is being considered by a number of researchers.

Review of test work

Very limited test work has been carried out on columns with embedded FRP reinforcement. However, some studies have been made of the effects of the confinement provided by external fibre-composite reinforcement.

Kobayashi and Fujisaki(44) carried out a series of tests on composite bars in compression. In all cases the compressive strength was considerably below the tensile strength, in the range 30–50% for carbon and 30–40% for glass but only 10% for aramid. Tests by Chaallal and Benmokrane(22) concluded that the compressive strength of glass fibre rods was about 75% of the tensile strength.

Kobayashi and Fujisaki also tested a number of short columns; they concluded that the strength of the column should be based on the gross concrete cross-section only and that the area of the bars in compression should be ignored. The authors noted some increase in loadcarrying capacity attributable to the confinement provided by the hoop reinforcement but they ignored the effect in their proposed design approach. In fact, considering strain compatibility, it would be reasonable to assume that there is a corresponding stress in the bars at failure of the concrete.

A number of series of tests(36–42) have considered the effects of confinement on the ultimate behaviour (see Figure D.3) but the approaches being developed are outside the scope of this document.

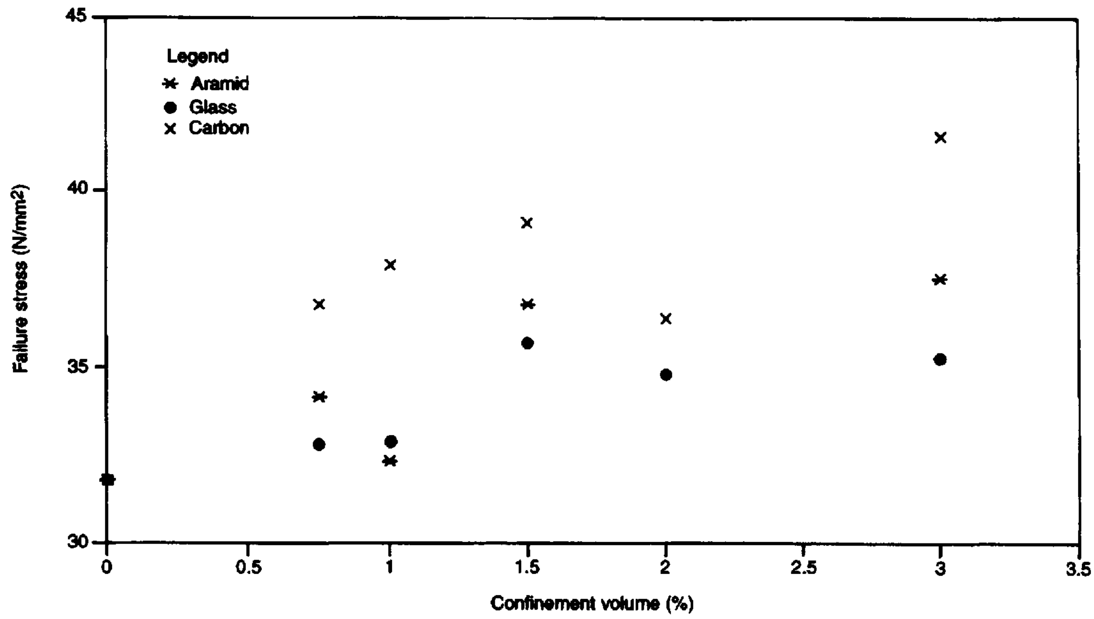

Figure D.3 Effect of confinement on failure stress

Amendments to Code requirements

The following section applies to axially loaded columns only. It has been assumed that clauses that relate solely to the geometry (e.g. slenderness) need not be altered.

At the normal failure strain assumed in design (0.0035 in British Codes) the longitudinal bars will carry some stress. However, their strength in compression is limited, as discussed in Section D.5.2. Hence, in the light of the limited test data available, it is recommended that, in line with the Japanese Ministry of Construction Guidelines(16) and the draft Canadian Standard(6), the longitudinal bars should be ignored when considering the compressive strength of columns and the strength calculated on the basis of the gross concrete area.

D.3.6 Crack control

Introduction

As discussed in Section D.2.3, there will probably be a need to control crack widths in some structures from the point of view of aesthetics. In this situation, until 'deemed to satisfy' rules based on bar spacing are developed, it will be necessary to calculate crack widths.

The Codes link surface crack width to the strain in the nearest reinforcing bar, taking no account of the type of reinforcement nor of its bond. It is not clear what approach should be adopted when using non-ferrous reinforcement, but it is likely that no change should be necessary, provided that the bond is 'adequate'.

Though there are only a few papers that have reported measuring crack widths, Nakano *et al*(35) tested beams while Mogahadam and Sentler(52) tested slabs. The crack width was calculated for each beam at its service load, taken as approximately half the actual failure load. The beams tested by Nakano *et al* were subjected to reverse bending. Hence cracks were only calculated during the first cycle of loading.

Figure D.4 shows measured crack widths plotted against the predicted values. It may be seen that the measured values are generally higher than predicted, up to a maximum of about 1.4 times. However, Beeby(53) suggests that the BS 8110 formulae predict, at service load, a crack width with a 20% chance of being exceeded

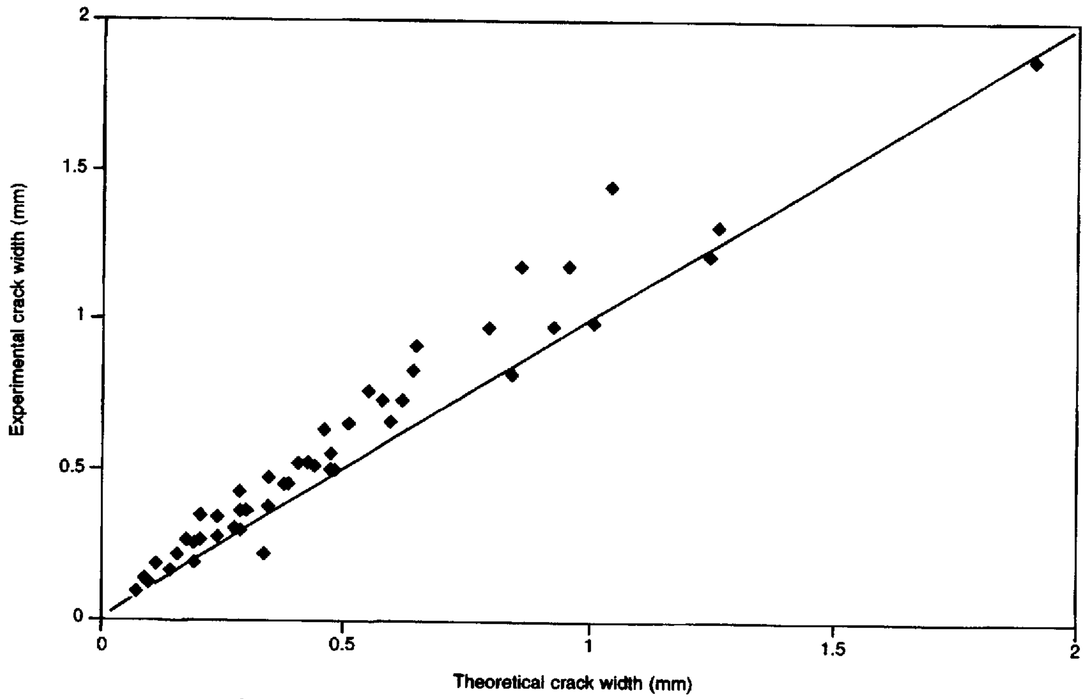

Figure D.4 Measured v. predicted crack widths

and that measured crack widths may be up to 1.2 times the predicted values. Assuming that a similar distribution of crack widths will occur with FRP reinforcement, it would appear from Figure D.3 that the BS 8110 approach gives a reasonable estimate of the crack width, though it may be necessary to apply a modifying factor in the region of 1.2. This is an area that requires further study.

D.3.7 Deflections

Introduction

Section 3 in Part 2 of BS 8110 gives two approaches for determining the deflection of beams. The first requires the calculation of curvature at successive sections along the member followed by a numerical integration to determine the deflection. The second, simpler, approach assumes that the deflection is equal to:

$$Ks^2c$$

where:

s is the effective span

c is the curvature at mid-span or, for cantilevers, at the support

K is a constant that depends on the shape of the bending moment diagram.

A table gives values of K for various bending moment diagrams. The deflection is calculated on the assumption that the section properties are the same throughout the length of the member, i.e. if it is cracked at mid-span then it is assumed to be cracked throughout the length. This is obviously a safe simplification as the parts of the beam near the supports will be uncracked, leading to reduced rotations.

For calculating the rotation of cracked sections, the Code assumes that the concrete below the neutral axis acts in tension, with a value at the centroid of the tension steel of $1.0N/mm^2$. (This reduces to $0.55N/mm^2$ when determining long-term rotations.)

Assuming some tensile strength for the concrete at a cracked location makes an allowance for the capacity of the concrete in the uncracked region between cracks. This gives an approximation to the average deflection behaviour of a member with cracked sections separated by uncracked sections of significantly higher stiffness.

The design approach should be equally applicable to beams with FRP reinforcement. The only uncertainty is whether the figure of 1.0N/mm^2, to allow for the stiffness of the zone between cracks, is still applicable.

Review of test evidence

A number of authors have measured deflections on FRP reinforced beams[31, 34, 35, 39, 54]. When comparing the data with the BS 8110 predictions only the first cycle of loading has been considered.

Modifications to Codes

As the approach in the British Code is based on the calculated rotation at the point of maximum moment, which will automatically take into account the type of reinforcement used, no additional modification to the calculation method should be necessary.

The design Codes also specify allowable span:depth ratios for structures for which no deflection calculations are carried out. They are based on the assumption that steel reinforcement is used, which has the same modulus of elasticity irrespective of its strength. When changing to non-ferrous material it might be logical to multiply the allowable span:depth ratios by (E_r/200). However, this would probably lead to too drastic a reduction in the span:depth ratios. While it would be appropriate for fully cracked sections, the concrete in the uncracked region between cracks will stiffen the beam and reduce deflections. This will have a relatively larger effect when using reinforcing bars of a lower stiffness. This is an area that requires further study.

Comparison with data

Calculation of the rotation at the point of maximum moment requires a value for the modulus of elasticity of the concrete. For the purposes of this comparison the mean values given in BS 8110 for the static modulus have been used. Deflections have been calculated at approximately half the ultimate load of the beam, taken as being the service load. To ensure that the assumption of an elastic response for the concrete is realistic, the compressive stress has been limited to half the cube strength.

Figure D.5 shows comparisons between measured and predicted deflections using data from the tests reviewed above.

It may be seen that the agreement is generally very good. Few of the measured deflections are significantly greater than the predicted values. Some are significantly below, presumably indicating that the concrete had not actually cracked though the calculations suggested that it had. Thus it may be concluded that the approach in the British Code may be used to calculate deflections with sufficient accuracy.

The allowable span:depth ratios in BS 8110 are intended to limit deflections to span divided by 250. For most of the beams tested the span was about 2.5m leading to an acceptable deflection of about 10mm. Most of the results lie below this value; those that are significantly above are from high strength concrete beams (90N/mm^2). The span:depth ratio suggested by BS 8110 for a steel reinforced simply supported beam is 20. For the FRP reinforced beams actually tested it was about 8. This is roughly the span:depth ratio that would be obtained by modifying the steel value by the relative stiffness when using carbon or aramid bars.

Thus one may conclude that the approach suggested above is appropriate, at least as a

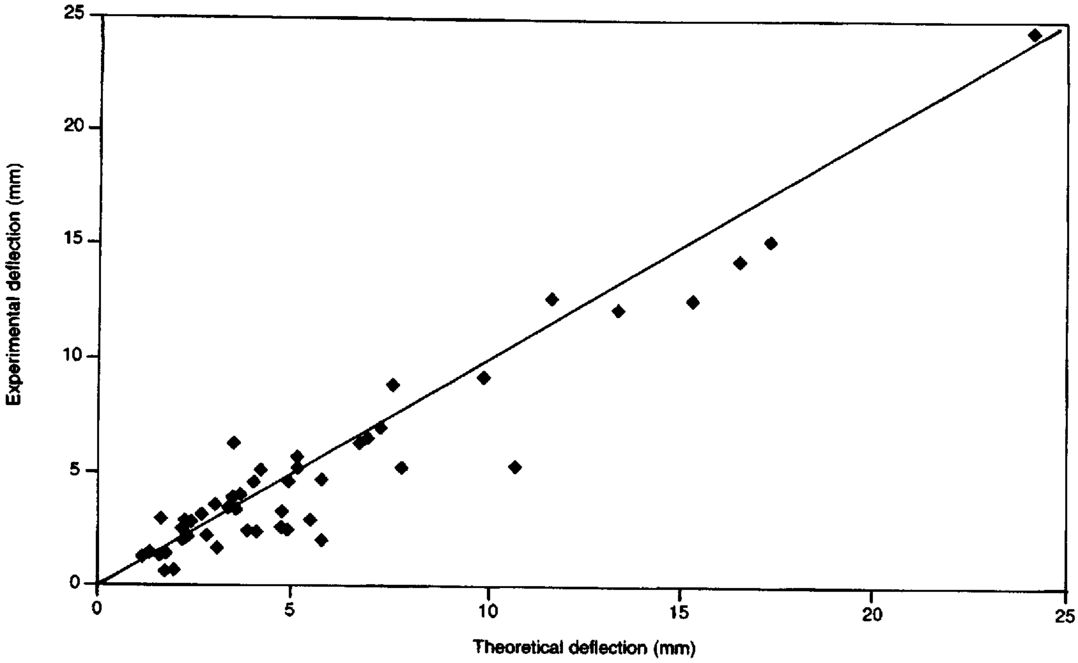

Figure D.5 Measured v. predicted deflection

first estimate. However, further data are required before firm guidance can be given.

D.3.8 Minimum amounts of reinforcement

In general a minimum amount of reinforcement is required to control, rather than to prevent, the formation of cracks. The basic approach is that the tensile strength of the concrete should be replaced by an equivalent amount of reinforcement. It should be pointed out that there has been no published work on the use of non-metallic material to provide minimum amounts of reinforcement. The proposals given below may be unnecessarily conservative.

BS 5400: Part 4 gives minimum amounts of tensile reinforcement for beams and slabs: 0.15% of high yield reinforcement or 0.25% of mild steel is required. To make these requirements applicable to non-metallic reinforcement, a simple expression may be derived, incorporating the characteristic strength of the material. In addition the material partial safety factor should be included. Thus the minimum area becomes:

$$0.6\gamma_m b_a d/f_r$$

Reverting to steel would give 0.15% for high yield material and a slightly increased value of 0.28% for mild steel.

Similar adjustments are made to requirements covering minimum areas of secondary reinforcement, which may be particularly important for controlling any cracks caused by the different coefficients of expansion of the main reinforcement and the concrete.

Elsewhere minimum amounts of reinforcement are specified which are based solely on the area of the cross-section and are unrelated to the type of reinforcement. It may be assumed that these requirements are stiffness related and hence the specified amounts should be multiplied by the ratio of the elastic modulus of steel to that of the non-metallic reinforcement. It may be necessary to make the same adjustments to the areas calculated on the basis of strength discussed above.

BS 8110 has similar requirements, though the minimum percentage of reinforcement is based on the full, uncracked area of the concrete while the bridge Code uses the effective depth. In addition, minimum percentages are given for less common cross-sections or loading situations.

Minimum areas of links have to be provided for compression members. It is usually assumed that they are there to prevent buckling of the reinforcement and not to provide any form of biaxial restraint to the concrete. (This latter may be highly significant for the compression zones of beams and for columns reinforced with non-metallic reinforcement as discussed earlier). Thus the requirements need not be altered, unless the stiffness of the link material differs from that of the longitudinal material. It is suggested that if the link is less stiff the required area should be increased in proportion to the relative stiffness but should not be reduced if the converse is true. Once again there has been no reported experimental work in this area.

D.3.9 Shrinkage and temperature reinforcement

The amount of reinforcement required to prevent excessive cracking due to shrinkage and thermal movement is assumed to be sufficient to replace the tensile strength of the concrete. There has been no reported test work on this topic.

D.3.10 Maximum amounts of reinforcement

The maximum amounts of reinforcement for beams, slabs, columns and walls included in Codes of Practice are there from practical considerations of construction and the ability to place concrete. There is no reason to change the requirements when using non-metallic reinforcement.

D.3.11 Bond, anchorage, laps and joints

Review of test work

Many studies have been carried out on the bond of FRP bars embedded in concrete. However, the results obtained will be specific to the particular bar being tested.

Nanni *et al*(20) and Chaallal and Benmokrane(55) have concluded that the ultimate bond strength is independent of the grade of concrete. British and other Codes assume that, for steel, the bond strength is proportional to the square root of the concrete strength.

Ehsani *et al*(56) considered the use of bars with a 90° bend as anchors. As expected, with small radius bends premature failure occurred at the start of the bend. With larger radii, full anchorage could be obtained but only with a significant straight portion before the bend, presumably reflecting the reduced strength at the bend itself, which will be a function of the method of manufacture of the particular bar used.

Tests by Kanakubo *et al*(57), using a bond specimen with a large amount of confining reinforcement, showed no significant difference between the ultimate bond stress for top cast and bottom cast bars. (In the British Codes for steel reinforcement the bond strength of top cast bars, which are taken to be in areas of poor compaction, is taken as about 70% of equivalent bottom cast bars, which are in areas of good compaction). This again suggests that failure is actually taking place in the resin and not in the concrete.

However, Chaallal and Benmokrane(55) showed that, on average, top cast bars in normal strength concrete developed about 80% of the bond of bottom cast bars. In the high strength concrete the figure was 85%. The Codes would suggest 70%. Hence the phenomena is present but to a lesser extent than for steel reinforcement.

Very limited work has been carried out on the behaviour of laps. This is obviously an area that requires further study.

Modifications to Codes

As discussed in Section D.2.5 the ultimate bond stress should be determined by testing. For design purposes this should be divided by an appropriate partial safety factor to give the design bond stress, which should be assumed to be independent of the concrete strength. Anchorage and lap lengths should be determined on the basis of the design bond stress.

Anchorage should generally be by means of straight lengths of bar. Bends or hooks should only be used if their strengths have been proved by testing. Similarly the strengths of any connectors used for joining bars or any form of mechanical anchorage should be proved by testing. The strength of any connector or anchorage should be greater than that of the bar, taking into account the long-term properties of the bars, the connector or anchorage, and any resin used in the assembly.

D.4 Design and detailing: Precast and composite (concrete–concrete) construction

D.4.1 General

The only significant changes required will be to the clauses dealing with the design of joints between precast and *in situ* concrete.

D.4.2 Horizontal, or longitudinal, shear

Introduction

Both BS 5400: Part 4 and BS 8110 assume that the longitudinal capacity of joints depends on the amount of reinforcement passing through the interface. For steel, the reinforcement is assumed to be at its design tensile capacity. It provides a clamping force across the interface which resists shear by means of friction. The dowel effect of the reinforcement is not specifically taken into account.

In BS 5400: Part 4 the capacity is the sum of the frictional force developed by the steel and a component from the concrete itself. In BS 8110, for joints with more than a minimum of 0.15% of steel passing through, only the frictional force developed by the reinforcement is considered.

Review of test work

Only very limited experimental work has been carried out on non-ferrous reinforcement under this type of loading. Some tests have been carried out by Rich(58) on initially uncracked specimens with glass FRP rods or aramid FRP hoops passing through the interface.

Amendments to Code requirements

The Codes use the characteristic tensile strength of the steel to determine the resistance. In BS 5400: Part 4 the total longitudinal shear capacity is based on both the amount of steel and also on the shear resistance of the concrete itself. In line with the recommendations for the vertical shear resistance of beams (Section D.3.2), the strain in the non-ferrous reinforcement should be limited to 0.0025.

BS 8110 gives values for the shear strength of interfaces with nominal percentages of reinforcement passing through. For steel 'nominal' is taken as 0.15%. For FRP this should be increased inversely in proportion to the stiffness, i.e. 0.15 becomes:

$$0.15(200/E_T)$$

For amounts of reinforcement above this level, the strain in the FRP should again be limited to 0.0025. The resulting shear stress for design purposes should be the greater of the resistance provided by the reinforcement alone and the resistance of the concrete with nominal reinforcement passing through.

Comparisons with data

The results from Rich[58] have been compared with the BS 5400: Part 4 approach. Average values of measured divided by predicted capacity were 1.73, suggesting that the proposed amendment is safe.

D.4.3 Handling stresses

For precast units, a significant proportion of the reinforcement may be required solely to resist stresses due to handling, storage, transport and erection. The required amounts of reinforcement may be determined using the partial safety factors for short-term loading given in Section D.2.2. The same factors may be used when designing reinforcement solely to resist early thermal movements and shrinkage stresses.

D.4.4 Design by testing

For small precast units, design by testing rather than analysis may be appropriate Testing representative prototype units will provide information on both the service load behaviour, the ultimate load capacity and the mode of failure. Safety factors can then be applied to the load capacity, appropriate to the mode of failure and the critical material, i.e. dependent on whether failure is in the concrete or the reinforcement.

D.4.5 Reinforcement detailing

As methods of composite fabrication are developed it should be possible to form integral two- or three-dimensional reinforcement cages. These would eliminate many of the bond and anchorage problems that may be encountered with traditional loose bars. The cages would be simpler and more efficient, requiring less material.

It is probable that they would be less susceptible to degradation of the resin by alkali attack or other damage which will affect the surface first. The reinforcement cages could incorporate spacers to ensure accurate location within the mould.

D.4.6 Construction temperatures

When using steam curing, or other means of heating the concrete, care must be taken to ensure that the raised temperature does not adversely affect the resin of the reinforcement.

D.5 Summary and conclusions

The aim throughout Part D has been to justify the proposed design approaches, which would be applicable to all types of FRP reinforcement material. The design methods predict behaviours which are generally in good agreement with published data, though this is very limited in some areas. In due course the design methods will probably have to be modified to reflect more accurately the behaviour of the various different materials. However, at present they should lead to safe, though perhaps not economic, structures.

There are a number of areas in which considerable further work is required, leading to improvements in design. These are outlined in Part E.

PART E

Future work

PART E Future work

E.1 Introduction

The recommended changes to the design Codes given in the preceding parts have been developed on the basis of the best information currently available. As indicated in Part D, there are a number of areas in which little, if any, test work has been carried out to date. Hence the guidance has had to be made on the basis of judgment, with appropriately large factors of safety. Key areas for research are identified below.

E.2 Structural research

Areas in which further structural testing and the development of appropriate design methods are required are listed below. They have been broadly ranked in three groups, in order of importance, though the order in which the topics are presented within each group is not significant.

High priority

- behaviour of columns
- punching of slabs
- behaviour of structures in fire
- shear of beams, including behaviour of shear links
- effect of load reversals on bar capacity
- methods for providing confinement in the compression zone of beams
- development of mechanical/adhesive anchorage and connection systems

Medium priority

- connections between precast elements
- behaviour of continuous elements and frames
- rotation capacity and redistribution
- bundle theory (behaviour of multiple bars in a member)
- fatigue
- laps and anchorages
- torsion
- impact

Low priority

- early thermal crack control

Some of these gaps will be filled naturally as experience of using FRP reinforcement increases and as more demonstration structures are built.

E.3 Improved design approaches

The current proposals are all modifications of the approaches used for structures reinforced with traditional steel. There is considerable scope for developing novel design approaches that are more suited to the new materials, which should result in more economic structures. An example is the design of columns. The proposals suggest that FRP bars in compression will not contribute to the loadcarrying capacity; significant improvements in the loadcarrying capacity can be achieved by using the FRP externally to provide containment, an approach not currently covered by the Code.

E.4 Materials development

The key areas for materials development are:

- thermo-plastic materials, for forming bent shapes, such as hooks and shear links.
- resins with improved durability
  - resistance to alkalis in concrete
  - resistance to common admixtures
  - resistance to normal site chemicals (e.g. mould oil)
- resins with improved thermal properties
- higher modulus fibres

E.5 Manufacture and manufacturing processes

Unlike traditional materials, such as steel reinforcing bars, FRP materials are not currently manufactured to agreed Standards. As FRP reinforcement moves out of the development stage into full-scale commercial production, such Standards are urgently required. They should include the following:

- agreement on manufacturing standards
- agreement on acceptance criteria for durability
- agreement on tests for determining mechanical properties, including strength, stiffness and bond.

In addition manufacturing processes need to be developed for forming complex shapes, such as integral reinforcement cages.

References

1. Nanni, A., Dolan, C. W. (Eds.): (1993) *Fibre-Reinforced-Plastic Reinforcement for Concrete Structures,* Detroit, American Concrete Institute, SP-138, pp. 977.

2. Taerwe, L. (Ed.): (1995) *Non-Metallic (FRP) Reinforcement for Concrete Structures,* E. & F. N. Spon, London, pp. 714.

3. El-Badry, M. M. (Ed.): (1996) *Advanced Composite Materials in Bridges and Structures,* Canadian Society for Civil Engineering, Montreal, pp. 1027.

4. Japan Concrete Institute: (1997) *Non-Metallic (FRP) reinforcement for Concrete Structures,* Sapporo, Japan, pp. 728 and 813 (2 volumes).

5. University of Sherbrooke: (1998) *First International Conference on Durability of Composites for Construction,* Sherbrooke, Canada.

6. Canadian Standards Association: (1996) *Canadian Highways Bridge Design Code,* Section 16, *Fibre Reinforced Structures* (Final Draft).

7. Wolff, R., and Meisseler, H. J.: (1993) *Glass fibre prestressing system, Alternative materials for the reinforcement and prestressing of concrete* (Ed. J. L. Clarke), Blackie Academic & Professional, Glasgow, pp. 127-152.

8. Tsuji, Y., Kanda, M. and Tamura, T.: (1993) 'Applications of FRP materials to prestressed concrete bridges and other structures in Japan', *PCI Journal,* July-August, p. 50.

9. Noritake, K. *et al*: (1993) 'Practical applications of aramid FRP rods to prestressed concrete structures', *Fibre reinforced plastic reinforcement for concrete structures* (Eds. A. Nanni and C. W. Dolan) SP 138, American Concrete Institute, p. 853.

10. Zoch, P. *et al:* (1991) *Carbon fibre composite cables: a new class of prestressing members,* 70th Annual Convention of the Transportation Research Board, Washington DC.

11. Casas, J. R., and Aparicio, A. C.: (1992) 'A full-scale experiment on a prestressed concrete structure with high strength fibres; the North ring-road in Barcelona', *FIP-XI International Congress,* Hamburg, p T15.

12. Clarke, J. L., Dill, M. J. and O'Regan, P.: (1998) 'Site testing and monitoring of Fidgett Footbridge', *Structural assessment – The role of large and full-scale testing,* (Eds. K. S. Virdi, F. K. Garas, J. L. Clarke and G. S. T. Armer) E. & F. N. Spon, London, pp. 29-35.

13. Grostad, T., Haugerud, S. A., Mathisen, L. L. and Clarke, J. L.: (1997) 'Case studies within Eurocrete – Fender in Qatar and bridge in Norway', *Non-Metallic (FRP) reinforcement for Concrete Structures,* Japan Concrete Institute, Sapporo, Japan, Vol. 1, pp. 657-664.

14. Iyer, S. L.: (1993) 'Advanced composite demonstration bridge deck', *Fibre reinforced plastic reinforcement for concrete structures* (Eds. A. Nanni and C. W. Dolan) SP 138, American Concrete Institute, Detroit, p. 831.

15. Anon: (1993) 'Carbon-fibre strands prestress Calgary span', *Engineering News Record,* 18 October 1993, p. 21.

16. *Guidelines for structural design of FRP reinforced concrete building structures* (1995) (Draft), Japanese Ministry of Construction.

17. *Recommendations for Design and Construction of Concrete Structures using Continuous Fiber Reinforcing Materials,* (1997) *Concrete Engineering* Series 23, Japan Society of Civil Engineers, Tokyo.

18. Bakht, B., *et al*: (1996) 'Design provisions for fibre reinforced structures in the Canadian highway bridge design Code', *Advanced Composite Materials in Bridges and Structures,* (Ed. M. M. El-Badry), Canadian Society for Civil Engineering, Montreal, pp. 391-406.

19. ACI Committee 440: (1996) *State-of-the-Art Report on Fiber Reinforced Plastic Reinforcement for Concrete Structures,* Report ACI 440R-96, American Concrete Institute, Detroit.

20. Nanni, A., Al-Zaharani, M. M., Al-Dulaijen, S. V., Bakis, C. E. and Boothby, T. E.: (1995) 'Bond of FRP reinforcement to concrete – Experimental results', *Non-Metallic (FRP) Reinforcement for Concrete Structures,* (Ed. L. Taerwe), E. & F. N. Spon, London, pp. 135-45.

21. Kumahara, S., Masuda, Y., Tanano, H. and Shimizu, D.: (1993) 'Tensile strength of continuous fibre bar under high temperature', *Fibre-Reinforced – Plastic Reinforcement for Concrete Structures,* (Eds. A. Nanni and C. W. Dolan), SP-138, American Concrete Institute, Detroit pp. 731-42.

22. Chaallal, O. and Benmokrane, B.: (1993) 'Physical and mechanical performance of an innovative glass-fibre-reinforced plastic rod for concrete and grouted anchorages', *Canadian Journal of Civil Engineering,* Vol. 20, No. 2, pp. 254-68.

23. Tanano, H., Masuda, Y., Kage, T., Fukuyama, H., Nishida, I. and Hashimoto, T.: (1995) 'Fire resistance of continuous fibre reinforced concrete', *Non-metallic (FRP) Reinforcement for Concrete Structures,* (Ed. L. Taerwe) E. & F. N. Spon, London, pp. 368-75.

24. Fujisaki, T., Nakatsuji, T. and Sugita, M.: (1993) 'Research and development of grid shaped FRP reinforcement', *Fibre-Reinforced-Plastic Reinforcement for Concrete Structures,* (Eds A. Nanni and C. W. Dolan), SP-138, American Concrete Institute, Detroit, pp. 177-92.

25. Tezuka, M., Ochiai, M., Tottori, S. and Sato, R.: (1995) 'Experimental study on moment redistribution of continuous beams reinforced or pretensioned with fibre reinforced plastic', *Non-Metallic (FRP) Reinforcement for Concrete Structures,* (Ed. L Taerwe) E. & F. N. Spon, London, pp. 387-94.

26. Kawaguchi, N.: (1993) 'Ultimate strength and deformation characteristics of concrete members reinforced with AFRP rods under combined axial tension or compression and bending', *Fiber-Reinforced-Plastic Reinforcement for Concrete Structures,* (Eds. A. Nanni and C. W. Dolan), SP-138, American Concrete Institute, Detroit, pp. 671-83.

27. Faza, S. S. and GangaRao, H. V. S.: (1992) 'Bending and bond behaviour of concrete beams reinforced with fiber reinforced plastic rebars', *Transportation Research Record,* No 1290, pp. 185-93.

28. Faza, S. S. and GangaRao, H. V. S.: (1993) 'Theoretical and experimental correlation of behaviour of concrete beams reinforced with fiber reinforced plastic rebars', *Fiber-Reinforced-Plastic Reinforcement for Concrete Structures,* (Eds. A. Nanni and C. W. Dolan), SP-138, American Concrete Institute, Detroit, pp. 599-614.

29. Nawy, E. G., Neuwerth, G. E. and Phillips, C. J.: (1971) 'Behaviour of fibre glass reinforced concrete beams', *Journal of the Structural Division, Proc. ASCE,* Vol. 97, pp. 2203-15.

30. Nawy, E. G. and Neuwerth, G. E.: (1977) 'Fiberglass reinforced concrete slabs and beams', *Journal of the Structural Division, Proc. ASCE,* Vol. 103 No. ST2, pp. 421-40.

31. Masmoudi, R., Chaallal, O. and Benmokrane, B.: (1994) 'An experimental investigation of the flexural behaviour of concrete beams reinforced with GFRP rebar', *Corrosion and Corrosion Protection of Steel in Concrete,* (Ed. N. Swamy), University of Sheffield, pp. 1340-51.

32. Al Sayed, S. M., Almusallam, T. H., Al-Salloum, Y. D. and Amjad, M. D.: (1995) 'Flexural behaviour of concrete elements reinforced by GFRP bars', *Non-metallic (FRP) Reinforcement for Concrete Structures,* (Ed. L. Taerwe), E. & F. N. Spon, London, pp. 219-26.

33. Buyle-Bodin, F., Benhouna, M. and Convain, M.: (1995) 'Flexural behaviour of JITEC-FRP reinforced beams', *Non-metallic (FRP) Reinforcement for Concrete Structures,* (Ed. L. Taerwe), E. & F. N. Spon, London, pp. 235-42.

34. Benmokrane, B., Chaallal, O and Masmoudi, R.: (1995) 'Glass fibre reinforced plastic (GFRP) rebars for concrete structures', *Construction and Building Materials,* Vol. 9, No. 6, pp. 346-53.

35. Nakano, K., Matsuzaki, Y., Fukuyama, H. and Teshigawara, M.: (1993) 'Flexural performance of concrete beams reinforced with continuous fibre bars', *Fiber-Reinforced-Plastic*

Reinforcement for Concrete Structures, (Eds. A. Nanni and C.W. Dolan), SP-138, American Concrete Institute, Detroit, pp. 743-66.

36. Maruyama, K. and Zhao, W. J.: (1994) 'Flexural and shear behaviour of concrete beams reinforced with FRP rods', *Corrosion and Corrosion Protection of Steel in Concrete,* (Ed. N. Swamy), University of Sheffield, pp. 1330-39.

37. Matthys, S. and Taerwe, L.: (1995) 'Loading tests on concrete slabs reinforced with FRP grids', *Non-metallic (FRP) Reinforcement for Concrete Structures,* (Ed. L. Taerwe), E. & F. N. Spon, London, pp. 287-97.

38. Nagasaka, T., Fukuyama, H. and Tanigaki, M.: (1993) 'Shear performance of concrete beams reinforced with FRP stirrups', *Fiber Reinforced-Plastic Reinforcement for Concrete Structures,* (Eds. A. Nanni and C. W. Dolan), SP-138, American Concrete Institute, Detroit, pp. 789-811.

39. Zhao, W., Maruyama, K. and Suzuki, H.; (1995) 'Shear behaviour of concrete beams reinforced by FRP rods as longitudinal and shear reinforcement', *Non-metallic (FRP) Reinforcement for Concrete Structures,* (Ed. L. Taerwe), E. & F. N. Spon, London, pp. 352-9.

40. Maruyama, K. and Zhao, W.: (1996) 'Size effect in shear behaviour of FRP reinforced concrete beams', *Advanced Composite Materials in Bridges and Structures,* (Ed. M. M. El-Badry), Canadian Society for Civil Engineering, Montreal, pp. 227-34.

41. Yonekura, D., Tazawa, E., Miyazawa, S. and Zhou, P.: (1995) 'Influence of torsion on mechanical characteristics of PC beams using FRP rods', *Non-metallic (FRP) Reinforcement for Concrete Structures,* (Ed. L. Taerwe), E. & F. N. Spon, London, pp. 448-55.

42. Ahmad, S. H., Zia, P., Yu, T. J. and Xie, Y.: (1994) 'Punching shear tests of slabs reinforced with 3-D carbon fibre fabric', *ACI Concrete International,* June, pp. 36-41.

43. Matthys, S. and Taerwe, L.: (1996) 'Punching shear behaviour of concrete slabs reinforced with FRP grids', *Advanced Composite Materials in Bridges and Structures,* (Ed. M. M. El-Badry), Canadian Society for Civil Engineering, Montreal, pp. 261-9.

44. Kobayashi, K. and Fujisaki, T.: (1995) 'Compressive behaviour of FRP reinforcement in non-prestressed concrete members', *Non-metallic (FRP) Reinforcement for Concrete Structures,* (Ed. L. Taerwe), E. & F. N. Spon, London, pp. 267-74.

45. Ahmad, S. H., Khaloo, D. R. and Irshard, D.: (1991) 'Behaviour of concrete spirally confined by fibreglass filaments', *Magazine of Concrete Research,* Vol. 43, No. 156, pp. 143-8.

46. Harmon, T. G. and Slattery, K. T.: (1992) 'Advanced composite confinement of concrete', *Advanced Composite Materials in Bridges and Structures*, (Eds. K. W. Neale and P. Labossiere), Canadian Society for Civil Engineering, Montreal, pp. 298-306.

47. Kubota, T. and Murakami, M.: (1991) 'Experimental study on stress-strain relationship of confined concrete reinforced by FRP bars', *Proceedings of Japan Concrete Institute,* Vol. 13 No. 2, pp. 843-8.

48. Watanabe, K., Sonobe, Y. and Okamoto, T.: (1991) 'Centrally loaded compression test of concrete columns reinforced with braided aramid fibre rods', *Proceedings of Japan Concrete Institute,* Vol. 13 No.2, pp. 801-6.

49. Hosoi, R., Mutsuyoshi, H., Tanaguchi, H. and Kita, T.: (1991) 'Compressive properties of concrete members reinforced with FRP as transverse reinforcement', *Proceedings of Japan Concrete Institute,* Vol. 14, No.2, pp. 637-42.

50. Fardis, M. N. and Khalili, H. H.: (1982) 'FRP-encased concrete as a structural material', *Magazine of Concrete Research,* Vol. 34, No.121, pp. 191-201.

51. Fardis, M. N. and Khalili, H.: (1981) 'Concrete encased in fibre-glass reinforced plastic', *Journal of the American Concrete Institute,* November-December, pp. 440-6.

52. Mogahadam, M. and Sentler, L.: (1995) 'GFRP as crack control reinforcement', *Non-Metallic (FRP) Reinforcement for Concrete Structures,* (Ed. L. Taerwe), E. & F. N. Spon, pp. 243-50.

53. Beeby, A. W.: (1978) 'Cracking and corrosion', *Concrete in the Oceans Technical Report*

No.1, CIRIA/UEG, Cement and Concrete Association, Department of Energy, Slough.

54. Kakizawa, T., Ohno, S. and Yanezawa, T.: (1993) 'Flexural behaviour and energy absorption of carbon FRP reinforced concrete beams', *Fiber-Reinforced-Plastic Reinforcement for Concrete Structures,* (Eds. A. Nanni and C. W. Dolan), SP-138, American Concrete Institute, Detroit, Detroit, pp. 585-98.

55. Chaalla, O. and Benmokrane, B.: (1993) 'Pullout and bond of glass-fibre rods embedded in concrete and cement grout', *RILEM Materials and Structures,* No. 26, pp. 167-75.

56. Ehsani, M. R., Saadatmanesh, H. and Tao, O.: (1995) 'Bond of hooked glass fibre reinforced plastic (GFRP) reinforcing bars to concrete', *ACI Materials Journal,* July-August, pp. 391-400.

57. Kanakubo, T., Yonemaru, K., Fukuyama, H., Fujisawa, M. and Sonabe, Y.: (1993) 'Bond performance of concrete members reinforced with FRP bars', *Fibre-Reinforced-Plastic Reinforcement for Concrete Structures,* (Eds. A. Nanni and C. W. Dolan), SP-138, American Concrete Institute, Detroit, pp. 767-88.

58. Rich, M.: (1996) 'Shear in concrete reinforced with fibre reinforced plastics', Fourth-year Project Report, University of Cambridge Engineering Department.

Interim guidance on the design of reinforced concrete structures using fibre composite reinforcement

This Guide is intended for use by engineers familiar with the design of conventionally reinforced concrete structures in accordance with the current design Codes but who have little, or no, experience of the use of FRP rods or grids as embedded reinforcement.

Fibre reinforced plastic (FRP) rods or grids, consist of high modulus continuous fibres, such as glass, carbon or aramid, combined with an appropriate resin. They have been developed for use as embedded reinforcement, particularly for concrete structures in highly aggressive environments, such as marine structures or bridges subjected to deicing salts. In these situations, the traditional approach of providing protection to the steel by the provision of concrete of appropriate quality may not be sufficient and corrosion may take place. The improved durability of correctly specified FRP materials should lead to improved performance.

Though various types of FRP reinforcement are now commercially available and have been used for a number of demonstration structures worldwide, design is not currently covered by appropriate Standards. This document gives interim guidance on the design of concrete structures reinforced with FRP. The guidance given is not specific to any particular material. The suggested modifications have been based on published information, available in the public domain. While the Guide contains suggested changes to the British design Codes, the approaches adopted are in line with similar recommendations being developed elsewhere in the world, principally in Japan, the USA and Canada. In the absence of an authoritative Code, this document is intended to provide safe design guidance.

It does not cover the use of FRP as prestressing tendons, the use of FRP material applied as reinforcement to the outer surface of the structure, or the use of short chopped fibres incorporated into the concrete during mixing as these were outside the original terms of reference.