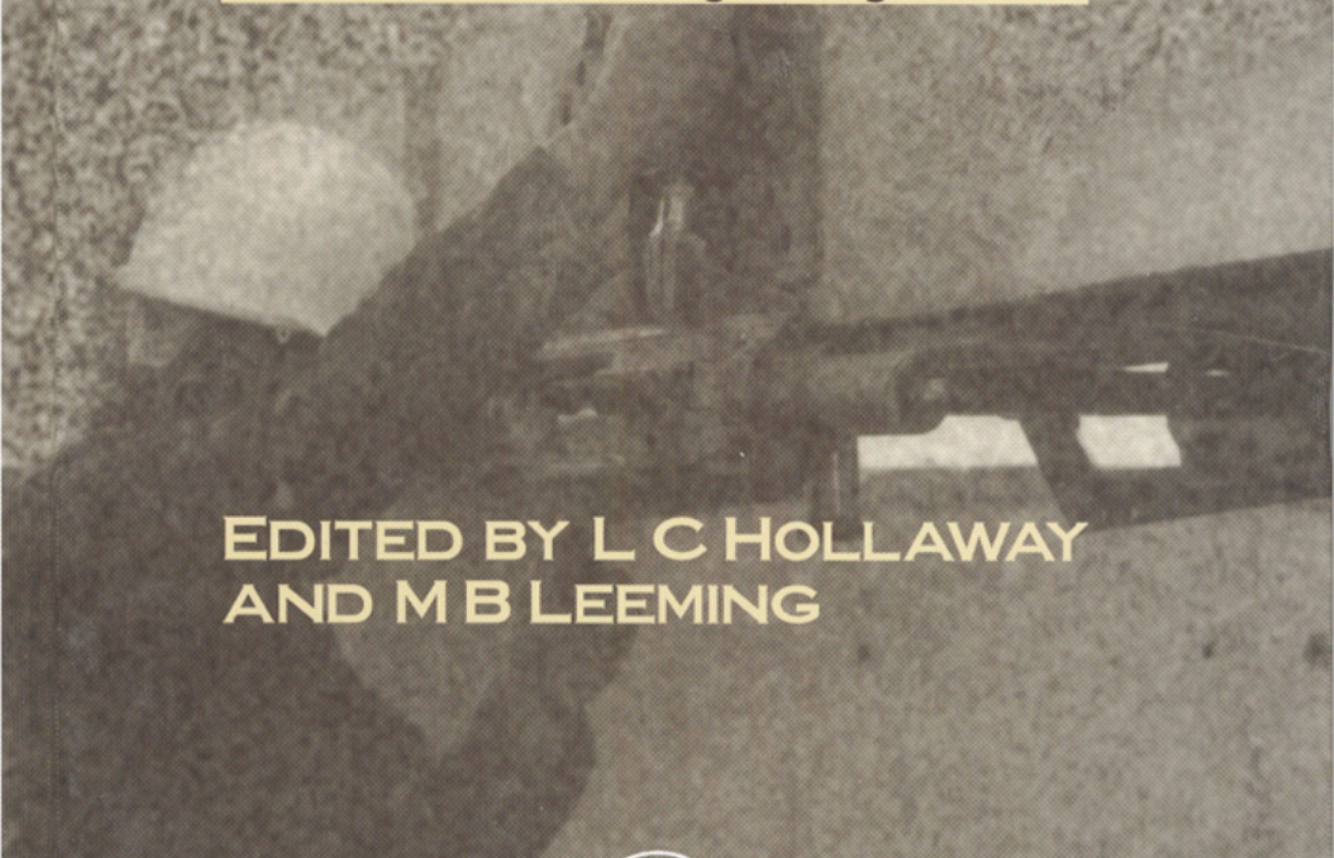
STRENGTHENING OF REINFORCED CONCRETE STRUCTURES

using externally-bonded FRP composites in structural and civil engineering



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Preface

Structures manufactured from engineering materials such as reinforced and prestressed concrete, steel and cast iron, although durable, do have a finite life. Structures with significant problems are those exposed to marine environments, de-icing salts on highways, aggressive industrial environments and to a lesser extent those which are exposed to normal weather conditions. From an economic point of view it is generally more realistic to repair, if possible, rather than to demolish and rebuild the structures.

The *in situ* rehabilitation or upgrading of reinforced concrete members using bonded steel plates has been proven in the field to be an effective, convenient and economic method of improving structural performance. However, disadvantages inherent in the use of steel have stimulated research into the possibility of using fibre reinforced polymer (FRP) materials in its place, providing a non-corrosive, more versatile strengthening system; it can also be used for prestressed concrete.

The construction industry is currently using polymer materials for the maintenance of structures and is showing great interest in the utilisation of FRP materials to maintain structural integrity or to upgrade structural systems. Advanced composite structural materials may have aligned continuous fibres and/or aligned angle plies encapsulated in a polymer to form plates, rods, tubes and structural profiles; the fibres would be either carbon, aramid or glass. When used as a structural component to repair or strengthen a system the polymer composite may be either unstressed or pretensioned at the time of bonding.

If two dissimilar materials which have composite action are to be used structurally, it is necessary for designers to have a thorough understanding of the mechanical and inservice material properties of the components, the methods of joining, the composite action and failure mechanisms and the overall structural analysis of these systems.

The book presents a detailed study of the flexural strengthening of reinforced and prestressed concrete members using fibre reinforced polymer composite plates encompassing both short term and long term performance through experimental testing at model and full scale and theoretical and numerical considerations. In addition, in Chapter 2 entitled ‘Review of materials and techniques for plate bonding’, the book discusses previous investigative and site work which has been undertaken to strengthen reinforced and prestressed concrete beams utilising steel bonded plates and discusses the advantages and disadvantages of using the two different plate materials, namely steel and composites. Shear plate bonding is also discussed in this chapter.

Chapter 11 of the book contains case histories of construction members which have been upgraded or strengthened by the utilisation of carbon fibre/polymer matrix composite materials bonded to the structural unit. The case histories also include upgrading reinforced and prestressed concrete and timber systems.

The data for this book has been derived to a large extent from material developed or provided by the consortium which studied and analysed the technology of plate bonding to upgrade structural units using carbon fibre/polymer composite materials. The research and trial tests have been undertaken as part of the ROBUST (St*r*engthening *o*f *B*ridges *U*sing Polymeric Compo*s*ite Ma*t*erials) project, one of several ventures in the UK Government’s DTI-LINK Structural Composites Programme. The editors wish to stress that their intention was to make available the very large amount of research information which resulted from this study. The ongoing work of using this information to generate/formulate design specifications must pass to practising engineers, and was not considered to be part of the purpose of the book. Such specifications have been and are being developed within the commercial engineering sector which continues to apply the technique in an ever-widening range of applications.

The industrial members in the consortium were Mouchel Consulting Ltd (lead partner), the Royal Military College of Science and Concrete Repairs Ltd (subcontractors to lead partner), Oxfordshire County Council, Balvac Whitley Moran Ltd, Techbuild Composites Ltd (now Fibreforce Reinforced Composites Ltd), Vetrotex (UK) Ltd, James Quinn Associates Ltd and Sika Ltd. The academic partners were the University of Surrey and Oxford Brookes University who were both financially supported for the project by the Engineering and Physical Sciences Research Council (EPSRC) within the DTI-LINK scheme. In addition, further research investigations were undertaken at Oxford Brookes University, financially supported by that University and at the University of Surrey, financially supported by the EPSRC under an allied investigation, and also research investigations financially supported by the University.

The writing of the various chapters in the book has been the responsibility of the named authors of those chapters and they have obtained their information from many sources but have relied heavily upon the coopera-

tion of the research assistants working on the project and the industrial members of the ROBUST consortium; this invaluable help is gratefully acknowledged. The members of the consortium and the Universities associated with the investigations during the ROBUST period are as follows:

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	– Mr M Leeming
	– Dr S Luke
	– Dr V Peshkam
Royal Military College of Science	– Professor G Mays
	– Mr R Barnes
Balvac Whitley Moran Ltd	– Mr P Dennis
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1
Role of bonded fibre-reinforced composites in strengthening of structures

J J DARBY

1.1 Introduction

The authors contributing to this volume have been immersed in the development of advanced composite materials for strengthening structures for a number of years. Yet, in 1998, this can still be described as a new technique, with the total number of applications worldwide measured at most in hundreds. From this cautious beginning, the author believes that a rapid expansion in usage will take place as the benefits are more widely realised. All clients and designers seek solutions that are durable and cost effective, exactly those requirements which fibre reinforced composite strengthening systems can be designed to meet. However, clients must also gain trust in new techniques before they will be willing to adopt them. That trust must be firmly based on an understanding of material behaviour, the design process and the risks of implementation. It is hoped that this book will assist in that process, particularly by disseminating some of the knowledge that has been gained during the 'ROBUST' research project. Inevitably it will take time to foster a wide appreciation of these new materials amongst the construction community. It will not be assumed that readers have any previous experience of composite materials. All aspects of composite plate bonding are covered in some detail in individual chapters, but a more general introduction to the techniques is appropriate first. This will take the form of a definition of terms.

1.2 What is 'strengthening with bonded fibre reinforced polymer composite plates'?

- *Fibre reinforced polymer (FRP) composites*: FRP composites comprise fibres of high tensile strength within a polymer matrix. The fibres are generally carbon or glass, in a matrix such as vinylester or epoxy. These materials are preformed to form plates under factory conditions, generally by the pultrusion process. For experimentation, plates may be manufactured in smaller quantities from pre-impregnated fibre mats.

- *Bonded plates*: the preformed plates are fixed externally to the structure with adhesives, usually of epoxy, to promote composite structural action, although additional mechanical fixings may be used if deemed necessary by particular circumstances.
- *Structural strengthening*: the load bearing capacity of structures may be increased or restored, either locally or overall. Plates may be installed unstressed, or stressed on site effectively to prestress the structure. Most experimental work has been undertaken by applying composites to concrete, but timber, stone, steel or cast iron may also be strengthened.

1.3 The market for strengthening

Modern civilisation relies upon the continuing performance of a wide variety of structures, ranging from industrial buildings and power stations to bridges. Although these structures may appear very different, their managers are likely to recognise a number of common features:

- structural deterioration perhaps increased by environmental factors
- changes in use or imposed loading
- the need to minimise closure or disruption during repairs
- the need to extend useful life whilst minimising capital outlay
- more stringent financial disciplines requiring the evaluation of the whole life cost of solutions.

The number of structures in the world continues to increase, as does their average age. The need for increased maintenance is inevitable. Complete replacement is likely to become an increasing financial burden and is certainly a waste of natural resources if upgrading is a viable alternative. The way in which FRP composite plate bonding can help will be illustrated by considering two particular structure types, buildings and bridges.

- *Buildings*: industrial buildings may be adapted for new uses, increasing floor or slab loading. Externally bonded plates will increase capacity with negligible increase in construction depth. Structural alterations may require removal of columns or holes to be cut through slabs for purposes such as new lifts or services. External reinforcement in these circumstances may be the only alternative to partial demolition and replacement, with all the disruption to production which that entails.
- *Bridges*: loads on bridges are increasing, due to increases in the permitted vehicle weights as well as the volume of traffic. At the same time

> material deterioration is becoming more evident, particularly that due to reinforcement corrosion induced by contamination with de-icing salts. For this reason, a large scale assessment programme is underway in the UK to examine the load capacity of all bridges of uncertain strength. This has already revealed the need for extensive strengthening. FRP composite plate bonding will offer the best solution for many of these structures, particularly where short construction periods may be a key factor.

Cost is probably the most influential factor when assessing the merits of alternative methods. Detailed costing would be out of place in a book of this kind and would date quickly. This is particularly the case for new techniques, as prices can be expected to fall as more material suppliers and contractors enter the growing market. However, the case for bonded fibre reinforced composites can best be illustrated by the fact that these materials are already winning competitive tenders against alternative solutions.

1.4 Strengthening techniques

The art of designing strengthening schemes with FRP composite plate bonding is at an early stage. Detailed guidance on what reinforcement and detailing should be used in every particular circumstance cannot be provided. Economical solutions depend upon an understanding of the materials and experience of what they can safely achieve. There are many options open to the skilful designer. Just as reinforced concrete may be designed to behave differently according to the mix of concrete and reinforcement, so the composite plates may use different reinforcement materials, in different proportions, and within different matrix materials. These plates may then be of different lengths, and multiple layers may be used. These may be fixed at any required geometry on the surface of the structure. The adhesives and surface preparation may vary. The plates may be stressed or unstressed and the ends mechanically anchored or bonded by adhesive only.

This wide range of options must be seen as an advantage and as an opportunity for the knowledgeable designer to tailor the strengthening scheme to the needs of the particular structure. But there is also a potential danger arising from application by designers without experience. It is difficult at the present stage of composite plate bonding to write a specification that covers all potential situations. Reinforced concrete specifications are still developing a century after the initial application of the material was concentrated in the hands of a number of specialist exponents. Fortunately, development of FRP composite plate bonding will be much faster. Analysis methods are available to speed up the process of understanding structural behaviour and we can build upon previous knowledge.

Projects such as ROBUST have provided a solid basis for designers to use FRP composite plate bonding to enhance flexural behaviour, using both stressed and unstressed plates. Much has also been learned about the need for fixing of the plates due to end peel effects. Shear strengthening, on the other hand, has been little researched to date.

The range of structural needs and deficiencies for which FRP composite plate bonding already offers an appropriate solution is very wide, as illustrated in Table 1.1.

1.5 Advantages and disadvantages of FRP composite plate bonding

All structural problems have more than one technical solution, and final selection will ultimately rest upon an economic evaluation of the alternatives. Enlightened clients will ensure that this evaluation includes an estimate of the total costs that will be incurred during the required service life, rather than selection of the scheme with the minimum initial cost. The total costs will include future maintenance, as well as all consequential costs such as loss of production or traffic delay costs.

The most obvious technical solution with which to compare FRP composite plate bonding is steel plate bonding, as many of the aspects are common to both. Such a comparison is made below. However, FRP composite plate bonding should not be thought of as simply an improved form of steel plate bonding. The new material offers such versatility that new solutions will become practicable, particularly those arising from prestressing of the plates

The potential advantages of FRP composite plate bonding are as follows:

- *Strength of plates*: FRP composite plates may be designed with components to meet a particular purpose and may comprise varying proportions of different fibres. The ultimate strength of the plates can thus be varied, but for strengthening schemes the ultimate strength of the plates is likely to be at least three times the ultimate strength of steel for the same cross-sectional area.
- *Weight of plates*: the density of FRP composite plates is only 20% of the density of steel. Thus composite plates may be less than 10% of the weight of steel of the same ultimate strength. Apart from transport costs, the biggest saving arising from this is during installation. Composite plates do not require extensive jacking and support systems to move and hold in place. The adhesives alone will support the plate until curing has taken place. In contrast, fixing of steel plates constitutes a significant proportion of the works costs.

Table 1.1 Applications for composite plate bonding

Structural need/deficiency	FRP composite plate bonding solution	Comments
Corrosion of reinforcement in reinforced concrete	Replacement of lost reinforcement by plates of equivalent effect	Damaged concrete must be replaced without impairing behaviour of plates
Inadequate flexural capacity of reinforced concrete	Design FRP composite plate bonding solution to add tensile elements	Extent of strengthening limited by capacity of concrete in compression. Plates anchored by bond or mechanically at their ends
Lost prestress due to corrosion in prestressed concrete	Replace prestress that has been lost with stressed composites	Need to ensure no overstress of concrete in the short term
Safety net to cover uncertain durability of prestressed concrete	Add plates, either stressed or unstressed, to ensure safety. Particularly appropriate if corrosion unlikely but possible	Method may be particularly appropriate with segmental construction. May be combined with a monitoring system
Inadequate stiffness or serviceability of cracked reinforced concrete structure	Add external prestress by means of a stressed composite plate	
Potential overstress due to required structural alteration	Analyse stresses due to alteration, and design composite reinforcement before removing load-bearing members	
Avoidance of sudden failure by cracking of cast iron	Addition of FRP composite plate bonding, either stressed or unstressed, to tensile face	
Increase in structural capacity of timber structures	Increase in stiffness and ultimate capacity by plate bonding	Particularly appropriate with historic structures
Enhancement of shear capacity	Enhanced by external bonding of stressed plates, or by web reinforcement	Web reinforcement techniques little researched

- *Transport of plates*: the weight of plates is so low that a 20 m long composite plate may be carried on site by a single man. Some plates may also be bent into a coil as small as 1.5 m diameter, and thus may be transported in a car or van without the need for lorries or subsequent craneage facilities. The flexibility of plates enables strengthening schemes to be completed within confined spaces.
- *Versatile design of systems*: steel plates are limited in length by their weight and handling difficulties. Welding *in situ* is not possible, because of damage to adhesives, and expensive fixing of lap plates is therefore required. In contrast, composite plates are of unlimited length, may be fixed in layers to suit strengthening requirements, and are so thin that fixing in two directions may be accommodated by varying the adhesive thickness.
- *Easy and reliable surface preparation*: steel plates require preparation by grit blasting, followed by careful protection until shortly before installation. In contrast, the ROBUST project has demonstrated that composite plates may be produced with a peel-ply protective layer that may be easily stripped off just before the adhesive is applied.
- *Reduced mechanical fixing*: composite plates are much thinner than steel plates of equivalent capacity. This reduces peeling effects at the ends of the plates and thus reduces the likelihood of a need for end fixing. The overall depth of the strengthening scheme is reduced, increasing head-room and improving appearance.
- *Durability of strengthening system*: there is the possibility of corrosion on the bonded face of steel plates, particularly if the concrete to which they are fixed is cracked or chloride contaminated. This could reduce the long term bond. Composite plates do not suffer from such deterioration.
- *Improved fire resistance*: composite plates are a low conductor of heat when compared with steel, thus reducing the effect fire has on the underlying adhesives. The composite itself chars rather than burns and the system thus remains effective for a much longer period than steel plate bonding.
- *Reduced risk of freeze/thaw damage*: there is theoretical risk of water becoming trapped behind plate systems, although this should not occur if they are properly installed. In practice, this has not been found to be a problem. However, if water did become trapped in this way, the insulating properties of the composite materials would reduce the risk of disruption of the concrete due to freeze/thaw. Loss of bond would also be evident by tapping the composite, but would be more difficult to detect with steel.
- *Maintenance of strengthening system*: steel plates will require maintenance painting and may incur traffic disruption and access costs as well

as the works costs. Composite plates will not require such maintenance, reducing the whole life cost of this system.

- *Reduced construction period*: many of the practical advantages described above combine to enable composite plates to be installed in greatly reduced time periods when compared with steel plates. As well as lower contract costs, the traffic delay costs are minimised. Installation from mobile platforms becomes possible and it may become practicable to confine work within such restraints as limited railway possessions or night-time working.
- *Ability to prestress*: the ability to prestress composites opens up a whole new range of applications for plate bonding. The plate bonding may be used to replace lost prestress and the shear capacity of sections will be increased by the longitudinal stresses induced. Formation of cracks will be inhibited and the serviceability of the structure enhanced. Strengthening of materials such as cast iron also becomes more practicable.

The potential disadvantages of FRP composite plate bonding are as follows:

- *Cost of plates*: fibre reinforced composite plates are more expensive than steel plates of the equivalent load capacity. However, the difference between the two materials is likely to be reduced as production volumes and competition between manufacturers increases. Comparison of total contract costs for alternative methods of strengthening will be based on labour and access costs as well as material costs. Open competition has already shown that FRP composite plate bonding is the most economic solution in virtually all tested cases, without taking into account additional advantages such as durability.
- *Mechanical damage*: FRP composite plates are more susceptible to damage than steel plates and could be damaged by a determined attack, such as with an axe. In vulnerable areas with public access, the risk may be removed by covering the plate bonding with a render coat. Fortunately, if damage should occur to exposed FRP composite plate, such as by a high load, repairs can be undertaken much more easily than with a steel plate. A steel plate may be dislodged, or bond broken over a large area, which would damage bolt fixings and necessitate complete removal and replacement. However, with FRP composite plate bonding the damage is more likely to be localised, as the plate is thinner and more flexible. With FRP composite, the plate may be cut out over the damaged length, and a new plate bonded over the top with an appropriate lap.

1.6 Client concerns when introducing new techniques

The construction industry is cautious over the introduction of any new technique, and this is understandable in view of problems with earlier materials. Even concrete is now known to deteriorate in many more ways than was widely appreciated just 30 years ago, and clients have become much more aware of the durability and maintenance implications of the solutions they adopt.

New techniques inevitably involve risk, because there cannot be a long track record of successful performance. Reassurance must be gained in other ways. Risk can still be managed and evaluated, a process which has been made much easier by advanced technology. We have the benefit of considerable understanding of design parameters and deterioration mechanisms. Finite element tools for analysing structural behaviour are more powerful than ever before. Thus we have the opportunity to analyse potential problems in advance and then to extend applications only within the boundaries of understanding.

The ROBUST project demonstrates what can be achieved. At a cost of about £1 million, a large number of strength, fatigue and durability tests have been performed. The expertise of designers, researchers, contractors and clients has been combined to investigate those risks that may result from applying the new technology. The volume of research work on composite plate bonding exceeds that which was undertaken before steel plate bonding was introduced and which was deemed sufficient to accept steel plate bonding in public works. Significantly, the replacement of steel by composite materials is a smaller technological change than was the first use of external reinforcement. The extensive research work on composites was fully justified and reflects the fact that clients in the present climate need reassurance, particularly where structural safety is involved.

1.7 Risk to clients when adopting FRP composite plate bonding

The decision by clients to use FRP composite plate bonding will rest upon balancing the clear advantages arising from use of the material, which have been indicated above, against the risk of potential problems. Those risks have been minimised by the work to date, and could be summarised as follows:

- *Durability of carbon fibre reinforcement*: carbon is a basic element occurring naturally in the environment. There is no known degradation mechanism.

- *Durability of glass fibre reinforcement*: E-glass is attacked by alkalinity from concrete if the fibre comes into direct contact with it, although this may be overcome to some extent by use of Z-glass; it should be mentioned, however, that the laminating polymer protects the fibre from direct contact with the concrete in the plate bonding technique. Stress corrosion may also be a problem with glass under continuous high states of stress. For these reasons, and also because the ultimate strength of glass is less than carbon, it is not advisable to use this material for main reinforcement without further research, despite its lower cost.
- *Durability of the composite matrix*: vinylester has been used in the chemical process industry for several decades as the preferred corrosion resistance barrier against a multiplicity of highly corrosive chemicals. Sunlight can cause some yellowing and surface degradation if no ultraviolet stabilisers are introduced into the formulation, but tensile and flexural strengths and moduli are not found to be significantly affected.
- *FRP composite plate performance*: carbon fibre reinforced composites have been used for 20 years in highly stressed areas in commercial and military aircraft and racing cars. The construction industry makes much lower demands on the performance of the material.
- *Adhesive performance*: many composite aircraft wings are glued to the main fuselage, which demonstrates the structural performance of composites and compatible adhesives. The adhesives used for composite plate bonding have been used in the construction industry for 20 years and have no known degradation mechanism. Performance with steel plate bonding has been accepted, and materials testing has shown the adhesives to be equally effective with composite materials. Contamination must be avoided, but peel ply reduces risk of poor workmanship. Moisture within the concrete has not been found to be a problem.
- *System performance*: the fatigue properties of FRP composite plate bonding have been found to be excellent, with fatigue failure not being initiated by any plate bonding component. Instead, tests have shown the fatigue life of reinforced concrete beams to be limited by the fatigue life of the embedded reinforcement. The addition of plate bonding will therefore only be significant if it increases the stress range of the reinforcement. Creep properties of carbon fibre are excellent and creep of the adhesive has not been found to be a problem.
- *System design*: much has been learned about the failure modes of beams reinforced by composite plate bonding and computer predictions are now remarkably accurate. Nevertheless, this is clearly a complicated area. Applications will inevitably extend to structures with features that differ in certain aspects from those tested. A period of monitored application is required before an all-embracing specification can be produced. The risks in the interim are minimal if design is undertaken by

those with an understanding of the behaviour of composite materials, who are thus able to recognise the need for caution and to invoke conservative features when the situation demands. The risk is higher if FRP composite plate bonding is designed by those without sufficient background knowledge, as deficiencies will not necessarily show up at the time of construction.

1.8 Conclusions

This introductory chapter can do no more than summarise the present position. It is within the later chapters that the progress made in researching FRP composite plate bonding can be illustrated in detail. Clients and designers will want to know whether the time is now right for them to utilise composite plate bonding techniques. There will undoubtedly be variation in the willingness to adopt new solutions, but it is to be hoped that the text within this volume will support the following conclusions:

- Fibre reinforced composite plate bonding offers significant advantages over steel plate bonding for the vast majority of strengthening applications.
- Fibre reinforced composite plate bonding is so versatile that the range of applications for which external reinforcing is appropriate will increase significantly.
- No construction or repair method involving structural analysis and deterioration mechanisms can be said to be completely understood, including all of those currently in everyday use. However, FRP composite plate bonding has been sufficiently researched to enable the techniques to be applied confidently on site, providing care is taken.
- The method of FRP composite plate bonding is here to stay and is already being actively marketed. The number of applications worldwide is set to grow very fast. The challenge is to ensure that these applications take full account of the current state of knowledge. The benefits must not be put at risk by inappropriate or badly detailed applications undertaken by the inexperienced.

2
Review of materials and techniques for plate bonding

L C HOLLAWAY AND M B LEEMING

2.1 Introduction

This chapter provides an introduction to the flexural rehabilitation or strengthening of reinforced concrete (RC), prestressed concrete and steel members using externally bonded steel or fibre reinforced polymer (FRP) composites plates by reviewing the most significant investigations reported in the literature. In addition, a section is devoted to the strengthening of RC members in shear utilising FRP plates. However, since the external plating and its application as a strengthening technique has only been made possible by the development of suitable adhesives, consideration is initially given to the types of adhesive which may be used for external plate bonding and their requirements for this application. After considering reported plate bonding studies, a brief review of surface preparation techniques applicable to FRP and concrete adherends is presented.

2.2 Structural adhesive bonding

Structural adhesives are generally accepted to be monomer composites which polymerise to give fairly stiff and strong adhesive uniting relatively rigid adherends to form a load-bearing joint (Shields, 1985). The feasibility of bonding concrete with epoxy resins was first demonstrated in the late 1940s (ACI, 1973), and the early development of structural adhesives is recorded by Fleming and King (1967). Since the early 1950s adhesives have become widely used in civil engineering (Mays, 1985). However, although the building and construction industries represent some of the largest users of adhesive materials, many applications are non-structural in the sense that the bonded assemblies are not used to transmit or sustain significant stresses (e.g. crack injection and sealing, skid-resistant layers, bonding new concrete to old). Truly structural application implies that the adhesive is used to provide a shear connection between similar or dissimilar materials, enabling the components being bonded to act as a composite structural unit. A comprehensive review of applications involving the use of adhesives in civil engineering is given by Hewlett and Shaw (1977), Tabor (1982) and Mays and Hutchinson (1992).

Assessment of an adhesive as a suitable product for structural use must take into account the design spectrum of loads, the strength and stiffness of the material under short term, sustained or cyclic loads and the effect on these properties of temperature, moisture and other environmental conditions during service (Mays, 1993). Concern regarding the durability properties of adhesive joints has meant that resistance to creep, fatigue and fracture are considered of greater importance than particularly high strength (Vardy and Hutchinson, 1986). Temperature is important at all stages in the use and performance of adhesives, affecting viscosity and therefore workability, usable life and contact time, rate of cure, degree of cross-linking and final cured performance (Tu and Kruger, 1996). Controlled conditions are therefore generally required during bonding. This applies equally during the surface treatment procedures if a durable system is to be achieved. Adhesives, which are workable and cure at ambient temperatures, have been used and are able to tolerate a certain amount of moisture without a marked reduction in performance. These must have adequate usable time under site conditions and a cure rate which does not hinder the construction programme. Workmanship under conditions prevalent on site is less conducive to quality control than in other industries, and thus ability to tolerate minor variations in proportioning and mixing, as well as imperfect surface treatment, is important. In addition, the products involved are more toxic, require more careful storage and, bulk for bulk, are considerably more expensive than traditional construction materials. Non-destructive test methods for assessing the integrity of bonded joints are now available for civil engineering applications.

Despite some drawbacks, structural adhesives have enormous potential in future construction applications, particularly where the combination of thick bondlines, ambient temperature curing and the need to unite dissimilar materials with a relatively high strength joint are important (Mays and Hutchinson, 1992).

2.2.1 Type of structural adhesives

The principal structural adhesives specifically formulated for use in the construction industry are epoxy and unsaturated polyester resin systems, both thermosetting polymers. The formulation of adhesives is considered in detail by Wake (1982), whilst Tabor (1978) offers guidance on the effective use of epoxy and polyester resins for civil engineering structures.

Two-part epoxies, first developed in the 1940s (Lee and Neville, 1967), consist of a resin, a hardener or cross-linking agent which causes polymerisation, and various additives such as fillers, tougheners or flexibilisers, all of which contribute to the physical and mechanical properties of the resulting adhesive. Formulations can be varied to allow curing at ambient temperature, the so-called cold cure epoxies, the most common hardeners for which are aliphatic polyamines, whose use results in hardened adhesives which are rigid and provide good resistance to chemicals, solvents and water (Mays and Hutchinson, 1992). Correct proportioning and thorough mixing are imperative when using epoxy resin systems. The rate of curing doubles as the temperature increases by 10 °C and halves as the temperature drops by 10 °C and many of the formulations stop curing altogether below a temperature of 5 °C. Fillers, generally inert materials such as sand or silica, may be used to reduce cost, creep and shrinkage, reduce exotherm and the coefficient of thermal expansion, and assist corrosion inhibition and fire retardation. Fillers increase the viscosity of the freshly mixed system but impart thixotropy, which is useful in application to vertical surfaces.

Unmodified epoxy systems tend to be brittle when cleavage or peel forces are imposed. Toughening of the cured adhesive can be achieved by the inclusion of a dispersed rubbery phase which absorbs energy and prevents crack propagation. Epoxies are generally tolerant of many surface and environmental conditions and possess relatively high strength. They are preferred for bonding to concrete since, of all adhesives, they have a particularly high tolerance of the alkalinity of concrete, as well as moisture. By suitable formulation, their ability to wet out the substrate surfaces can even be achieved in the presence of water, the resin being able to disperse the water from the surface being bonded (Tabor, 1978).

Unsaturated polyester resins were discovered in the mid-1930s and have adhesive properties obtained by cross-linking using a curing agent. They are chemically much more simple than epoxy resins but have a 10% contraction by volume during curing due to a volume change during the transition from the uncured liquid phase to the hardened resin resulting in further curing shrinkage. As a result of these factors, there are usually strict limits on the volume of material that can be mixed and applied at any one time and as a general rule polyester resins do not form as strong adhesive bonds as do epoxy resins. In storage, the polyester resins are also somewhat less stable and present a greater fire hazard than epoxies. These limitations significantly restrict their applications.

The advantages of epoxy resins over other polymers as adhesive agents for civil engineering use can be summarised as follows (Mays and Hutchinson, 1992):

- High surface activity and good wetting properties for a variety of substrates.
- May be formulated to have a long open time (the time between mixing and closing of the joint).

- High cured cohesive strength, so the joint failure may be dictated by the adherend strength, particularly with concrete substrates.
- May be toughened by the inclusion of a dispersed rubbery phase.
- Minimal shrinkage on curing, reducing bondline strain and allowing the bonding of large areas with only contact pressure.
- Low creep and superior strength retention under sustained load.
- Can be thixotropic for application to vertical surfaces.
- Able to accommodate irregular or thick bondlines.
- Formulation can be readily modified by blending with a variety of materials to achieve desirable properties.

These various modifications make epoxy adhesives relatively expensive in comparison to other adhesives. However, the toughness, range of viscosity and curing conditions, good handling characteristics, high adhesive strength, inertness, low shrinkage and resistance to chemicals have meant that epoxy adhesives have found many applications in construction, for example, repair materials, coatings and as structural and non-structural adhesives.

2.2.2 Requirements of the adhesive for plate bonding

There are many features of an adhesive product, in addition to its purely adhesive properties, which will form the basis for the selection of a particular bonding system. Mays (1985) has considered requirements for adhesives to be used for external plate bonding to bridges under conditions prevalent in the UK. These requirements are extended and refined in a later publication referred to as a proposed Compliance Spectrum (Mays and Hutchinson, 1988), which addresses the general engineering requirements of adhesives, bonding procedures and test methods for structural steel-to-concrete bonding, based on research work at the University of Dundee (Hutchinson, 1986). The requirements proposed for the adhesive itself can be considered to be equally applicable to fibre reinforced polymer (FRP) plate bonding. An epoxy resin and polyamine hardener are recommended.

Choice of a suitable adhesive is only one of a number of requirements for a successfully bonded joint. Other factors also affect the joint strength and performance (Mays and Hutchinson, 1988) namely:

- appropriate design of the joint
- adequate preparation of the adherend surfaces
- controlled fabrication of the joint
- protection from unacceptably hostile conditions in service
- postbonding quality assurance.

Both short term and long term structural performance are likely to be improved by using an appropriately designed joint and suitably preparing the surface of the substrate materials. A review of factors important to the satisfactory design of joints is given by Adams and Wake (1984) and Lees (1985) and will not be considered here. Full account must be taken of the poor resistance of adhesives to peel and cleavage forces; shear strength itself is unlikely to be a limiting factor. With concrete structures, the tensile/ shear, or tear-off strength of the concrete should be the critical design factor if a suitable adhesive formulation is selected and appropriate methods of surface preparation implemented. This has been demonstrated through detailed shear testing on site and in the laboratory (Moustafa, 1974; Hugenschmidt, 1975; Schultz, 1976).

2.2.3 Tests to measure structural adhesive bond strength

A number of tests are available for testing adhesive and thin films (Adams and Wake, 1984; Kinloch, 1987). However, appropriate tests for assessing bond strength in construction are complicated by the fact that the loading condition in service is difficult to simulate, and one of the adherends, namely concrete, tends to be weaker in tension and shear than the adhesives which may be used, making discrimination between adhesive systems difficult. As a result, confirmation of the suitability of a proposed adhesive system is generally limited to demonstrating that, when the bondline is stressed in the test configuration chosen, the failure surface occurs within the concrete substrate. Such tests may also be used to exhibit the adequacy of the surface preparation techniques employed, since it is difficult to separate the individual effects on adhesion of the adhesive type and method of surface treatment.

Several possible test methods have evolved to measure the bond strength between adhesive and concrete substrates, mainly for applications in concrete repair (Franke, 1986; Naderi *et al.*, 1986). The Réunion Internationale des Laboratoires d'Essais et de Recherches sur les Matériaux et les Constructions (RILEM) Technical Committee 52-RAC lists some currently used laboratory and field test methods for assessing the bond between resin and concrete (Sasse and Friebrich, 1983). Procedures are mentioned on the strength of adhesion in tension, shear and bending, as well as shrinkage and thermal compatibility in the context of coatings, concrete repair, concrete/ concrete and steel/concrete bonds.

Variations of the slant shear test (Kreigh, 1976), in which two portions of a standard cylinder or prism are joined by a diagonal bondline and then tested in compression, have been found to produce discriminating and consistent results (Kreigh, 1976; Naderi, 1985; Wall *et al.*, 1986). Tu and Kruger (1996) used such a configuration to demonstrate that a flexible, tough epoxy provided improved adhesion compared to a more brittle material because it allows redistribution of forces before fracture. However, Tabor (1985) concluded that the slant shear test is of little use in assessing adhesion between resin and concrete because the interfaces are not subjected to tensile forces.

In assessing the shear connection in steel/concrete composite construction, tests at the Wolfson Bridge Research Unit at the University of Dundee employed a kind of double-lap joint configuration as described by Solomon (1976), in which fracture was characterised by shear failure of the concrete adjacent to the interface with the adhesive.

The University of Surrey (Quantrill *et al.*, 1995) have reported a programme of small scale tests to investigate three different adhesives, two of which were two-part cold cure epoxies and the third a two-part acrylic. The tests involved subjecting an adhesive/concrete joint to tensile force and a composite/adhesive/concrete joint to shear, to verify the adequacy of the surface preparation of the concrete and composite bond surfaces. In these tests the Sikadur 31 PBA epoxy adhesive was superior to the two other products and demonstrated strengths in both tension and shear which exceeded those of the concrete. The acrylic adhesive failed within the adhesive under very small ultimate loads.

Chajes *et al.* (1996) used a single-lap specimen, in which a strip of carbon composite was bonded to a concrete prism, to study the bond strength of composite plate materials bonded to concrete. Four different adhesives were used to bond the composite strip; three two-part cold cure structural epoxies and a two-part cold cure urethane. Three methods of surface preparation were studied, varying in severity from untreated to mechanically abraded to expose the coarse aggregate. It was found that all epoxy-bonded joints failed as a result of the concrete shearing directly beneath the bond surface at similar loads. The final strength was therefore a function of the concrete strength. The surface treatment which involved exposing the coarse aggregate produced the highest average strengths. The urethane adhesive, which was much less stiff and had a much higher ductility to failure in tension than the epoxies, failed within the adhesive at lower ultimate loads. It is of interest to note that a silane surface primer was used on two of Chajes' adhesives (the primer used was Chemglaze 9926) and it improved the bond performance of the joints compared with a joint not treated thus; when used on concrete the primer tends to improve the bond by strengthening the surface of the concrete and making it water repellent.

Karbhari and Engineer (1996) describe the use of a modified peel test for investigation of the bond between composite and concrete, in which a composite strip is pulled away from the concrete at a known angle and at a controlled rate. The test is said to provide a good estimate of interfacial energy and could be used in durability assessment.

2.3 External strengthening using steel plates

2.3.1 Introduction

A review of some significant experimental investigations conducted using steel plates is presented to demonstrate some of the structural implications of external plating. Research work into the performance of members strengthened with steel plates was pioneered simultaneously in South Africa and France in the 1960s (L'Hermite and Bresson, 1967; Fleming and King, 1967; Lerchenthal, 1967; Gilibert *et al.*, 1976). Continued development of suitable adhesives and the increased use of the technique in practice stimulated further research work. Eberline *et al.* (1988) present a literature review on research and applications related to steel plate bonding.

2.3.2 Structural investigations

The history of bonded external reinforcement in the UK goes back to 1975 with the strengthening of the Quinton Bridges on the M5 motorway. This scheme followed a number of years of development work by the Transport and Road Research Laboratory (TRRL), (now TRL), in association with adhesive manufacturers and the Department of Transport. In terms of testing programmes, research and development work continued at the TRRL and at several academic institutions in the UK, most notably at the University of Sheffield. Theoretical investigations and the evaluation of suitable adhesives were allied to the extensive beam testing programmes which were undertaken.

Preliminary studies were conducted by Irwin (1975). Macdonald (1978) and Macdonald and Calder (1982) reported four point loading tests on steel plated RC beams of length 4900 mm. These beams were used to provide data for the proposed strengthening of the Quinton Bridges (Raithby, 1980 and 1982), and incorporated two different epoxy adhesives, two plate thicknesses of 10.0 mm and 6.5 mm giving width-to-thickness (*b/t*) ratios of 14 and 22, and a plate lap-joint at its centre.

In all cases it was found that failure of the beams occurred at one end by horizontal shear in the concrete adjacent to the steel plate, commencing at the plate end and resulting in sudden separation of the plate with the concrete still attached, up to about mid-span. The external plate was found to have a much more significant effect in terms of crack control and stiffness. The loads required to cause a crack width of 0.1 mm were increased by 95%, whilst the deflections under this load were substantially reduced. The postcracking stiffness was found to be increased by between 35–105% depending upon the type of adhesive used and the plate dimensions.

The features of this work became the subject of a more detailed programme of research at the TRRL (Macdonald, 1982; Macdonald and Calder, 1982), in which a series of RC beams of length 3500 mm were tested in four point bending. The beams were either plated as-cast or plated after being loaded to produce a maximum crack width of 0.1 mm. The effect of widening the plate whilst maintaining its cross-sectional area constant was studied. It was found that the plated as-cast and the precracked beams gave similar load/deflection curves, demonstrating the effectiveness of external plating for strengthening purposes.

An extensive programme of research work carried out at the University of Sheffield since the late 1970s has highlighted a number of effects of external, epoxy-bonded steel plates on the serviceability and ultimate load behaviour of RC beams. A brief summary of some of the research findings is presented by Jones and Swamy (1995).

Steel plate strengthening of existing structures has also been investigated in Switzerland at the Swiss Federal Laboratories for Material Testing and Research (EMPA) (Ladner and Weder, 1981). Bending tests were carried out on RC beams 3700 mm in length, and the plate width-to-thickness (b/t) ratio was studied whilst maintaining the plate cross-sectional area constant. The external plate continued through and beyond the beam supports, with which they were not in contact, for a distance such that the bonded area (48 000 mm^2) was the same for each plate width. The external plate was not bonded to the concrete beam except in the anchorage areas beyond the supports. The results clearly showed that thin plating was more effective than thick narrow plating, as noted in studies conducted in the UK. The effective anchorage length l_a which allowed the plate to reach yield before shear failure adjacent to the bonded areas was found to be inversely proportional to the b/t ratio. Therefore, as b/t increased (wide, thin plates), the anchorage length decreased.

2.3.3 Plate separation and anchorage

The ultimate behaviour of steel plated RC beams appears to be closely related to the geometry of the plated cross-section. For thin plates, failure usually occurs in flexure. However, if the plate aspect ratio falls below a certain value, separation of the plate from the beam can occur, initiating from the plate end and resulting in the concrete cover being ripped off. These observations are consistent with the fact that simple elastic longitudinal shear stresses are inversely proportional to the plate width. Consequently, as the steel plate width decreases, the longitudinal shear stresses increase. In addition, the bending stiffness of the plate increases, thereby increasing the peeling stresses normal to the beam.

However, the levels of stress at the steel plate ends are thought to be well in excess of those due to simple elastic considerations (Macdonald, 1982). Concentrations of shear and normal stress arise at the plate ends of beams subjected to flexure as a result of stiffness incompatibility between the plate and concrete, which can only be accommodated by severe distortion of the adhesive layer. The sudden transition from the basic unplated members to the plate reinforced member is usually situated in a region of high shear and low bending moment. The changing bending moment and distortion in the adhesive layer causes a build-up of axial force at the end of the external plate; this induces high bond stresses on the adhesive/plate and adhesive/concrete interfaces which may reach critical levels, thereby initiating failure. The magnitude of these plate end stresses for externally strengthened beams depends upon the geometry of the plate reinforcement, the engineering properties of the adhesive and the shear strength of the original concrete beam (Swamy and Mukhopadhyaya, 1995). The existence of peak peeling and shear stresses at the plate end, in addition to bending stresses, results in a biaxial tensile stress state which forces the crack initiated at the plate end to extend horizontally at the level of the internal steel.

When failure occurs in this way, the use of a more flexible adhesive is advantageous, since the region over which the tensile strain builds up in the external steel plate is extended, thereby resulting in a lower peak stress. This has been verified experimentally by Jones *et al.* (1985), where beams strengthened using an adhesive with an elastic modulus of around $1.0 \times 10^3\,\mathrm{N\,mm^{-2}}$ gave slightly improved strengths when failure occurred by plate separation than strengths given by an adhesive with a modulus of around $10 \times 10^3\,\mathrm{N\,mm^{-2}}$.

As the structural benefits of external plating with steel are enhanced by the use of larger, thicker plates, an alternative to limiting the areas (or perhaps as a safeguard against separation), would be the provision of some form of plate anchorage. Jones *et al.* (1988) presented theoretical and experimental studies into the problem of anchorage at the ends of steel plates. A series of RC beams 2500 mm in length, strengthened with epoxy-bonded steel plates of 6.0 mm thickness were tested to investigate different plate end anchorage schemes. Four 6.0 mm diameter bolts at each end of the plate, which penetrated to a depth of 75 mm, were used in one configuration, whilst different sizes of angle plates were also tried, one of which covered the extent of the shear span, and compared with those of a beam plated with a single unanchored steel plate of *b/t* ratio 21, which failed suddenly by plate separation at a load which was below that of the unplated control beam. It was found that the anchorage detail had no apparent effect on the deflection performance of the beams. The use of bolts did not prevent debonding, but complete separation was avoided and increases in strength up to 8% over the unplated beam were achieved. The bonded anchor plates were more effective, producing yielding of the tensile plates and allowing the full theoretical strength to be achieved, 36% above that of the unplated beam. The anchorage detail was also found to affect the ductility of the beams near the ultimate load. Unanchored, the beams failed suddenly with little or no ductility. The beams with bolts or anchor plates all had similar ductilities, at least as high as the unplated control.

Hussain *et al.* (1995) investigated the use of anchor bolts at the ends of steel plated beams, in an attempt to prevent brittle separation of the plate. In agreement with Jones *et al.* (1988) the bolts, which were 15 mm in diameter and penetrated to half the depth of the beam, were found to improve the ductility of the plated beams considerably, but to have only a marginal effect on the ultimate load. The percentage improvement in ductility due to the addition of bolts was found to decrease as the plate thickness increased. The end anchorage could not prevent premature failure of the beams, although in this case failure occurred as a result of diagonal shear cracks in the shear spans.

It will be realised that in providing anchorage to the steel plated beams, considerable extra site work is involved and this in turn will increase the cost of the plate bonding technique considerably. However, with steel plate bonding this anchorage is completely necessary.

2.3.4 Disadvantages of external strengthening using steel plates

The *in situ* rehabilitation or upgrading of RC beams using bonded steel plates has been proven in the field to control flexural deformations and crack widths, and to increase the load-carrying capacity of the member under service load for ultimate conditions. It is recognised to be an effective, convenient and economic method of improving structural performance. However, although the technique has been shown to be successful in practice, it also has disadvantages. Since the plates are not protected by the concrete in the same way as the internal reinforcement, the possibility of corrosion exists which could adversely affect the bond strength, leading to failure of the strengthening system. Uncertainty remains regarding the durability and the effects of corrosion. To minimise the possibility of corrosion, all chloride-contaminated concrete should be removed prior to bonding and the plates must be subjected to careful surface preparation, storage and the application of resistant priming systems. After installation, the integrity of the primer must be periodically checked, introducing a further maintenance task to the structure. The plates are generally prepared by grit blasting which, unless a minimum thickness of typically 6 mm is imposed, can cause distortion.

Steel plates are difficult to shape in order to fit complex profiles. In addition, the weight of the plates makes them difficult to transport and handle on site, particularly in areas of limited access, and can cause the dead weight of the structure to be increased significantly after installation. Elaborate and expensive falsework is required to maintain the steelwork in position during bonding. The plates are required to be delivered to site within flatness tolerances to prevent stresses being introduced normal to the bondline during cure. The weight of the plates and this flatness requirement generally restricts the maximum plate length to between 6–8 m. Since the spans requiring strengthening are often greater than this length, joints are required. Welding cannot be used in these cases since this would destroy the adhesive bond. Consequently, lapped butt joints have to be formed, adding further complications to the design of the system.

Studs are required to assist in supporting the steel plates during installation and under service loading conditions. This is especially true towards the ends of the plates where anchorages are required due to the high bending stiffness of the plate. The position of these studs must therefore be established prior to bonding. This process can involve a considerable amount of site work. Finally, if the plates are loaded in compression, buckling may occur, causing the plates to become detached.

The process involved in strengthening with steel plates can therefore be considered as relatively time consuming and labour intensive.

2.4 External strengthening using composite materials

2.4.1 Introduction

To overcome some of the shortcomings that are associated with steel plate bonding, it was proposed in the mid-1980s that fibre reinforced polymer (FRP) plates could prove advantageous over steel plates in strengthening applications (Meier, 1987; Kaiser, 1989; Meier and Kaiser, 1991). Unlike steel, FRPs are unaffected by electrochemical deterioration and can resist the corrosive effects of acids, alkalis, salts and similar aggressive materials under a wide range of temperatures (Hollaway, 1993). Consequently, corrosion-resistant systems are not required, making preparation prior to bonding and maintenance after installation less arduous than for steel.

The reinforcing fibres can be introduced in a certain position, volume fraction and direction in the matrix to obtain maximum efficiency, allowing the composites to be tailor made to suit the required shape and specification. The resulting materials are non-magnetic, non-conductive and have high specific strength and stiffness in the fibre direction at a fraction of the weight of steel. They are consequently easier to transport and handle, require less falsework, can be used in areas of limited access and do not add significant loads to the structure after installation. Continuous lengths of

FRP can be readily produced which, because of their low bending stiffness, can be delivered to site in rolls. The inclusion of joints during installation is thus avoided. With the exception of glass fibre composites, FRPs generally exhibit excellent fatigue and creep properties and require less energy per kilogram to produce and transport than metals. As a result of easier installation in comparison to steel, less site disruption should be experienced in the process, allowing faster, more economical strengthening.

The benefits of utilising FRP materials over steel in plate bonding applications are thus clear. The drawbacks are the intolerance to uneven bonding surfaces which may cause peeling of the plate, the possibility of brittle failure modes (Swamy and Mukhopadhyaya, 1995) and the material cost, since fibre composites are between 4–20 times as expensive as steel in terms of unit volume. However, in a rehabilitation project, where material costs rarely exceed 20% of the overall project cost, the installation savings can offset the higher material costs (Meier, 1992). Peshkam and Leeming (1994) have considered the commercial viability of FRP plate bonding for bridge strengthening. In a straight comparison with steel plate bonding for a typical application, despite the fact that material costs will be increased, labour and equipment costs will be reduced, construction times will be shorter and durability will be improved. It is shown that 2 kg of FRP could replace 47 kg of steel on an equal strength basis. The costs of installing both materials are shown to be similar; however, when traffic management, traffic delay and maintenance costs are included, the use of FRP provides a saving of 17.5% over steel. There are situations where steel plate bonding is not a viable option because of the extent of chloride contamination of the concrete. In such cases, the use of FRP may avoid the need for demolition and replacement. Peshkam and Leeming (1994) presented a cost comparison of bridge replacement against strengthening with FRP, in which possible savings of 40% are demonstrated. These cost comparisons were made before true manufacturing and installation costs were known and were at the best estimates. Subsequently the tendering process for real installation projects has shown carbon fibre reinforced polymer (CFRP) plate bonding to be very competitive against steel plate bonding in first cost, before even future maintenance costs are added to the whole life cost equation.

The general versatility of composite materials makes them a viable alternative to steel plates in strengthening applications, resulting in both short term and long term savings. Meier and Winistorfer (1995) consider that for applications in which the possibility of corrosion is minimal and the length of the strengthening is less than 8 m, steel will remain the most favourable option. However, more recent work by other researchers (ROBUST) and trends in costs, show that this position is changing and the indications are that FRP is more economical than steel whatever the length. This is the case mainly in building construction, although plate thickness may be important from an aesthetic viewpoint. In applications where corrosion, length of the required strengthening and handling on site are of greater significance, for example bridge rehabilitation, fibre composites become a more attractive alternative.

Concerns have been expressed regarding the behaviour of FRP strengthened members when exposed to fire. A series of tests has been carried out at EMPA in Switzerland in which the performance of steel and CFRP plated beams was compared when exposed to extreme high temperatures (Deuring, 1994). It was found that a steel plate became detached after a matter of minutes of exposure, whereas the CFRP laminates progressively lost cross-sectional area due to burning at the surface, causing a gradual loss of stiffness of the member, before final detachment after over an hour. This superior behaviour is a consequence of the low thermal conductivity of the composite. In addition, detachment of a heavy steel plate from a structure for any reason presents a far greater hazard than that of a lightweight FRC material. Aspects of the effects of fire on resin compounds are considered by Tabor (1978) and Hollaway (1993a).

Glass, aramid and carbon fibre composites may be considered for strengthening applications. With particular regard to plate bonding, a comparison of the important characteristics of FRP produced from these fibre types is shown in Table 2.1, in which the fibre volume fraction is typically around 65% and the fibres are unidirectionally aligned.

In common usage, glass is the most popular reinforcing fibre since it is economical to produce and widely available. However, concern exists regarding the durability of composites composed of glass fibres, especially for structural uses involving concrete, as discussed in Chapters 2 and 6. Carbon fibres exhibit better resistance to moisture, solvents, bases and weak acids, and can withstand direct contact with concrete (Santoh *et al.*, 1983). Composite materials produced from them are light in weight, with strengths

Table 2.1 Comparison of characteristics of FRC sheet produced from different fibres (Meier, 1995)

Characteristics	Carbon	Aramid	E-glass
Tensile strength	Very good	Very good	Very good
Compressive strength	Very good	Inadequate	Good
Stiffness	Very good	Good	Adequate
Long term behaviour	Very good	Good	Adequate
Fatigue behaviour	Excellent	Good	Adequate
Bulk density	Good	Excellent	Adequate
Alkaline resistance	Very good	Good	Inadequate
Cost	Adequate	Adequate	Very good

higher than steel and stiffnesses higher than either glass or aramid composites. For example, laminates fabricated from glass fibre must be three times thicker than CFRP laminates to achieve the same tensile stiffness for the same fibre volume fraction. CFRP has excellent fatigue properties and a very low (or even negative) linear thermal coefficient of thermal expansion in the fibre direction. Quality assurance can be performed by non-destructive testing, for example infrared inspection in the field, if CFRP laminates are used; this is not possible with steel plates. This technique allows fast and accurate judgement on the quality of the strengthening work.

Despite the higher cost, carbon composites appear to provide the best characteristics for structural strengthening.

2.4.2 Review of experimental investigations

The following section reviews, on a geographical basis, experimental work reported to investigate the flexural strengthening of RC members using non-prestressed FRP plates. These studies have utilised fibrous materials in various forms, including pultruded plates, precured prepreg plates, prepreg sheets or tapes cold laminated in place, and dry fibre sheets impregnated at the time of bonding.

2.4.2.1 Some investigations in Europe

Recent work on the use of FRP materials as a replacement for steel in plate bonding applications was pioneered at the EMPA in Switzerland. Four point loading tests were initially performed on RC beams 2000 mm (Meier, 1987; Kaiser, 1989) or 7000 mm (Ladner *et al.*, 1990) in length. Strengthening was achieved through the use of pultruded carbon fibre/epoxy laminates up to 1.0 mm thick bonded with the same epoxy adhesives used in earlier steel plating work (Ladner and Weder, 1981). For the 2000 mm length beams, the ultimate load was almost doubled over the unplated control beam, although these beams were designed with a low proportion of internal steel, and hence the strength of the unplated beam was low. In the case of the 7000 mm length beam, strengthened with a 1.0 mm CFRP laminate, the increase in the ultimate load was about 22% (Ladner and Holtgreve, 1989). However, for both beam lengths the ultimate deflection was considerably reduced, although it was claimed that there was still sufficient rotation to predict impending failure.

The following modes were observed either individually or in combination in the tests carried out at the EMPA:

- sudden, explosive, tensile failure of the CFRP laminates
- compressive failure in the concrete

- slow, continuous peeling of the laminate during loading resulting from an uneven concrete bond surface
- sudden peeling of the laminate during loading due to relative vertical displacement across a shear crack in the concrete
- horizontal shearing of the concrete in the tensile zone
- interlaminar shear within the CFRP sheet.

The CFRP plate was found to reduce the total width of cracks and produce a more even crack distribution over the length of the beam (Meier and Kaiser, 1991). Meier *et al.* (1992) recommended that in strengthening applications, the external CFRP should fail in tension after yielding the internal steel but before failure of the concrete in the compressive zone, since this would ensure a more ductile failure mode.

Deblois *et al.* (1992) investigated the application of unidirectional and bidirectional glass fibre reinforced polymer (GFRP) sheets for flexural strengthening. A series of RC beams 1000 mm long were tested after strengthening. The use of bidirectional sheets increased the ultimate load by up to 34%, whereas unidirectional GFRP resulted in an increase of only 18%. The authors of this current chapter feel that this is an unexpected conclusion and emphasise that the FRP material used was GFRP. The additional bonding of bidirectional GFRP to the sides of the beam increased the load carried with unidirectional sheets to 58%. To further the programme of study, Deblois *et al.* (1992) epoxy-bonded a bidirectional GFRP sheet to the soffit of a 4100 mm long RC beam. Bolts were also used as additional anchorage at the plate ends. The maximum load increased by 66% over the unplated control beam. It was noted that for all tests the application of GFRP reduced the ductility of the beam.

Research into external FRP plating has been conducted at Oxford Brookes University (Hutchinson and Rahimi, 1993). The effect of plate-end geometries on the stress concentrations at the plate ends was of primary interest in this investigation, for which RC beams 2300 mm in length were used. Two beams were preloaded to 80% of their ultimate strength, before plating, to cause cracking of the concrete and yielding of the steel reinforcement. Unidirectional carbon fibre/epoxy prepreg tape of total thickness 0.78 mm was used for the plating, with the various plate end geometries, tapering in either plan or section, cut whilst the composite was in the uncured state. Several different two-part cold cure epoxy adhesives were evaluated using a modified Boeing wedge cleavage test, of which no detail is given, developed to measure adhesion to concrete surfaces. The adhesive selected as most suitable was Sikadur 31 PBA, an epoxy which has been used in both steel and FRP plate bonding applications in Switzerland (Meier and Kaiser, 1991) and in steel plating applications in the UK (Shaw, 1993).

It was found that the flexural performance of all strengthened beams was significantly better than the unplated specimens, in terms of both strength and stiffness. The ultimate load-carrying capacity was increased by as much as 230%; however, it should be pointed out that the actual increase is dependent upon the degree of internal reinforcement in the beam before plating. The increased stiffness resulted in an increased load to first cracking, but a substantial decrease in ductility to failure. After first cracking, cracks grew progressively in number, covering most of the test span. Most of these were of hairline width even close to the ultimate load level. In all cases, failure was sudden and catastrophic, characterised by a shear crack running from the tensile zone towards the loading point and delamination of the concrete cover along the tensile reinforcement. This type of failure has been identified in steel plating work as described above. Tapering of the plate end in either plan or section appeared to have no effect on the flexural performance or failure mode for the cases considered.

As with steel plates, the beams which had been precracked before bonding had an equivalent performance to the other test beams, indicating the effectiveness of the plate bonding technique for repair. The load/deflection behaviour was similar for all different plate configurations, except for those with laminates bonded to the full length of the beam, clamped by the reaction at the supports, which resulted in an increase in strength over the other plated beams. It was concluded that for these particular beams and plates the ultimate loading capacity of the system appeared to have been reached, being governed by the shear capacity of the concrete beams.

The tests at Oxford Brookes University continued (Hutchinson and Rahimi, 1996), under the ROBUST programme of research, by utilising both glass and carbon fibre/epoxy laminates of different thicknesses built up from prepreg tapes. Three internal steel reinforcement ratios were examined. All beams with external reinforcement performed significantly better than their unplated counterparts in terms of stiffness and strength. The use of GFRP was found to provide significant ductility and reasonable strength, whilst enhancements were greater with CFRP but at the expense of a loss of ductility. Greater enhancements were achieved with lower steel ratios.

A limited programme of experimental testing has been carried out at the University of Bologna (Arduini *et al.*, 1994; Arduini *et al.*, 1995) in which small scale steel fibre reinforced concrete specimens of length 500 mm or 600 mm have been tested in three point bending after being strengthened with unidirectional aramid fibre/epoxy or glass fibre/epoxy composites of thickness between 2.0–5.0 mm. These small scale tests were used to demonstrate that the load-carrying capacity of the basic unplated beam could be increased through external plating with FRC but that different failure modes, often brittle, were involved. It was noted that peeling and shear cracks at the plate ends were responsible for causing premature, brittle failure. The use of thicker FRC plates was found to increase the occurrence of peeling failure. Ductility was increased and peeling failure delayed through the use of plates bonded to the sides of the beams in the plate end regions; the effects were enhanced by coupling the side and soffit plates together, in which case failure was observed to occur by diagonal shearing at the highest attained loads.

The University of Surrey (Quantrill *et al.*, 1995) under the ROBUST programme of research, undertook a parametric study on flexurally strengthened RC beams using GFRP bonded plates. The study involved varying the concrete strength, the pultruded composite plate area and its aspect ratio (*b/t*), and as discussed above in steel plating applications, thick narrow plates with aspect ratios of less than 50 have been associated with brittle peeling failure modes. Consequently, ratios of 38 and 67 were tested in the study. The effect of the *b/t* ratio was isolated in these tests by maintaining a constant plate cross-sectional area. The tests showed that plating can considerably enhance both the strength and stiffness of RC members, although at the expense of ductility at failure. It appeared that the higher strength concrete produced the greatest increase in strength over the unplated section and the aspect ratio of the plate has little effect on the overall behaviour.

The above programme continued with further investigations at the University of Surrey (Quantrill *et al.*, 1996a) into the experimental and analytical strengthening of reinforced concrete beams with fibre reinforced polymer plates, and analysed the effects of different plate parameters on the overall behaviour of the system. It was shown that testing relatively small scale 1 m long specimens can reveal useful information on strengthened beam behaviour. By reducing the plate area the expected reduction in strengthening and stiffening caused the ductility and the plate strains for a given load to increase; the aspect ratio for the values tested had little effect on the overall response. Plating with CFRP components increased the serviceability, yield and ultimate loads and increased the strengthened member stiffness after both cracking and yielding; ductility was reduced. The iterative analytical model accurately predicted the tensile plate strain and compressive concrete strain responses of the beam for a partially cracked section.

Quantrill *et al.* (1996b) continued with tests on small scale specimens and showed that when the CFRP plated beams were uncracked at their extremities the theoretical shear stress reached $11.15\,\mathrm{N\,mm^{-2}}$ and the peel stress $6.37\,\mathrm{N\,mm^{-2}}$. The anchored CFRP plated beams were able to sustain higher levels of shear and peel stress before failure occurred around $14.1\,\mathrm{N\,mm^{-2}}$ in shear and $8.10\,\mathrm{N\,mm^{-2}}$ in peel stress.

In France, a programme of small scale tests has also been carried out to study the effects of different adhesive and FRC combinations when used for external strengthening (Varastehpour and Hamelin, 1995). A series of plain concrete specimens 280 mm in length were tested to failure in four point bending after being strengthened with glass or carbon fibre/epoxy sheets bonded to both the tension and side faces of the specimen with one of four different epoxy or acrylic adhesives. The composite plate on the tension face was anchored by the reactions at the supports in all cases. Failure occurred either by FRP rupture, interface failure or by debonding of the plate from the concrete. In all cases the flexural and shear capacity of the beams was increased by plating, although this was found to be dependent on the choice of adhesive; in general, the epoxies performed better than the acrylics, the tests demonstrating that a rubber-toughened epoxy was superior.

Under the ROBUST research programme, Garden *et al.* (1996) showed that the ultimate capacity of the CFRP beams falls with reducing the width–thickness *b/t* and beam shear span/depth ratios. Failure under low shear span/beam depth ratios is associated with high plate strains (the value being in the region of 70% of the plate ultimate strain) and relatively high longitudinal shear stresses at the adhesive/concrete interface, and although the concrete failed in the cover concrete area, debonding from the concrete was not observed. Plate end anchorage delays failure by resisting plate separation but does not increase stiffness until the internal reinforcement has yielded.

He *et al.* (1997a), at the University of Sheffield, used steel and CFRP plates with the same axial stiffness-to-strength precracked reinforced concrete beams in which a new, but unspecified, plate anchorage system was adopted. The basic improvement in structural performance due to plating was verified and it was found that the CFRP plates produced a greater improvement in ultimate load than the steel plates. The authors (He *et al.*, 1997b) noted that the high stress and strain potential of the CFRP will not be utilised unless the plate is prestressed.

Bencardino *et al.* (1997) tested CFRP plated beams at the University of Calabria, Italy, recording reductions in member ductility due to plating without end anchorage; the ductility was restored when anchorage was fitted in the form of externally bonded U-shaped steel stirrups. The method of CFRP plating was used successfully to strengthen an experimental portal structure.

2.4.2.2 Some investigations in North America

During the late 1980s, a pilot study was carried out at the University of Arizona to establish the feasibility of poststrengthening concrete bridge beams with GFRP plates (Saadatmanesh and Ehsani, 1989 and 1990a). Selection of a suitable epoxy adhesive for plate bonding purposes was the main subject of investigation. Five RC beams 1675 mm in length were tested in four point bending to determine their static strength. None of the beams contained shear reinforcement, which resulted in premature failure in the first tests. To prevent shear cracks causing separation of the plate from the beam, external shear reinforcement was thus provided in the end regions by means of several large G-clamps. One beam was unplated, while the remainder were strengthened with a 6.0 mm thick GFRP plate bonded with one of four different types of two-part cold cure epoxy with a range of shear strengths from 13 MPa to 16 MPa using aluminium substrates.

It was found that flexible epoxies did not allow any measurable shear to be transferred between the plate and the beam, and no increase in the ultimate strength was achieved in comparison to the unplated control beam. For the most rigid epoxy, after the concrete had cracked in tension, the plate was observed to separate from the beam in a very brittle manner, again resulting in no increase in ultimate capacity. The beam strengthened using a relatively viscous rubber-toughened epoxy was found to perform best in the tests carried out. This beam was significantly stronger and stiffer than the unplated control beam. Substantial force developed in the plate indicated good shear transfer and composite action between the plate and the concrete beam. The cracks were found to be considerably smaller throughout the range of loading and distributed more evenly along the length of the beam. Failure occurred when a layer of concrete delaminated about 10 mm above the bondline, indicating satisfactory performance of the epoxy.

Following on from this initial study, a further experimental research project was undertaken at the University of Arizona (Saadatmanesh and Ehsani, 1990b and 1991). In this project, five rectangular beams and one T-beam were tested to failure in four point bending over a clear span of 4570 mm. All beams were strengthened with a GFRP plate 6.0 mm thick bonded to the concrete with the epoxy adhesive identified as most suitable for the application from the previous study. Three different reinforcement ratios were used for the tension steel in the beams. The majority of the beams were overdesigned for shear to prevent premature shear failure.

The tests indicated that significant increases in the external load, at which the steel yielded, and an increase in flexural strength, could be achieved by bonding GFRP plates to the tension face of RC beams. The gain in the ultimate flexural strength was found to be significant in beams with lower steel reinforcement ratios, as noted later by Hutchinson and Rahimi (1996). In addition, plating reduced the crack size in the beams at all load levels. For several of the beams tested, failure occurred as a result of sudden longitudinal shear failure of the concrete between the plate and internal steel reinforcement. The flexural stiffness was increased, although the ductility of the beams and curvature at failure were reduced by the addition of the GFRP plates.

An experimental programme was undertaken by Chajes *et al.* (1994) at the University of Delaware, in which RC beams 1120 mm in length were loaded to failure in four point bending after the majority had been externally strengthened with composite fabric of either bidirectional woving aramid, E-glass or carbon fibre reinforcement. In each case, the fabrics had a tensile capacity close to the yield strength of the steel. Fabrics were used as an alternative to plates to exploit their ability to conform to irregular surface geometries, thus reducing the possibility of the continuous peeling failures observed in testing at the EPMA. No shear reinforcement was provided in the beams. A variety of layers of each fabric were epoxy-bonded to the concrete. A set of three beams were also prepared and tested with twice the amount of internal steel reinforcement.

It was found that the mode of failure of the strengthened beams varied depending upon the fabric used; those externally strengthened with E-glass and carbon fibres failed by tensile rupture of the fabric. The first aramid strengthened beam exhibited a fabric debonding mode of failure and consequently, for the remaining specimens, end tabs were included, bonded to the sides of the beam and enclosing the soffit reinforcement in the end regions. The extent of the end tabs in the shear spans is not clear from the publication, but their use prevented debonding, allowing failure of the concrete in compression to occur.

For each of the fabric types used, increases in flexural strength similar to those found in the beams with additional steel reinforcement were achieved, in the range 34–57%. The fabric reinforcement beams also exhibited increases in flexural stiffness within the range 45–53%. Both of the failure modes observed were said to yield a reasonable amount of ductility, although this was around half that obtained from the unstrengthened beams.

The research carried out at the University of Delaware forms part of a wider study concerned with the possibility of rehabilitating deteriorated prestressed concrete box beam bridges using transversely bonded advanced composite materials (Chajes *et al.*, 1993; Finch *et al.*, 1994; Chajes *et al.*, 1995b; Chajes *et al.*, 1996).

The US Navy has been studying the possibility of using external FRP plating for upgrading waterfront structures affected by reinforcement corrosion (Malvar *et al.*, 1995). Enhancements of both bending and shear strength are being considered through the use of unidirectional CFRP tow sheets. RC beams 1680 mm long have been tested in an experimental investigation, none of which contained shear reinforcement. Beams strengthened longitudinally demonstrated that the flexural strength could be significantly enhanced, but failure occurred, not surprisingly, in shear.

When additional CFRP was wrapped onto the sides and soffit of the beam over its full span, to provide shear reinforcement and additional anchorage for the longitudinal CFRP sheets, sufficient shear strength was provided to revert to a bending failure in which the steel yielded, the concrete crushed and then the CFRP material ruptured. However, this occurred at a ductility which was somewhat less than that of the unplated control beam.

In addition to upgrading reinforced concrete beams, research into the feasibility of externally reinforcing continuous RC slab bridges in response to observed longitudinal cracking was initiated in South Dakota (Iyer *et al.*, 1989). To close the observed cracks, the possibility of bonding the external reinforcement whilst the beam was relieved of dead load was examined. The use of both steel (Iyer *et al.*, 1989) and CFRP plates (Iyer, 1988) has been reported. Initial results on small scale beams showed that the strains in the concrete and internal steel were considerably reduced by the introduction of external reinforcement, while the stiffness was increased and cracking was controlled.

2.4.3 Prestressing composite plates for strengthening concrete beams

The utilisation of prestressed composite plates, at the time of bonding, for strengthening concrete members has been studied only relatively recently in comparison with investigations of non-prestressed plates, although the benefits of external prestressing with plate materials have been recognised for many years. For example, Peterson (1965) considered the external prestressing of timber beams using prestressed steel sheets and found significant improvements in bending stiffness and ultimate capacity. External prestressing with composite plates also provides these benefits as well as cost savings. Triantafillou and Deskovic (1991) noted that this method of prestressing is a more economical alternative to conventional prestressing methods used in new construction.

Initial research on the strengthening of reinforced concrete beams by external plate prestressing at EMPA in Switzerland has been widely reported (Meier and Kaiser, 1991; Meier *et al.*, 1993; Deuring, 1994). This work included the cyclic loading of a beam whose plate was prestressed to 50% of its strength. Although this prestress ensures the mean stress level in the cyclic loading was high, there was no evidence of damage to the plate after 30×10^7 cycles and the cracking of the concrete was well controlled. The non-prestressed beam loading tests reported by Deuring (1993) revealed failures by the initiation of plate separation from the base of a shear crack. It was found that the compression transfer into the concrete by the plate prestress could delay or even prevent this type of failure, thereby allowing the plate to reach its ultimate tensile strain so that the beam failed in flexure rather than by premature plate separation (Deuring, 1993). The ability of the plate to alter the failure mode from premature plate separation to flexure is influenced by the prestressing force and the cross-sectional area of the plate. One of the conclusions of the work was that the greatest flexural resistance of a strengthened section is reached when the plate fractures in tension, either after or at the same time as yield of the internal steel rebars.

Saadatmanesh and Ehsani (1991) conducted an experimental study of the strengthening of reinforced concrete beams using non-prestressed and prestressed GFRP plates. One of the two prestressed beams contained a relatively small amount of internal tensile steel reinforcement, while the other contained larger bars and was precracked prior to bonding of the plate. The plate prestress in the precracked case closed some of the cracks, indicating the benefit of prestressing from a serviceability point of view. The beam with little original reinforcement before plating experienced a large improvement in ultimate capacity due to the additional moment couple provided by the plate prestress. In both cases, the prestress was generated by cambering the beam before bonding the plate so that a tensile load was transferred to the plate when the camber was released. Improved concrete crack control was observed with prestressed plates, a clear advantage from a serviceability point of view. A previous experimental study by Saadatmanesh and Ehsani (1990) also included the prestressing of a beam by cambering; this particular specimen had a low internal reinforcement area ratio of 0.32% (based on effective depth) so that a very high increase in ultimate load (323%) was observed as a result. The GFRP plate was not anchored at its ends, and failure occurred by premature plate separation associated with the removal of a layer of concrete from the tension face of the beam.

Triantafillou *et al.* (1992) tested reinforced concrete beams in three point bending with various quantities of internal reinforcement and magnitudes of CFRP plate prestress. Improved control of concrete cracking was brought about not only by a greater internal reinforcement provision, but also by higher plate prestress, indicating the serviceability advantage gained by prestressing the composite. It was noted that prestressed composite plates can potentially act as the sole tensile reinforcement in new concrete construction and prefabrication is also possible due to the simplicity with which composites may be handled and applied. The confinement imposed by the initial compressive stress at the base of the beam was thought to be capable of improving the shear resistance of the member. Also, an advantage from a cost point of view is that the same strengthening to failure may be achieved with a prestressed plate of relatively small cross-section, like that achieved with a larger non-prestressed plate (Triantafillou *et al.*, 1992).

Char *et al.* (1994) conducted an analytical parameter study to determine the effects of varying the cross-sectional area and material type of the composite plate and the prestress in the plate. The parameter study revealed that prestressing a GFRP plate would not necessarily increase the ultimate moment capacity over that of a beam with a non-prestressed plate, for the particular beam configuration and prestress level considered. This was because both the non-prestressed and prestressed beams failed by plate fracture. Garden and Hollaway (1997) showed that prestressing with CFRP plates increases the ultimate capacity of a beam but the magnitude of the increase depends on the failure modes of the beams with and without prestress; the failure mode of the prestressed beam depends on the prestress magnitude.

Wight *et al.* (1995) reported data on the strengthening and stiffening achieved with prestressed CFRP plates. The control of concrete crack widths and numbers of cracks was improved by prestressing the plates. The beam with an initial non-stressed plate failed by concrete fracture in the cover thickness within one of the shear spans of the four point loaded beam, whereas the prestressed plated beams failed by plate fracture in the constant moment region. The compression generated in the concrete near the beam soffit, due to the plate prestress, was sufficient to reduce the magnitudes of vertical displacements across shear cracks and to transfer failure into the plate. The avoidance of concrete failure in the shear spans was associated with a much improved ultimate load.

The testing at the University of Surrey, under the ROBUST programme, continued (Quantrill and Hollaway 1998) by pretensioning the ROBUST pultruded composite plates, prior to bonding to the concrete. The prestressing technique employed was developed and refined on small scale 1.0 m long specimens before being applied to larger 2.3 m long beams. Pretensioning the plate prior to bonding to the concrete beam considerably increased the external applied load at which cracking of the concrete occurred, reduced overall member stiffness and also the load at which visible cracking occurred. The observation of crack control is of significant importance to serviceability based design criteria. It was generally concluded that this technique has the potential to provide a more efficient solution to strengthening problems.

Furthermore, Garden and Hollaway (1997) tested 1.0 m and 4.5 m lengths of reinforced concrete beams in four point bending after strengthening them with externally bonded prestressed CFRP plates. The plates were bonded without prestress and with prestress levels ranging from 25–50% of the plate strength. The ultimate capacities of the plated non-prestressed beams were significantly higher than those of the unplated members and plate prestress brought about further strengthening. The non-prestressed beams failed by concrete fracture in the cover to the internal rebars, whilst most of the prestressed beams failed by plate fracture. The plate prestress prevented cracking of the adhesive layer, a phenomenon associated with shear cracking in the concrete. The plates of the prestressed beams had an initial tensile strain before any external load was applied to the beam system and consequently at this stage, the beams had a relatively high stiffness. It was found that prestressed plates were utilised more efficiently than non-prestressed plates since a given plate strain was associated with a lower plated beam deformation in a prestressed member. Prestressing the composite plates lowers the position of the neutral axis so more of the concrete section is loaded in compression, making more efficient use of the concrete.

All the above experiments were carried out in the laboratory on relatively small scale beams and the method of prestressing could not have been used on site on a real structure where the plate would have to be stressed before bonding within the confines of the abutments or supports. Within the ROBUST project, two 18 m long beams recovered from a real bridge structure, which had to be demolished and reconstructed, were strengthened with plates that were prestressed under conditions that were little different from those that would occur on a real bridge structure (Lane *et al.*, 1997). These tests are described in more detail in Chapter 5.

2.5 Strengthening of reinforced concrete members in shear

Some research work has been conducted on the use of fibre reinforced composite plates for strengthening structures in shear.

Al-Sulaimani *et al.* (1994) experimentally studied the use of GFRP plates for the shear strengthening of initially shear-cracked concrete with a shear capacity 1.5 times lower than their flexural capacity. A low shear span/beam depth ratio of 2.7 was used, which would have ensured that shear was dominant in the beam behaviour. The shear repair comprised three different systems, with and without the soffit plate in each case. The first repair involved the external bonding of 20 mm wide strips over the side and soffit of the beam at regular intervals throughout each shear span. The second repair utilised the bonding of side plates, throughout each shear span, covering 80% of the beam depth and located centrally in the depth. The third method involved the bonding of a U-shaped jacket covering the sides of the beam and the soffit plate throughout each shear span. The beams repaired with side strips and side plates failed by diagonal tension, with dominant cracks at failure following the cracks initially present in the beams from the preloading stage. Concrete compression failure occurred in the beams with the jackets.

The programme of experimental work by Chajes *et al.* (1994), on small scale specimens, concentrated on GFRP composites as the external reinforcing medium (Chajes *et al.*, 1995a). Increases in flexural and shear capacity of beams 1120 mm in length were examined when tested to failure in four point bending. These small scale beams, which again had no shear reinforcement, were externally strengthened with unidirectional CFRP tow sheets to the basic control beam configuration. To evaluate the effect of composite shear reinforcement, a CFRP sheet was wrapped around the section; again, the extent of this reinforcement along the span is unclear. It was found that the control beam was increased by 158% by adding a single CFRP sheet to the tensile face of the beam. Increases in the load cracking of the concrete and yielding of the internal steel were also noted. In addition to the increase in capacity, a 115% increase in stiffness, a change in failure mode from flexural to shear, and a decrease in ductility were observed. By wrapping the beam with a CFRP sheet, shear failure was prevented and tensile failure of the composite occurred. Finally, by adding a second CFRP sheet to the tensile face, a 292% increase in capacity and a 178% increase in stiffness were achieved. It should be stressed, however, that these large percentages are a function of the initial structural capacities of the beam.

Chajes *et al.* (1995b) tested beams reinforced externally with CFRP plates bonded to their soffit and sides to study flexural and shear behaviours. The fibre orientation in the shear plates was in the vertical direction of the beams only. This orientation was believed to be the reason for the similarity in the load–deflection responses of flexurally strengthened beams with and without external material; the vertical fibres had little effect on the flexural behaviour of the beams. The composite material used by Chajes *et al.* (1995b) was a unidirectional CFRP tow sheet having a dry thickness of 0.11 mm and a tensile modulus of elasticity of 227.37 GPa. The continuous strips were able to control shear crack opening due to their greater axial stiffness, resulting in reduced shear deflection. This result showed that, unlike the flexural soffit reinforcement, a thin sheet covering as much of the concrete as possible will not necessarily produce the greatest improvement in crack control where shear is concerned, but the sheets were able to avoid concrete shear failure, the failure mode observed without the sheets. The tests showed a logical progression of failure modes as more and more external reinforcement was added. There was an increase in capacity of 115% in stiffness, a change in failure mode from flexure to shear and a decrease in ductility. When a further single CFRP sheet was applied to the beam, shear failure was prevented and a flexural failure initiated as a tensile failure of the composite occurred. Finally by adding a second single layer of CFRP sheet to the tensile face a 292% increase in capacity and a 178% increase in stiffness were achieved.

Taljsten (1997) studied the shear force capacity of beams when these had been strengthened by CFRP composites applied to the beams by four different techniques. These were:

- hand-lay-up, by two different systems
- prepreg in combination with vacuum and heat
- vacuum injection.

The results of the four point loaded tests showed, in all cases, a good strengthening effect in shear when the CFRP composites were bonded to the vertical faces of concrete beams. The strengthening effect of almost 300% was achieved and it was possible to reach a value of 100% with an initially completely fractured beam. Generally it was easier to apply the hand-lay-up system and Taljstenn suggested that although the prepreg and vacuum injection methods gave higher material properties than those of the hand-lay method, the site application technique seemed to be more controllable for the hand-lay process.

Hutchinson *et al.* (1997) has described tests that were undertaken at the University of Manitoba to investigate the shear strengthening of scaled models of the Maryland bridge which required shear capacity upgrading in order to carry increased truck loads. The bridge had an arrangement of stirrups which caused spalling off of the concrete cover followed by straightening of the stirrups and sudden failure. CFRP sheets were effective in reducing the tensile force in the stirrups under the same applied shear load. The CFRP plates were clamped to the web of the Tee beams in order to control the outward force in the stirrups within the shear span. This allowed the stirrups to yield and to contribute to a 27% increase in the ultimate shear capacity. Hutchinson showed that diagonal CFRP sheets are more efficient than the horizontal and vertical CFRP sheet combination in reducing the tensile force in the stirrups at the same level of applied shear load.

2.6 Applications of FRP strengthening

Of the applications of FRP strengthening reported in the literature, the majority occur in Switzerland where the concept was first proposed and developed. In these cases, which are considered in more detail by Meier (1995), pultruded carbon fibre/epoxy laminates have been used exclusively. The first reported application was the repair in 1991 of the Ibach Bridge in the canton of Lucerne, for which several prestressing tendons had been severed during the installation of traffic signals. The bridge was repaired with three CFRP sheets of dimensions 150 mm wide by 5000 mm long and of thickness 1.75 mm or 2.0 mm. The total weight of the CFRP used was only 6.2 kg, compared with the 175 kg of steel which would have been required for the repair. In addition, all work was carried out from a mobile platform, eliminating the need for expensive scaffolding. A loading test with an 840 kN vehicle demonstrated that the rehabilitation work had been satisfactory.

The wooden bridge at Sins in Switzerland was stiffened in 1992 to meet increased traffic loading (Meier *et al.*, 1993). Two of the most highly loaded cross beams were strengthened using 1.0 mm thick CFRP laminates. The appearance of the historic structure was unaltered by the strengthening technique. Other CFRP strengthening applications in Switzerland include slab reinforcement around a newly installed lift shaft in the City Hall of Gossau St. Gall, the upgrading of a supermarket roof using laminates 15.5 m in length to allow the removal of a supporting wall, ground floor strengthening of the Rail Terminal in Zurich, and the strengthening of a multistorey car park in Flims. A chimney wall at the nuclear power plant in Leibstadt has also been poststrengthened for wind and seismic loading after the installation of ducts.

Rostasy *et al.* (1992) report the use of GFRP plates at the working joints of the continuous multispan box girder Kattenbusch Bridge in Germany to reduce fatigue stresses in the prestressing tendons and transverse cracking due to thermal restraint. A representative specimen of the joint was tested in the laboratory to verify the technique prior to field application. Ten joints required rehabilitation; eight of these were strengthened with steel plates 10 mm thick, whilst the remaining two utilised GFRP plates 30 mm thick to provide the same area stiffness as the steel plates. The installation of such plates, of which twenty were used at each joint, took place in 1987 and was found to reduce the stress amplitude at the joints by 36% and the crack widths by around 50%.

Greenfield (1995) describes applications of composite strengthening in the United States, in which the integrity of a sewage treatment basin was restored with carbon fibre/epoxy laminates 1.65 mm thick. The laminates were also used to relieve overstress in areas of the basin due to lack of reinforcing steel. The seismic retrofit of bridge columns in California using GFRP jackets has been reviewed by Priestley *et al.* (1992).

An existing roof structure at Kings College Hospital, South London has been strengthened using epoxy-bonded, 1.0 mm thick, 11 m long pultruded CFRP laminates (NCE, 1996). An extra floor was added to the building such that the existing roof was strengthened to meet new floor requirements. The installation took place quickly and conveniently, 2 kg of CFRP being used instead of 60 kg of steel.

Nanni (1995) reported the findings of a visit to Japan to determine the scale of FRC use as external reinforcement. He concluded that a greater number of field applications in Japan in recent years have used thinner FRC sheets than the plates used in Europe, Saudi Arabia and North America. The use of FRC sheets for the structural strengthening of concrete in Japan has addressed problems in bridges, tunnels, car parks and other structures

(Greenfield, 1995). The following five examples of FRC strengthening were cited by Nanni (1995), carbon fibre composites having been used in all cases:

- Strengthening of a cantilever slab of the Hata Bridge along the Kyushu Highway in order to accommodate large parapet walls which caused elevated bending moments due to the higher wind force;
- Increase of the load rating of the Tokando Highway bridge at Hiyoshikura, a reinforced concrete deck supported on steel girders, causing a 30–40% reduction of stress in the internal rebars;
- Arrest of the internal steel reinforcement corrosion of the concrete beams in the waterfront pier at the Wakayama oil refinery;
- Strengthening and stiffening of the concrete lining of the Yoshino Route tunnels on Kyushu Island, necessary due to cracking which arose from unexpected fluctuations in the underground water pressure. No loss of tunnel cross-sectional area occurred and the road remained open during the bonding work;
- Longitudinal strengthening of the sides and soffit of a culvert at the Fujimi Bridge in Tokyo.

Chapter 11 of this book discusses some case histories of plate bonding undertaken in the UK and in Europe and goes into the technologies used in those cases in greater depth than has been possible in this chapter.

2.7 Summary and conclusions of literature review

The review given in this chapter is based on steel and composite plate bonding and has been covered extensively but not exhaustively. It has demonstrated the improvement in structural strength and stiffness brought about by externally bonded material. The worldwide level of interest in the technique reflects its potential benefits and also the current importance placed on economical rehabilitation and upgrading methods. Although the level of experience in the bonding technique of composite plates is limited, the investigations reported in this chapter have gone some way to illustrate its potential and to establish a basic technical understanding of short term and long term behaviour. Despite the growing number of field applications, there remain many material and structural implications that need to be addressed, in particular with regard to long term performance under loads. The procedure for specifying plate and adhesive materials and for obtaining approval for their use has been simplified in the UK by the amalgamation of the individual product approval systems of the Highways Agency, the British Board of Agrément and the County Surveyors' Society, into one all encompassing scheme known as the Highways Authorities Products Approval Scheme (HAPAS), as described in a recent publication by Robery and Innes (1997).

Although this book is not specifically concerned with the shear strengthening of reinforced concrete, some research on this topic has been reported in this chapter using externally applied composite plates or fabrics; this technique also appears to have great potential.

2.8 References

ACI (1973) *Use of Epoxy Compounds with Concrete*, ACI Committee 503, ACI, Detroit.

Adams R D and Wake W C (1984) *Structural Adhesive Joints in Engineering*, Elsevier Applied Science, London.

Al-Sulaimani G J, Sharif A, Basunbul I A, Baluch M H and Ghaleb B N (1994) 'Shear repair for reinforced concrete by fiberglass plate bonding', *ACI Struct J* **91**(4) 458–464.

Arduini M, D'Ambrisi Ambrisi A and Di Tommaso A (1994) 'Shear failure of concrete beams reinforced with FRP plates', in *Infrastructure: New Materials and Methods of Repair*, ed. K Bashan, Proc 3rd Materials Engineering Conf, ASCE, San Diego, 13–16th Nov 1994, pp 123–130.

Arduini M, Di Tommaso A and Manfroni O (1995) 'Fracture mechanisms of concrete beams bonded with composite plates', in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, ed. L Taerwe, E & FN Spon, London, pp 483–491.

Baluch M H, Ziraba Y N, Azad A K, Sharif A M, Al-Sulaimani G J and Basunbul I A (1995) 'Shear strength of plated reinforced concrete beams', *Mag Concrete Res* **47**(173) 369–374.

Bencardino F, Spades G and Swamy R N (1997) 'Design to repair/up-grade RC structures: the key to a successful utilisation of CFRP laminates', *Proc 7th International Conference on Structural Faults and Repair*, University of Edinburgh, July 8th–10th, 1997, Vol 2, pp 183–190.

Chajes M J, Karbhari V M, Mertz D R, Kaliakin V N and Faqiri A (1993) 'Rehabilitation of cracked adjacent concrete box-beam bridges', *Proc Symp Practical Solutions for Bridge Strengthening and Rehabilitation*, NSF, Des Moines, Iowa, 1993, pp 265–274.

Chajes M J, Thomson T A, Finch W W and Januszka T F (1994) 'Flexural strengthening of concrete beams using externally bonded composite materials', *Construct Build Mater* **8**(3) 191–201.

Chajes M J, Thomson T A and Tarantino B (1995a) 'Reinforcement of concrete structures using externally bonded composite materials', in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, ed. L Taerwe, E & FN Spon, London, pp 501–508.

Chajes M J, Thomson T A and Farschman C A (1995b) 'Durability of concrete beams externally reinforced with composite fabrics', *Construct Build Mater* **9**(3) 141–148.

Chajes M J, Finch W W, Januszka T F and Thomson T A (1996) 'Bond and force transfer of composite material plates bonded to concrete', *ACI Struct J* **93**(2) 208–217.

Char M S, Saadamanesh H and Ehsani M R (1994) 'Concrete girders externally prestressed with composite plates', *PCI J* (May–June), pp 40, 51.

Deblois M, Picard A and Beaulieu D (1992) 'Renforcement de poutres en beton armé à l'aide de matériaux composites: études théoretique et expérimentale', in *Advanced Composite Materials in Bridges and Structures*, eds K W Neale and P Labossière, Proc 1st Int Conf ACMBS, Canadian Society for Civil Engineering, Sherbrooke, Quebec, Oct 1992, pp 265–275.

Deuring M (1993) 'Verstärken von Stahlbeton mit gespannten Faserverbundwerkstoffen (Poststrengthening of concrete structures with prestressed advanced composites)', *EMPA Research Report No. 224*, EMPA, Dubendorf, CH-8600, Dubendorf, Switzerland (in German).

Deuring M (1994) 'Brandversuche an nachtraglich verstärken Tragern aus Beton', *EMPA Report No. 148795*, EMPA, Dubendorf, CH-8600, Dubendorf, Switzerland (in German).

Eberline D K, Klaiber F W and Dunker K (1988) 'Bridge strengthening with epoxy-bonded steel plates', *Transport Res Record* (1180) 7–11.

Finch W W, Chajes M J, Mertz D R, Kaliakin V N and Faqiri A (1994) 'Bridge rehabilitation using composite materials', in *Infrastructure: New Materials and Methods of Repair, Proc 3rd Materials Engineering Conf,* ASCE, ed. K Bashan San Diego, 13–16th Nov 1994, pp 1140–1147.

Fleming C J and King G E M (1967) 'The development of structural adhesives for three original uses in South Africa', *Proc RILEM Symp Synthetic Resins in Building Construction,* Paris, Sept 1967, pp 75–92.

Franke L (1986) 'The dimensioning of adhesive-bonded joints in concrete building components', in *Adhesion Between Polymers and Concrete, Proc Int Symp, RILEM,* ed. H R Sasse, Aix-en-Provence, Sept 1986, Chapman and Hall, London, pp 461–473.

Garden H N and Hollaway L C (1997) 'An experimental study of the strengthening of reinforced concrete beams using prestressed carbon composite plates', *Proc 7th International Conference on Structural Faults and Repair,* University of Edinburgh, July 8th – 10th, 1997, Vol 2, pp 191–199.

Garden H N, Hollaway L C, Thorne A M and Parke G A R (1996) 'A parameter study of the strengthening of reinforced concrete beams with bonded composites', in *Bridge Management 3,* eds J E Harding, G A R Parke and M J Ryall, E & FN Spon, London, pp 400–408.

Gilibert Y, Bernasconi J and Colloi C (1976) 'Mésure des déformations et des contraintes engendrées lors d'un essai de cisaillement en traction à la surface de plaque d'acier collées', *Mater Struct* **9**(52) 255– 265.

Greenfield K T (1995) 'Carbon fiber laminates for repair of concrete structures', *Mater Perform* **34**(3) 36–38.

He J H, Pilakoutas K and Waldron P (1997a) 'CFRP plate strengthening of RC beams', *Proc 7th International Conference on Structural Faults and Repair,* University of Edinburgh, July 8th–10th, 1997a, Vol 2, pp 119–127.

He J H, Pilakoutas K and Waldron P (1997b) 'Analysis of externally strengthened R C beams with steel and CFRP plates', *Proc 7th International Conference on Structural Faults and Repair,* University of Edinburgh, July 8th–10th, 1997b, Vol 2, pp 83–92.

Hewlett P C and Shaw J D N (1977) in *Developments in Adhesives*, ed. W C Wake, Applied Science, London, pp 25–75.

Hollaway L C (1993a) *Polymer Composites for Civil and Structural Engineering*, Blackie Academic and Professional, Glasgow, Scotland.

Hollaway L C (1993b) *Use of Advanced Composites in Highway Structures*, Report written for the SERC/DTp/LINK Project, Feb 1993.

Hugenschmidt F (1975) 'Epoxy adhesives for concrete and steel', in *Polymers in Concrete, Proceedings 1st International Congress Polymer Concretes*, 7th May 1975, The Construction Press, London, 1976, pp 195–209.

Hussain M, Sharif A, Basunbul I A, Baluch M H and Al-Sulaimani G J (1995) 'Flexural behaviour of precracked reinforced concrete beams strengthened externally by steel plates', *ACI Struct J* **92**(1) 14–22.

Hutchinson A R (1986) *Durability of Structural Adhesive Joints*, PhD Thesis, University of Dundee.

Hutchinson A R and Rahimi H (1993) 'Behaviour of reinforced concrete beams with externally bonded fibre reinforced plastics', *Proc 5th International Conference on Structural Faults and Repair*, University of Edinburgh, Vol 3, pp 221–228.

Hutchinson A R and Rahimi H (1996) 'Flexural strengthening of concrete beams with externally bonded FRP reinforcement', *Proc 2nd International Conference on Advanced Composite Materials in Bridges and Structures*, Canadian Society for Civil Engineering, Montreal, Quebec, 11–14 Aug 1996.

Hutchinson R, Abdelrahman A and Rizkalla S (1997) 'Shear strengthening using CFRP sheets for a prestressed concrete highway bridge in Manitoba, Canada', *Proceedings of the US–Canada–Europe Workshop in Bridge Engineering – Advanced Rehabilitation, Durable Materials, Non-destructive Evaluation and Management*, eds U Meier and R Betti, EMPA, Dubendorf, July 1997, pp 97–104.

Irwin C A K (1975) *The Strengthening of Concrete Beams by Bonded Steel Plates*, Supplementary Report 160, Transport and Road Research Laboratory, Crowthorne, UK.

Iyer S L (1988) 'Strengthening of reinforced concrete bridges with external reinforcements', *Proc 2nd Workshop Bridge Engineering Research in Progress*, NSF, Reno, Nevada.

Iyer S L, Sivaramakrishnan C and Atmaram S (1989) 'Testing of reinforced concrete bridges for external reinforcement', *Proc Struct Cong*, ASCE, May 1989, pp 116–122.

Jones R and Swamy R N (1995) 'Plate bonding – What can it do to your beams?', *Construc Repair* **9**(3) 43–47.

Jones R, Swamy R N and Salman F A R (1985) 'Structural implications of repairing by epoxy-bonded steel plates', *Proc 2nd International Conference on Structural Faults and Repair*, London, April/May 1985, pp 75–80.

Jones R, Swamy R N and Charif A (1988) 'Plate separation and anchorage of reinforced concrete beams strengthened by epoxy-bonded steel plates', *The Struct Eng* **66**(5) 85–94.

Kaiser H P (1989) 'Strengthening of reinforced concrete with epoxy-bonded carbon-fiber plastics', *Doctoral Thesis*, Diss. ETH, Nr. 8918, ETH Zurich, Ch-8092 Zurich, Switzerland, 1989 (in German).

Karbhari V M and Engineer M (1996) 'Investigation of bond between concrete and composites: use of a peel test', *J Reinforced Plastics Compos* **15**(2) 208–227.

Kinloch A J (1987) *Adhesion and Adhesives; Science and Technology*, Chapman and Hall, London.

Kreigh J D (1976) ‘Arizona slant shear test: a method to determine epoxy bond strength’, *ACI J* **73**(7) 372–373.

Ladner M and Holtgreve K (1989) ‘Verstärken von Stahlbetonbauten mit augeklebten CFK-Lamellan’, *IABSE Symp*, Lisbon, 1989, pp 677–689.

Ladner M and Weder C (1981) *Concrete Structures with Bonded External Reinforcement*, EMPA Report No. 206, EMPA, Dubendorf, CH-8600, Dubendorf, Switzerland.

Ladner M, Pralong J and Weder C (1990) *Geklebte Bewehrung: Bemessung und Erfahrung*, EMPA Report No.116/5, EMPA, Dubendorf, CH-8600, Dubendorf, Switzerland (in German).

Lane J S, Leeming M B, Darby J J and Fashole-Luke P S (1997) ‘Field testing of 18 m post tensioned concrete beams strengthened with CFRP plates’, *7th Int Conf on Structural Faults and Repair 97*, Edinburgh, July 1997, Vol 1 Extending the Life of Bridges, pp 209–214.

Lee H and Neville K (1967) *Handbook of Epoxy Resins*, McGraw-Hill, New York.

Lees W A (1985) ‘The adhesive bond: a new look at the interaction between adherend and adhesive’, *Plastics Rubber Int* **10**(3) 11–15.

Leibengood L D, Darwin D and Dodds R H (1986) ‘Parameters affecting FE analysis of concrete structures, *J Struc Eng Div ASCE* **112** 326–341.

Lerchenthal C H (1967) ‘Bonded sheet metal reinforcement for concrete slabs’, *Proc RILEM Symp Synthetic Resins in Building Construction*, Paris, Sept 1967, pp 165–174.

Lewis A F (1972) ‘Stress endurance limit of lap shear adhesive joints’, *Adhesives Age* **15** 38–40.

Lewis A F, Kinmouth R A and Kreahling R P (1972) ‘Long term strength of structural adhesive joints’, *J Adhesion* **3**(3) 249–257.

L’Hermite R and Bresson J (1967) ‘Beton armé d’armatures collées’, *Proc RILEM Symp Synthetic Resins in Building Construction*, Paris, Sept 1967, pp 175–203.

Macdonald M D (1978) *The Flexural Behaviour of Concrete Beams with Bonded External Reinforcement*, Supplementary Report 415, Transport and Road Research Laboratory, Crowthorne, UK.

Macdonald M D (1981) ‘Strength of bonded shear joints subjected to movement during cure’, *Int J Cement Compos Lightweight Concrete* **3**(4) 267–272.

Macdonald M D (1982) *The Flexural Performance of 3.5 m Concrete Beams with Various Bonded External Reinforcements*, Supplementary Report 728, Transport and Road Research Laboratory, Crowthorne, UK.

Macdonald M D and Calder A J J (1982) ‘Bonded steel plating for strengthening concrete structures’, *Int J Adhesion and Adhesives* **2**(2) 119–127.

Mackie R I and Su N (1993) ‘The effect of ageing and environment on the static and fatigue strength of adhesive joints’, *J Adhesion* **42** 191–207.

Malvar L J, Warren G E and Inaba C (1995) ‘Rehabilitation of navy pier beams with composite sheets’, in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, ed. L Taerwe, E & FN Spon, London, pp 533–540.

Mays G C (1985) ‘Structural applications of adhesives in civil engineering’, *Mater Sci Technol* **1** 937–943.

Mays G C (1993) ‘The use of bonded external reinforcement in bridge strengthening: structural requirements of the adhesive', in *Bridge Management 2*, eds JE Harding, GAR Parke and MJ Ryall, Thomas Telford, London, pp 672–680.

Mays G C and Hutchinson A R (1988) 'Engineering property requirements for structural adhesives', *Proc ICE* **85**(2) 485–501.

Mays G C and Hutchinson A R (1992) *Adhesives in Civil Engineering*, Cambridge University Press, Cambridge.

Meier U (1987) 'Bridge repair with high performance composite materials', *Mater Technik* **15** 125–128 (in French and German).

Meier U (1992) 'Carbon fiber-reinforced polymers: modern materials in bridge engineering', *Struct Eng Int* **1** 7–12.

Meier U (1995) 'Strengthening of structures using carbon fibre/epoxy composites', *Construct Build Mater* **9**(6) 341–351.

Meier U and Kaiser H P (1991) 'Strengthening of structures with CFRP laminates', *Proc Advanced Composite Materials in Civil Engineering Structures*, Mats Div ASCE, Las Vegas, Jan 1991, pp 224–232.

Meier U and Winistorfer A (1995) 'Retrofitting of structures through external bonding of CFRP sheets', in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, ed. L Taerwe, E & FN Spon, London.

Meier U, Deuring M, Meier H and Schwegler G (1992) 'Strengthening of structures with advanced composites', in *Alternative Materials for the Reinforcement and Prestressing of Concrete*, ed. J L Clarke, Blackie Academic and Professional, Glasgow, pp 153–171.

Moustafa S E (1974) 'Ultimate load test of a segmentally constructed prestressed concrete I beam', *J Prestressed Concrete Inst* **19**(4) 54–75.

Naderi M (1985) *Internal Research Report*, Civil Engineering Department, Queen's University, Belfast, Oct 1985.

Naderi M, Clelund D and Long AE (1986) '*In situ* test methods for repaired concrete structures', in *Adhesion Between Polymers and Concrete*, ed. H R Sassa, Proceedings International Symposium RILEM, Aix-en-Provence, Sept 1986, Chapman & Hall, London, pp 707–718.

Nanni A (1995) 'Concrete repair with externally bonded FRP reinforcement', *Concrete Int* **17**(6) 22–26.

NCE (1996) 'Sticking to the task', *New Civil Eng* (11 July) 22.

Peshkam V and Leeming M (1994) 'Application of composites to strengthening of bridges: Project ROBUST', *Proc 19th British Plastics Federation Composites Cong*, Birmingham, UK, 22–23 Nov 1994.

Peterson J (1965) 'Wood beams prestressed with bonded tension elements', *J Struct Div, ASCE* **91**(1) 103–119.

Priestley M J N, Seible F and Fyfe E (1992) 'Column seismic retrofit using fibreglass/epoxy jackets', in *Advanced Composite Materials in Bridges and Structures, Proc 1st Int Conf ACMBS*, eds K W Neale and P Labossière, Canadian Society for Civil Engineering, Sherbrooke, Quebec, Oct 1992, pp 287–298.

Quantrill R J and Hollaway L C (1998) 'The flexural rehabilitation of reinforced concrete beams using prestressed advanced composite plates', *Composite Science and Technology* **58**(8).

Quantrill R J, Hollaway L C, Thorne A M and Parke G A R (1995) 'Preliminary research on the strengthening of reinforced concrete beams using FRP', in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, ed. L Taerwe, E & FN Spon, London, pp 543–550.

Quantrill R J, Hollaway L C and Thorne A M (1996a) 'Part 1. Experimental and analytical investigation of FRP strengthened beam response', *Mag Concrete Res* **48**(177) 331–342.

Quantrill R J, Hollaway L C and Thorne A M (1996b) 'Part 2. Predictions of the maximum plate end stresses of FRP strengthened beams', *Mag Concrete Res* **48**(177) 343–352.

Raithby K D (1980) *External Strengthening of Concrete Bridges with Bonded Steel Plates*, Supplementary Report 612, Transport and Road Research Laboratory, Crowthorne, UK.

Raithby K D (1982) 'Strengthening of concrete bridge decks with epoxy-bonded steel plates', *Int J Adhesion Adhesives* **2**(2) 115–118.

Roberts T M (1989) 'Approximate analysis of shear and normal stress concentrations in the adhesive layer of plated RC beams', *The Struct Eng* **67**(12) 229–233.

Robery P and Innes C (1997) 'Carbon fibre strengthening of concrete structures', *Proc 7th International Conference on Structural Faults and Repair*, University of Edinburgh, July 8th–10th 1997, Vol 1, pp 197–208.

Rostasy F S, Hankers C and Ranisch E H (1992) 'Strengthening of R/C- and P/C-structures with bonded FRP plates', in *Advanced Composite Materials in Bridges and Structures*, eds K W Neale and P Labossière, *Proc 1st Int Conf ACMBS*, Canadian Society for Civil Engineering, Sherbrooke, Quebec, Oct 1992, pp 253–263.

Saadatmanesh H and Ehsani M R (1989) 'Applications of fiber-composites in civil engineering', *Proc Sessions Related to Structural Materials, Structures Cong*, ASCE, 1989, pp 526–535.

Saadatmanesh H and Ehsani M R (1990a) 'Fiber composite plates can strengthen concrete beams', *Concrete Int ACI* **12**(3) 65–71.

Saadatmanesh H and Ehsani M R (1990b) 'Flexural strength of externally reinforced concrete beams', *Proc 1st Materials Engineering Cong*, ASCE, 1990, pp 1152–1161.

Saadatmanesh H and Ehsani M R (1991) 'RC beams strengthened with GFRP plates I: experimental study', *J Struct Eng* **117**(11) 3417–3433.

Santoh N, Kimura H, Enomoto T, Kiuchi T and Kuzubi Y (1983) 'Report on the use of CFCC in prestressed concrete bridges in Japan', *Proc Int Symp Fiber-Reinforced-Plastic Reinforcement for Concrete Structures*, eds A Nanni and C W Dolan, ACI SP-138, ACI, Detroit, pp 895–911.

Sasse H R and Friebrich M (1983) 'Bonding of polymer materials to concrete', *Mater Struct* **16**(94) 293–301.

Schutz R J (1976) 'Epoxy adhesives in prestressed and precast concrete bridge construction', *ACI J* 155–159.

Shaw J D N (1982) 'A review of resins used in construction', *Int J Adhesion Adhesives* **2**(2) 77–83.

Shaw M (1993) 'Strengthening bridges with externally bonded reinforcement', in *Bridge Management 2*, eds J E Harding, G A R Parke and M J Ryall, Thomas Telford, London, pp 651–659.

Shields J (1985) *Adhesives Handbook*, 3rd edn, Newnes-Butterworth, London.

Solomon S K (1976) *A Study of Steel and Concrete Sandwich Construction for Use in Bridge Roadway Decks*, PhD Thesis, University of Dundee, Jan 1976.

Swamy R N and Mukhopadhyaya P (1995) 'Role and effectiveness of non-metallic plates in strengthening and upgrading concrete structures', in *Non-Metallic (FRP)*

Reinforcement for Concrete Structures, ed. L Taerwe, E & FN Spon, London, pp 473–481.

Tabor L J (1978) 'Effective use of epoxy and polyester resins', in *Civil Engineering Structures, CIRIA Report 69*, Construction Industry and Information Association, London, Jan 1978.

Tabor L J (1982) 'Adhesives in construction', *Int J Adhesion Adhesives*, April 1982, 73–76.

Tabor L J (1985) 'Twixt old and new: achieving a bond when casting fresh concrete against hardened concrete', *Proc 2nd Int Conf Structural Faults and Repair*, Institution of Civil Engineers, London, April/May, 1985, pp 57–63.

Taljsten B (1997) 'Strengthening of concrete structures for shear with bonded CFRP-fabrics', *Proceedings of the US-Canada-Europe Workshop on Recent Advances in Bridge Engineering – Advanced Rehabilitation, Durable Materials, Non-destructive Evaluation and Management*, eds U Meier and R Betti, EMPA, Dubendorf, July 1997.

Triantafillou T C and Deskovic N (1991) 'Innovative prestressing with FRP sheets; mechanics of short term behaviour', *J Eng Mech* **117**(7) 1652–1671.

Triantafillou T C and Plevris N (1992) 'Strengthening of RC beams with epoxy-bonded fibre-composite materials', *Mater Struct* **25** 201–211.

Triantafillou T C and Plevris N (1995) 'Reliability analysis of reinforced concrete beams strengthened with CFRP laminates', in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, ed. L Taerwe, E & FN Spon, London, pp 576–583.

Triantafillou T C, Deskovic N and Deuring M (1992) 'Strengthening of concrete structures with prestressed fiber reinforced plastic sheets', *ACI Struct J* **89**(3) 222–235.

Tu L and Kruger D (1996) 'Engineering properties of epoxy resins used as concrete adhesives', *ACI Mater J* **93**(1) 26–35.

Varastehpour H and Hamelin P (1995) 'Structural behaviour of reinforced concrete beams strengthened by epoxy bonded FRP plates', in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, ed. L Taerwe, E & FN Spon, London, pp 559–567.

Vardy A E and Hutchinson A R (1986) 'Some uses of adhesives in civil engineering', *Proceedings, International Conference on Structural Adhesives in Engineering*, Institution of Mechanical Engineers, Bristol University, July 1986, pp 199–204.

Wake W C (1982) *Adhesion and the Formulation of Adhesives*, 2nd edn, Applied Science Publishers, London.

Wall J S, Shrive N G and Gamble B R (1986) 'Testing of bond between fresh and hardened concrete', in *Adhesion Between Polymers and Concrete, Proc Int Symp, RILEM*, Aix-en-Provence, Sept 1986, ed. H R Sasse, Chapman and Hall, London, pp 335–344.

Wight R G, Green M F and Erki M A (1995) 'Post-strengthening concrete beams with prestressed FRP sheets', in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, ed. L Taerwe, E & FN Spon, London, pp 568–575.

3
Materials

A R HUTCHINSON AND J QUINN

3.1 Adhesive bonded connections

The primary purpose of a connection is to transfer load between two components. The ability to join components comprising dissimilar materials efficiently using adhesives represents one of the key motivations for the whole concept of strengthening using the plate bonding technique. The most commonly used generic type of adhesive used for this purpose is an epoxy.

3.1.1 General considerations

The basic requirements for the creation of a satisfactory bonded joint are:

- selection of a suitable adhesive
- adequate preparation of the adherend surfaces
- appropriate design of the joint
- controlled fabrication of the joint itself.

Some form of postbonding quality assurance is also desirable.

Conceptually, adhesives represent natural candidates for joining FRP materials. This is because the resin matrix which is used as a binder in composite materials is itself an organic adhesive, and adhesives which are then used to join composite materials together may themselves be very similar in terms of their chemical composition and mechanical properties. For instance, 'like' is used to join 'like' when bonding an epoxy matrix composite with an epoxy adhesive. The main difference here is the lower curing temperature (say 15 °C) used to cure the adhesive, compared to the rather higher temperature (say 120–180 °C) used to cure the fibre reinforced vinyl ester or epoxy matrix resin.

There are two particular problems associated with adhesive bonding of FRP materials:

- attachment to the surface of a layered material
- the surface may be contaminated with mould release agents.

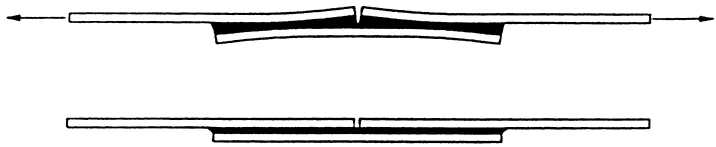


Figure 3.1 Bondline strain variations in a single cover butt shear joint under load

To these must be added the problems associated with adhesive bonding of concrete:

- attachment to the surface of a relatively weak and brittle material
- the surface may be cracked
- the surface may be cement rich and contaminated
- concrete is highly alkaline and may have a high moisture content.

Wherever possible, joint designs should limit the problems associated with low tensile strength of the concrete, low through-thickness strength and potentially weak surface layers of fibre reinforced plastic (FRP) materials. Thus careful joint design is required to minimise cleavage forces acting on bonded configurations and bond interfaces. Adequate surface preparation is also vital and contaminants such as mould release agents and dust must be removed from all surfaces prior to the application of a suitable adhesive.

3.1.2 Characteristics of bonded joints

The transfer of load between two components through an adhesive layer results in bondline stress (strain) variations. This is illustrated in Fig. 3.1 for a single cover butt shear joint. It is apparent that the adhesive and the interfaces are loaded in shear. Such a joint also tends to peel apart as shown, because the line of action of the loads is not coincident. Peel forces can be very high and bonded joint design should embrace those geometries which minimise the generation of peel forces.

3.2 Composite materials

3.2.1 Composition

The primary materials in a composite are the reinforcement fibre and the polymer matrix. Other materials are incorporated in the composite but they are of less significance in terms of both effect on cost and effect on properties. Although the term polymer composites includes both thermosetting and thermoplastic resins, we are only concerned with the former in this book.

The most commonly used thermosetting resins in composites are polyester, urethane methacrylate, vinylester, epoxy and phenolic. They are isotropic materials which allow load transfer between the fibres, but they perform several other duties. The matrix protects notch-sensitive fibres from abrasion and it forms a protective barrier between the fibres and the environment, thus preventing attack from moisture, chemicals and oxidation. It also plays an important role in providing shear, transverse tensile and compression properties. The thermomechanical performance of the composite is also governed by the matrix performance.

Reinforcement fibres fall into three main families of glass, aramid and carbon. There are other fibres but they are relatively insignificant. The most important property of the fibres is their elastic modulus, and the fibres must be significantly stiffer than the matrix which allows them to carry most of the stress. Consequently they must also be of high strength. Reinforcements are available in a variety of configurations of which there are three main categories:

- unidirectional, in which all the fibres lie in one direction.
- bidirectional, in which the fibres lie at 90° to one another. This is achieved either by the use of woven fabric, non-woven fabric or by the use of separate layers of fibres each unidirectional but successively laid at 90°.
- random, in which the fibres are randomly distributed and are in-plane.

The ROBUST Project used unidirectional reinforcement.

3.2.1.1 Glass fibres

As a broad generalisation, glass fibres can be categorised into two sets. Those with a modulus around 70 GPa and with strengths after processing in the range 1000–2000 MPa (i.e. E, A, C, E-CR), and those with a modulus around 85 GPa with strengths after processing in the range 2000–3000 MPa (i.e. R, S and AR). These fibre types and their principal uses are shown in Table 3.1.

The density of glass fibre is about 2500 kg m^{-3}, which is high in comparison to other reinforcing fibres but, by metallic standards, is very low (aluminium has a density of about 2800 kg m^{-3} and steel 7800 kg m^{-3}). Glass fibres are the most commonly used reinforcing fibres and there are several very good reasons for this. They have good properties both in an absolute sense and relative to weight. They also have very good processing characteristics and they are inexpensive.

Table 3.1 Types of glass fibre and principal uses

Fibre type	Principal use
E	Standard reinforcement, low alkali content (<1%)
A	High alkali content (10–15%) but inferior properties to type E
C	Superior corrosion resistance to type E, often used in the form of a surface veil or tissue
E-CR	Boron-free, good acid corrosion resistance, with similar properties to type E
R, S	Better mechanical properties than type E, used for higher performance applications
AR	Alkali-resistant, used for reinforcement of cement

The processing characteristics of particular types of glass fibre have been modified and optimised over many years to achieve the required performance, such as choppability, low static buildup and conformance to complex shape, and resin compatibility requirements such as fast wet out, good fibre/matrix adhesion, and so on.

Glass fibres and composites made with them are to a certain extent prone to moisture-induced degradation, particularly at significant sustained strain levels. Surface corrosion of fibres and the resultant reduction in fibre/matrix adhesion is known as stress-corrosion cracking.

3.2.1.2 Carbon fibres

Carbon fibres are the predominant reinforcement used to achieve high stiffness and high strength. The term carbon fibre (graphite fibres in the USA) covers a whole family of materials which encompass a large range of strengths and stiffness. The density of carbon fibre is of the order of 1900 kg m^{-3}. Typical fibre moduli may be 230–300 GPa, whilst strengths after processing are in the range of 3000–5000 MPa.

Carbon fibre is most commonly produced from a precursor of polyacrylonitrile (PAN) fibre which is processed by first stretching it to achieve a high degree of molecular orientation. It is then stabilised in an oxidising atmosphere while held under tension. The fibres are then subjected to a carbonising regime at a temperature in the range 1000–3500 °C. The degree of carbonisation determines such properties as elastic modulus, density and electrical conductivity. As an alternative to the use of PAN, routes via the use of pitch and rayon have been successfully utilised and such fibres are commercially available. These fibres tend to be of lower performance than PAN-based fibres; they are also cheaper due to their use of a lower cost precursor.

3.2.1.3 Aramid fibres

Aramid fibres are very tough organic synthetic fibres generally characterised as having reasonably high strengths up to 3000 MPa, moduli in the range 60–120 GPa, and a very low density (around 1400 kg m^{-3}). Composites made with aramid fibres fit well into a gap in the range of stress/strain curves left by the family of carbon fibres at one extreme and glass fibres at the other.

Aramid fibres are fire resistant and perform well at high temperatures. They are insulators of both electricity and heat and are resistant to organic solvents, fuels and lubricants. A major distinction of aramid fibres is that they are highly tenacious in the non-composite form and do not behave in a brittle manner as do both carbon and glass fibres.

The tensile stress/strain curve of aramid fibres is essentially linear to failure. They have two distinct categories; those in which their elastic modulus is about the same as glass fibre, typically 60–70 GPa, and those with a modulus at about twice this level. Kevlar 29 falls into the first category and Kevlar 49 into the higher modulus category. It is generally the higher modulus material which finds use in composites, but the lower modulus aramids do have applications in composites in those circumstances where high strain-to-failure or high work-to-failure are required. The specific performance of aramids is their primary advantage, that is their strength/weight and stiffness/weight ratios. The density of aramids exhibits advantage over many carbon and glass fibres if either specific strength or specific stiffness is the selection criterion. Some aramids have relatively very low compressive strength.

3.2.2 Manufacturing routes

The manufacturing options available are diverse and have been developed to suit the wide variety of production parameters encountered. They may be classified as shown in Table 3.2.

A full discussion of these options and typical properties of composites manufactured using these processes is given by Hollaway (1993). The processes used in the ROBUST Project were autoclave/vacuum bag and pultrusion.

3.2.2.1 Vacuum bag/autoclave moulding

Layers of prepreg (resin impregnated fibre reinforcement) are applied to a mould and rolled. A rubber or nylon sheet or bag is placed over the lay-up and the air is removed by means of a vacuum pump. The mould is then placed in an oven where heat (typically 80–200 °C) is applied, or into an autoclave which applies both heat and pressure (typically 7 bar).

Table 3.2 Classification of composite manufacturing options

Class	Process
Open lay-up	Hand lay-up Spray up
Intermediate	Cold press Resin transfer moulding (RTM)/resin injection Autoclave/vacuum bag
Compression	Hot press
Continuous	Pultrusion Continuous sheet
Winding processes	Filament winding

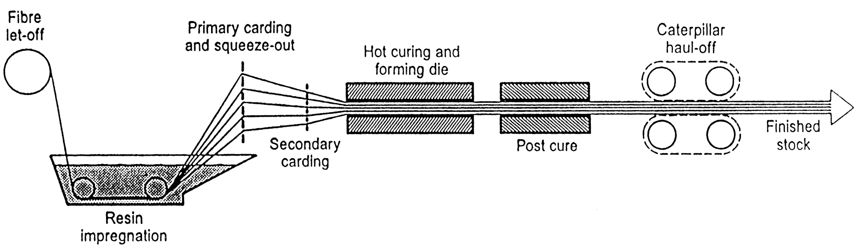


Figure 3.2 The pultrusion process

As an alternative to prepreg, dry reinforcement may be used which is then impregnated with resin on the mould. A cold cure resin system may be used with this method, that is the resin mix contains an accelerator as well as a catalyst. This is a useful option if a heat source is not available. The process is slow and laborious but produces excellent properties.

3.2.2.2 Pultrusion

Pultrusion is a continuous process which allows the production of straight, constant section profiles. Reinforcement is pulled through a device (typically a bath) which applies the resin. The wetted fibre is then pulled on through a heated steel die which is the shape of the section to be produced. The die is heated to about 150 °C which causes the resin to react, gel and cure. When the profile exits from the die it has already achieved a high degree of cure. The profile is pulled by either reciprocating pullers or a caterpillar haul-off and it is then automatically sawn to length. The process is illustrated schematically in Fig. 3.2. Resin systems must be highly reactive to cure in the available time. Machine speed, die temperature and resin reactivity are parameters which interact and must be balanced. Release agents are commonly dissolved in the resin and these migrate to the surface during cure in order to prevent the artefact from sticking to the surface of the forming dies. As the process is 'hot cure', an accelerator is not required.

Most of the reinforcement configurations may be used but in general pultrusions are dominated by the use of unidirectional reinforcement, which lends itself most appropriately to the process and gives maximum strength and stiffness in the axial direction of the profile. Fibre volumes of up to 65% are achievable with unidirectionally aligned fibres. A peel-ply fabric may be incorporated on the outermost surface(s) to provide both protection and a sacrificial surface treatment (see Section 3.4.5).

3.2.3 Manufacturing quality

It is sometimes argued that composites tend to be variable in nature due to the manner in which they are manufactured. This is a broad generalisation which does not apply to several composite types including prepreg mouldings and pultrusions, particularly those in which the reinforcement is unidirectional. These are the types of material used in the ROBUST Project and these materials can be expected to be highly consistent.

The performance of joints made with composite materials bonded *in situ* is dependent upon the quality of the supplied materials, both composite and adhesive, and the manner in which bonding is carried out. The latter is more likely to generate problems due to such factors as variations in temperature and humidity, and the likelihood of occurrence of dust, moisture, oils, and so on.

It is essential in the adhesive bonding process that a 'method of work' is used. This must take into account the possible working conditions which may occur and state the required procedure for each set of conditions. It must also state the conditions under which adhesive bonding may not take place.

3.2.4 Typical properties

Composite materials are not homogeneous. Their properties are dependent on many factors, the most important of which are the type of fibre, quantity of fibre (as volume fraction) and the configuration of the reinforcement. They are generally completely elastic up to failure and exhibit neither a yield point nor a region of plasticity. They tend to have low strain to failure (less than 3%). The resulting area under the stress/strain curve, which represents the work done to failure, is relatively small when compared to many metals.

The properties of composites are dependent on the properties of both the fibre and the matrix, the proportion of each and the configuration of the fibres. If all the fibres are aligned in one direction then the composite is

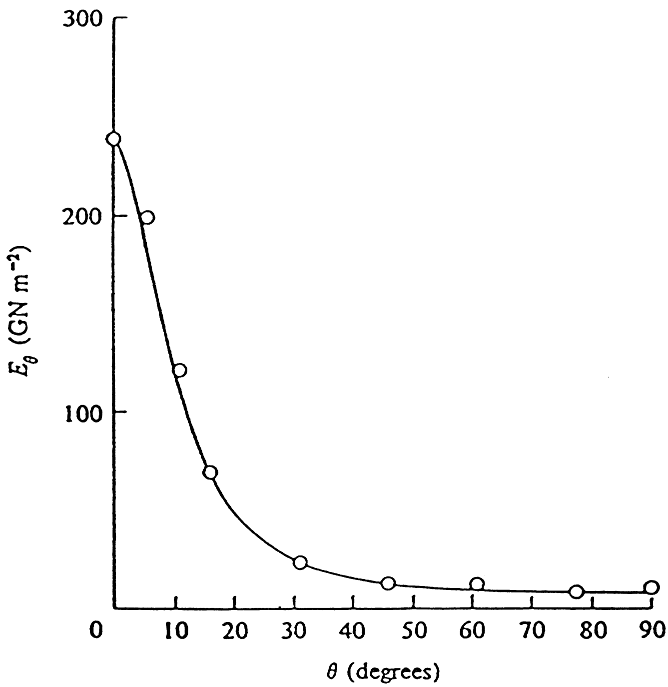


Figure 3.3 Theoretical variation in tensile modulus with the angle of load relative to the principal fibre direction (unidirectional carbon fibre reinforced plastic (UD CFRP), fibre volume fraction $V_f = 0.5$).

relatively stiff and strong in that direction, but in the transverse direction it has low modulus and low strength. When a unidirectional composite is tested at a small angle from the fibre axis, there is a considerable reduction in strength. A similar but less significant effect occurs with the tensile modulus, as shown in Fig. 3.3.

3.2.4.1 Strength and stiffness

3.2.4.1.1 Glass fibre reinforced polymer (GFRP)

E glass fibres, which have a modulus of about 70 GPa, produce composites with modest moduli. In the case of unidirectional fibres and the highest typical fibre volume fraction of 0.65, a composite has a modulus of about 45 GPa and a strength of around 1300 MPa. At right angles to this, in the transverse direction, the modulus approaches that of the resin itself at about 4 GPa and the strength would be 50–100 MPa. The unidirectional composites used in the ROBUST Project, manufactured using the vacuum bag process with prepreg material in an epoxy matrix, had the following properties:

- longitudinal tensile modulus: 36 GPa
- longitudinal tensile strength: 750 MPa
- elongation at break: 3.1%.

Bidirectional E glass laminates have a typical fibre volume fraction of about 0.4 and a tensile modulus at that volume fraction of about 14 GPa. Random laminates (e.g. chopped strand mat) have a typical fibre volume fraction of about 0.2 and a tensile modulus at that volume fraction of about 9 GPa. The use of S2 or R glass improves the composite modulus to about 60 GPa for unidirectional and 20 GPa for woven fabric (bidirectional) constructions. This is at some monetary disadvantage. They are both more expensive than E glass and they are only available in a fairly limited range of material types and resin compatibilities. Probably the most important virtue of S2 and R glass is their high strength, which is considerably higher than E glass.

3.2.4.1.2 Carbon fibre reinforced plastic (CFRP)

The dominant carbon fibres in current use (typically Toray T700) have a tensile modulus of about 230 GPa, a tensile strength of around 5000 MPa and a strain-to-failure of 2%. Unidirectional composites produced from them in either an epoxy or vinylester matrix have the following typical properties:

- longitudinal tensile modulus: 155–165 GPa
- longitudinal tensile strength: 2500–3000 MPa
- elongation at break: 1.2–1.3%

Carbon fibres are available which will give a tensile modulus of about 250 GPa in a unidirectional composite, comparing very favourably with steel at about 210 GPa. However, as this composite is unidirectional, it has extremely low modulus in the transverse direction. The principal attributes of carbon fibre composites are their very high specific stiffness (the ratio of modulus/density), excellent fatigue and environmental resistance.

Currently there are various pultruded CFRP plates available commercially for plate bonding applications. The pultruded plates used in the ROBUST Project, as well as the plates manufactured with prepreg materials, possessed a modulus of about 130 GPa and a strength of 1500 MPa. Pultruded plates now available from other sources typically exploit fibres with superior properties such as Toray T700, resulting in composites with the properties shown above. A financial penalty has to be paid for materials exhibiting properties significantly in excess of these; the strain-to-failure of composites made with them will also be reduced significantly.

3.2.4.1.3 Aramid fibre reinforced plastic (AFRP)

Unidirectional aramid composites have high tensile strength (1200–1400 MPa) and a very low density. This high specific tensile strength is an important attribute which makes them particularly suited to use as tension members. Some aramids exhibit a relatively low compressive yield strength of about 230 MPa. Thus composites using these fibres must be carefully designed, particularly for compression or bending. This makes them particularly suited to use in tension member applications, but often not suited to bending applications. It is usually the high modulus variants of aramids (e.g. Kevlar 49) which are most commonly used as composite reinforcement, conferring a tensile modulus for unidirectional composites of about 75 GPa. This is very similar to aluminium; however, as this is a unidirectional composite the associated transverse modulus is only about 5 GPa.

The tensile modulus of unidirectional and bidirectional aramid composites represents a reasonable compromise between the low modulus glass fibre composites and the much higher carbon fibre composites. Similarly the tensile strength of aramid composites is a compromise between E glass and carbon fibre composites.

3.2.4.2 Fatigue

E glass composites have a relatively low modulus which often results in them experiencing strains which approach the cracking strain of the matrix. This allows a fatigue process to occur which may result in a reduced fatigue life. It is therefore prudent to use a resin system with high strain-to-failure. Therefore as a general rule isophthalic resin is better than the orthophthalic resin and vinylester resin is significantly better than both.

The fatigue performance of carbon fibre composites is generally far superior to both metals and other composites. This is particularly the case for unidirectional composites loaded in the fibre direction, where they are relatively insensitive to tension fatigue damage even at very high stress levels. This does not necessarily apply to all constructions. The fatigue life is reduced if the performance of the matrix determines performance such as $\pm 45°$ lay-up composites or when the stress is transverse to the fibre direction.

The fatigue behaviour of aramids can be excellent, but under other circumstances is very poor. For instance, unidirectional Kevlar 49 composites in tension/tension fatigue are superior to S glass and E glass composites and 2024-T3 aluminium; only unidirectional carbon composites are superior. However in flexural fatigue, at a fairly low number of cycles, they have poorer resistance than E glass due to the innate poor static flexural performance of Kevlar 49.

3.2.4.3 Creep and stress rupture

Unidirectional composites subjected to tensile load resist long term creep very well. The total elastic performance of the fibres dominates the behaviour of the composite, but, when under compression or 'off-axis', then the matrix properties take on a more significant role. Polymers are viscoelastic materials and they deflect continuously with time under load. Under these circumstances, and particularly at elevated temperature, the creep performance of the composite requires particular consideration.

The most creep-resistant construction is unidirectional reinforcement, followed by 0,90° non-woven construction. Woven construction of 0,90° fibres has the disadvantage that the fibres tend to straighten out, thus increasing creep. Laminates with a random fibre array are the least resistant to creep, but this may be alleviated to some extent by the use of a high volume fraction of fibre.

The creep performance of carbon fibre composites in the fibre direction is comparable with 'low relaxation' steel and is significantly better than 'standard steel'. In spite of their high inherent tensile strength, and even in unidirectional configurations, aramid fibre composites exhibit creep rates generally very much higher than glass or carbon composites.

3.2.4.4 Thermal expansion coefficients

The thermal expansion of composites is dependent on several factors; the type of reinforcement, type of matrix, geometry of reinforcement and volume fraction. Table 3.3 shows the thermal expansion coefficients for a variety of composites. The two negative values are a result of the negative fibre expansion coefficients which are exhibited by carbon and aramid

Table 3.3 Typical thermal expansion coefficients ($\times 10^{-6}$ °C)

Fibre/matrix	0°	90°	0,90°
E glass/polyester	8.6	14	9.8
Carbon T300/epoxy	−0.3	28	1.9
Aramid K49/epoxy	−4	79	3.2

Note: volume fraction of fibres = 0.65.

fibres. It is possible to tailor the expansion coefficient of a composite to a specific requirement, for example to match that of aluminium, or even to be zero over a given temperature range.

3.2.5 Handling and machining

Composite plates used for externally bonded reinforcement have a high proportion of the fibre aligned with the axis of the plate. This implies high strength and stiffness in that direction but, in the transverse direction (across the plate), it will be relatively weak and prone to cracking if mishandled. Thus care must be taken, for example when cutting to length to ensure that longitudinal cracks are not started. Although composite materials may be machined with hand tools such as pistol drills and hacksaws, they may produce cracking in composites which have a large percentage of unidirectional fibres. Sawing to length, for instance, is often better achieved using diamond-tipped blades which have a less destructive tendency than, say, a hacksaw. Further details are given by Hutchinson (1997).

Composite reinforcing plates are sufficiently flexible to allow them to be coiled to a diameter of 1–2 m which makes them easy to transport. However the coiling process stores a significant amount of energy which could be dangerous if released in an uncontrolled manner. Care must therefore be taken when removing the coiling constraints.

3.3 Adhesive materials

3.3.1 General considerations

The purpose of the adhesive is to produce a continuous bond between fibre reinforced polymer (FRP) and concrete to ensure that full composite action is developed by the transfer of shear stress across the thickness of the adhesive layer. For this to occur, an excellent degree of adhesion to the surfaces involved must be achieved and sustained. Experience has shown that the best chance of success is likely to be achieved by using two-part cold curing paste epoxy adhesives which have been specially developed for use in the construction industry. Mays and Hutchinson (1988, 1992) identified the principal requirements for bonding steel to concrete and these are very similar for the case of bonding FRP to concrete. In summary, the adhesive requirements are that:

- It should exhibit adequate adhesion to the materials involved (in this case, concrete and FRP).
- A two-part epoxy resin with a polyamine-based hardener should be used, which exhibits good moisture resistance and resistance to creep.

- The glass transition temperature of the adhesive, T_g, should be at least 40 °C.
- The flexural modulus of the material should fall within the range of 2–10 GPa at 20 °C. Generally held views are that the lower boundary of this range might be reduced to 1 GPa.
- The bulk shear and tensile strength at 20 °C should be ≥ 12 MPa.
- The minimum shear strength at 20 °C, measured by the thick adherend shear test (TAST), should be 18 MPa.
- The mode I fracture toughness (K_{1c}) should be ≥ 0.5 MN m$^{-3/2}$.
- The equilibrium water content (M_∞) should not exceed 3% by weight after immersion in distilled water at 20 °C. The coefficient of permeability should not exceed 5×10^{-14} m^2 s^{-1}.
- It should possess gap-filling properties, be thixotropic, and be suitable for application to vertical and overhead surfaces.
- It should not be sensitive to the alkaline nature of concrete and its potential effect on the durability of joints.
- It should not be unduly sensitive to variations in the quality or moisture content of prepared surfaces.
- The working characteristics of the material should enable an adequate joint quality to be achieved with respect to mixing, application and curing.

A further item to add to this list is that the adhesive should exhibit sufficient tack or ‘grab’ to enable FRP materials to be attached to overhead or vertical surfaces without the need for temporary fixings whilst the adhesive cures.

The mechanical and thermal properties of an adhesive should also be considered in relation to those of concrete and FRP materials. The effects of environmental and other service conditions on the adhesive material and on the behaviour of bonded joints, must also be considered carefully (see Section 3.3.2 and Chapter 6).

The detailed effect of the mechanical properties of the adhesive on the overall performance of externally strengthened beams is not yet fully understood. Whilst bonding systems have been, and continue to be, used successfully, the optimum properties have not been identified. For example, relatively ductile adhesives are usually preferred for joining FRP materials (e.g. Hutchinson, 1997) in order to minimise surface stress concentrations. However, to date high modulus untoughened construction epoxies have been used for plate bonding which are characterised by their relatively low ductility. Intuitively, more flexibilised products would be preferred which possess superior toughness and increased strain to failure – provided that their other desirable properties are not compromised.

3.3.2 Epoxy adhesives

Epoxies represent an important class of thermosetting adhesives which have been available commercially since the 1940s. As structural adhesives, they are the most widely used and accepted. Other thermosetting adhesives include polyesters and particular variants of acrylics and polyurethanes. A detailed consideration of adhesives and their properties is given by Mays and Hutchinson (1992).

Epoxy adhesives can be formulated in a variety of forms to provide a broad range of application characteristics and mechanical properties when cured. Commercial formulations are, in general, complex and are sophisticated blends of many components, the most important being the resin. One of a range of different types of hardener is added to the base resin, together with fillers, toughening agents, plasticizers, diluents, surfactants, antioxidants and other materials. The choice of hardener results in distinction between the adhesives which cure either at ambient or at elevated temperature. However as a rule of thumb, the rate of chemical reaction is doubled for every 8 °C rise in temperature. The blending of the base resin with a variety of materials indicates that the possibilities for altering the properties and characteristics of an epoxy adhesive are numerous. On the negative side it should be said that blending with a variety of additives tends to make any adhesive more expensive than its unmodified counterpart.

Epoxy resins have several advantages over other polymers as adhesive agents for civil engineering use, namely:

- High surface activity and good wetting properties for a variety of substrates.
- May be formulated to have a long open time (the time between application and closing of the joint).
- High cured cohesive strength; joint failure may be dictated by adherend strength (particularly with concrete substrates).
- May be toughened by the inclusion of a dispersed rubbery phase.
- Lack of by-products from curing reaction minimises shrinkage and allows the bonding of large areas with only contact pressure.
- Low shrinkage compared with polyesters, acrylics and vinyl types; hence, residual bondline strain in cured joints is reduced.
- Low creep and superior strength retention under sustained load.
- Can be made thixotropic for application to vertical surfaces.
- Able to accommodate irregular or thick bondlines (e.g. concrete adherends).
- May be modified by (a) selection of base resin and hardener; (b) addition of other polymers; (c) addition of surfactants, fillers and other modifiers.

Typically they are suitable for service operating temperatures in the range −60 °C to +60 °C.

The properties of Sikadur 31PBA, the filled epoxy adhesive used in the ROBUST system of CFRP plate bonding, are discussed in Section 3.3.5 and in Chapter 11.

3.3.2.1 Resins

Epoxy resins are traditionally based upon the reaction of epichlorohydrin on bis-phenol A, to give a liquid compound of linear molecules terminating with epoxy groups and having secondary hydroxyl groups occurring at regular intervals. The resulting base resin is known as the diglycidyl ether of bis-phenol A (DGEBA) and is commonly encountered in two-part room temperature cure epoxy adhesives used in construction. The adhesive properties of epoxies are obtained by polymerisation using a cross-linking agent, or hardener, to form a tough three-dimensional polymer network.

3.3.2.2 Hardeners

Aliphatic polyamines are one of the most commonly used hardeners for ambient temperature curing epoxy adhesives, conferring a rigid structure with good resistance to chemicals, solvents and water. A common modification which is performed commercially to improve the performance selectively is to react a glycidyl ether resin with an excess of amine groups to produce a resinous amine adduct hardener. These have advantages over unmodified versions in that they result in more convenient mixing ratios, are less hazardous to handle and can exhibit reduced moisture sensitivity. Sikadur 31PBA, the epoxy adhesive used as part of the ROBUST system of CFRP plate bonding, is cured with an aliphatic polyamine adduct.

3.3.2.3 Epoxy additives

- Fillers: in practice most epoxy resin systems have fillers incorporated, often simply to reduce cost although they may also assist in gap-filling, reduction of creep, reduction of exotherm, corrosion inhibition and fire retardation. They will also alter the physical and mechanical properties of the adhesive. Construction resins in particular often include a large volume fraction of sand or silica.
- Diluents: these materials reduce viscosity of the freshly mixed adhesive to offset the effect of the filler. Pot life, flexibility and even the T_g of the cured adhesive can also be altered.
- Flexibilisers: these may be used to improve impact resistance, peel strength and ductility.

- Tougheners: unmodified epoxies tend to be relatively brittle when cleavage or peel forces are imposed. Different types of toughening agents can be added to the polymer structure to improve fracture energy, G_1 and therefore the fracture toughness, K_1.
- Adhesion promoters: these are sometimes incorporated, both to increase the filler–polymer adhesion and to enhance resin adhesion to surfaces such as glass or metals.

3.3.3 Primers and coupling agents

With some bonding surfaces the application of an adhesive-compatible primer coating may be desirable. In steel plate bonding applications it is virtually essential to prime the steel in order to generate a reliable and reproducible surface which confers good bond stability to an adhesive in the presence of moisture. No such primers are usually necessary for bonding FRP materials. Conceptually it might be desirable to prime cracked or porous surfaces such as concrete to generate suitable conditions for the application of a relatively viscous adhesive. In fact, the use of primers for concrete is rarely reported in structural adhesive bonding unless it is suspected that the alkaline nature of the substrate will result in degradation of the adhesive. One of the advantages of using a well-formulated epoxy adhesive is that it should be tolerant of such alkalinity, and will in fact seal and adhere to a concrete surface satisfactorily.

3.3.4 Properties of adhesives

The engineer will be concerned about the behaviour and performance of the selected adhesive from the time it is purchased from the manufacturer, through the mixing, application and curing phases to its properties in the hardened state within a joint over the intended design life. Thus the properties of interest in approximate chronological order are likely to include:

- unmixed – shelf life
- freshly mixed – viscosity, usable life, wetting ability, joint open time
- during cure – rate of strength development
- hardened – strength and stress/strain characteristics, fracture toughness, temperature resistance, moisture resistance, creep, fatigue.

3.3.4.1 Shelf life

This is the period for which the unmixed components may be stored without undergoing significant deterioration. Most materials have a recommended shelf life of less than a year although this can generally be extended by storage at temperatures around 5 °C. Guidance from the adhesive manufacturer should be sought for storage on site under particular conditions.

3.3.4.2 Viscosity

The viscosity of adhesive materials can vary markedly with temperature, affecting workability, application quality and satisfactory wetting of the adherend surfaces. The property of thixotropy aids wetting of the substrate during spreading, in adhesives which otherwise require a high viscosity. For relatively low ambient working temperatures it may be possible to obtain adhesives of a suitable viscosity from the manufacturer (e.g. summer and winter grades).

3.3.4.3 Usable life

Where two component adhesives are mixed together in a can or container, there is a finite working life or pot life. This is related to the reactivity of the adhesive material itself, the mixed volume and the ambient temperature. The working life is extended at low temperatures and decreases at high temperatures. It should be noted that the chemical reaction is exothermic and large volumes of mixed adhesive can become very hot. A typical usable life might be in the range of 30–60 min.

3.3.4.4 Wetting ability

The ability of an adhesive to wet out a substrate surface is fundamental to obtaining adhesion (see Section 3.4.2). Adhesives applied towards the end of their usable life tend to lack wetting ability.

3.3.4.5 Joint open time

This starts when the adhesive has been applied to the concrete, or FRP, or both surfaces. It represents the time limit during which the FRP should be applied to the concrete surface, or else the strength of the bond may be reduced. This is a function of the degree of hardening and reaction with the atmosphere itself. A typical open time might be of the order of 30 min.

3.3.4.6 Rate of strength development during cure

The cure time depends upon the type of adhesive and the ambient temperature. The majority of cold curing epoxy formulations take 6–12 h to achieve a satisfactory degree of cure for handling purposes, with full cure being attained in 24 h or so (see also Section 3.5.6).

3.3.5 Mechanical characteristics

The properties of adhesive polymers are generally determined by their internal structure, but the blend of components in many commercial formulations prevents simple relationships from being drawn. A comparison of the tensile stress/strain behaviour of a range of epoxies is given in Fig. 3.4, whilst typical properties and characteristics are collected in Table 3.4. The table indicates the properties of Sikadur 31PBA, the adhesive used in the ROBUST system of strengthening. A further discussion of the properties of this and other plate bonding adhesives is reserved for Chapter 11.

Adhesives possess a relatively low modulus which decreases with increasing temperature. For epoxies, Young's modulus is typically between 1–10 GPa. Variations in temperature can transform materials which are tough and strong at 20 °C to ones which are soft and weak at 100 °C. The glass transition temperature, T_g, denotes a marked change in the mechanical properties of a polymer. Above T_g the material will be rubbery, and below it will be glass-like and stiff (Fig. 3.5). The T_gs of commercial building sealants are around −40 °C, whereas those of ambient temperature curing epoxies are around 40–50 °C. By warm- or heat-curing adhesives, their T_g values are increased.

Organic polymers all absorb moisture and one effect is to plasticise the adhesive itself. This modifies its response to mechanical deformation in a manner analogous to a lowering of the T_g. Such effects are reversible, depending upon proximity to moisture in liquid or vapour form. Thus water and heat have similar effects and this is illustrated in Fig. 3.6 and 3.7 for the

Table 3.4 Typical properties and characteristics of epoxy adhesives (2-part, cold cure)

Property (at 20 °C)	Range	Sikadur 31PBA
Shear strength (MPa)	15–35	28
Tensile strength (MPa)	20–40	30
Tensile modulus (GPa)	1–10	7
Tensile strain to failure (%)	1–4	0.7
Fracture energy (kJ m^{-2})	0.2–1.0	0.4
Glass transition temperature (°C)	40–60	58
Coefficient of thermal expansion (10^{-6} °C^{-1})	30	30
Characteristics		
Creep resistance	Excellent	
Moisture resistance	Excellent	
Heat resistance	Good	
Cold or hot cure	Cold/hot	
Cure time	Medium/long	
Gap filling	Yes	

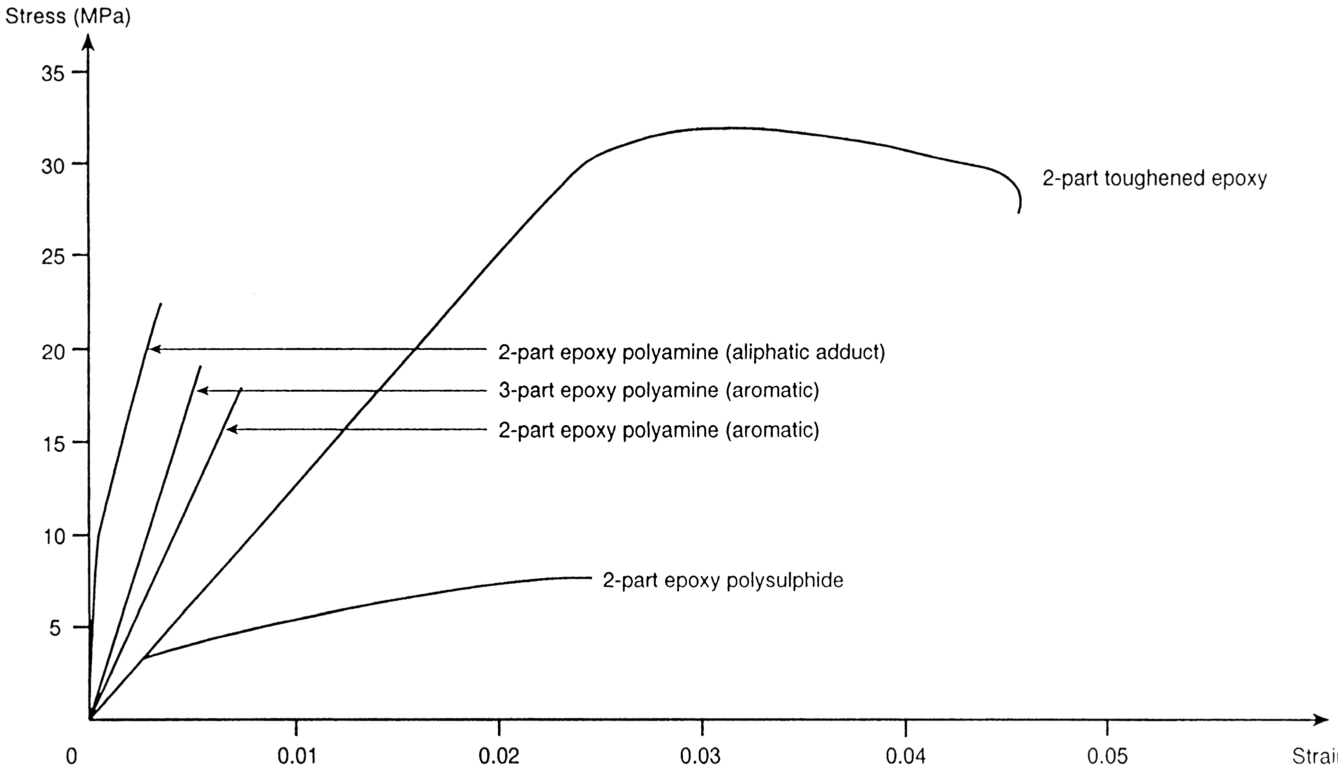


Figure 3.4 Typical tensile stress/strain curves for a range of epoxy adhesives.

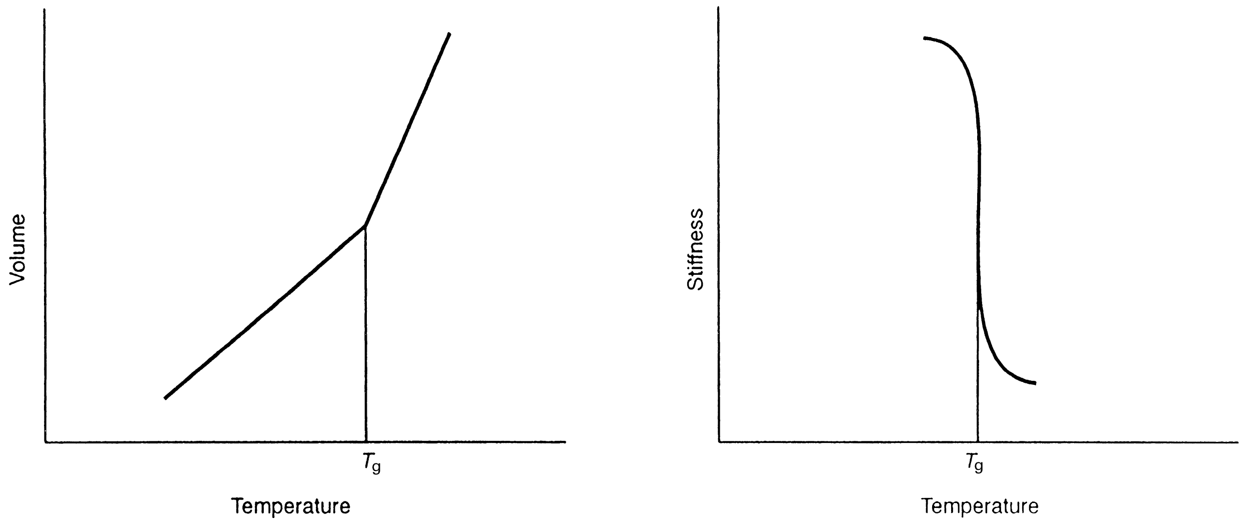


Figure 3.5 The glass transition temperature.

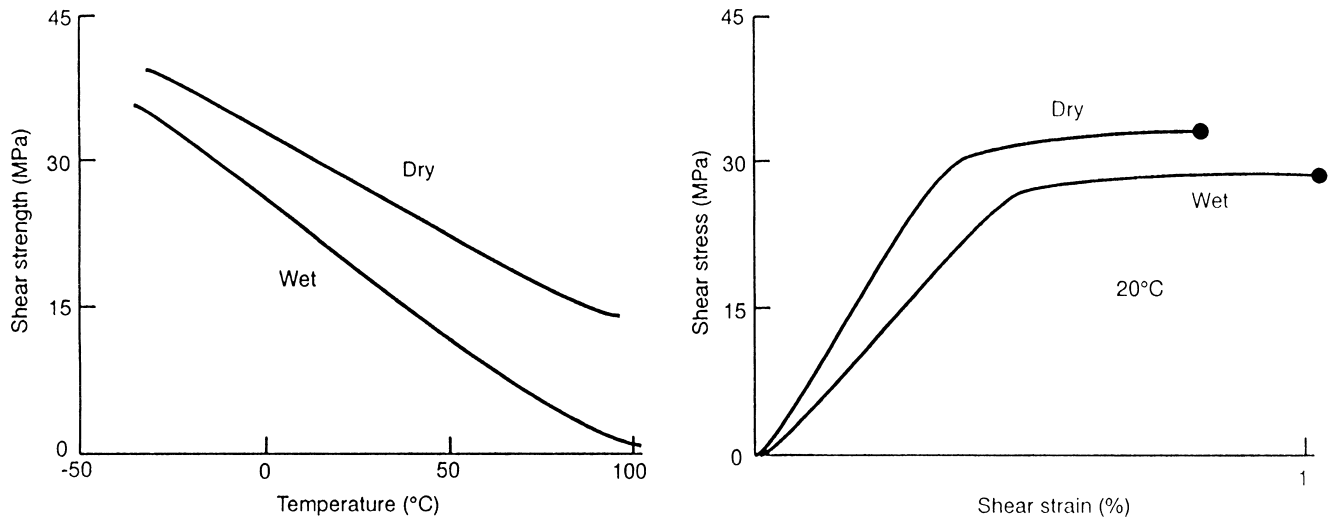


Figure 3.6 Typical effects of temperature and moisture on the mechanical behaviour of an unmodified cold curing epoxy adhesive in shear.

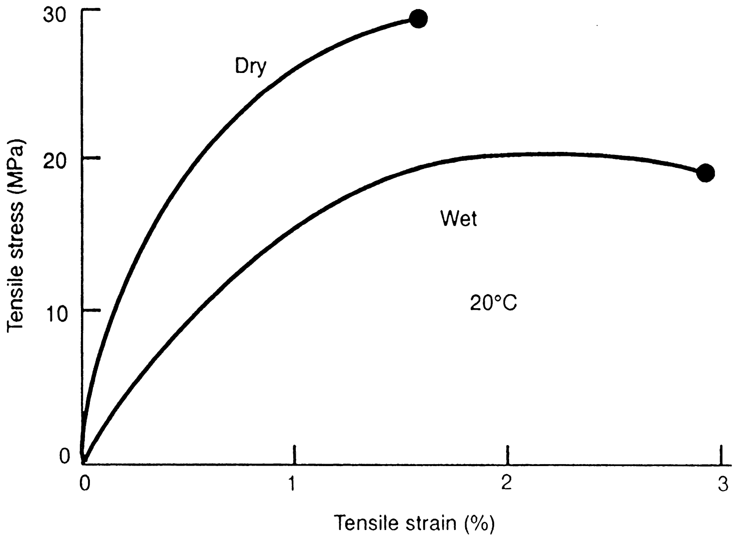


Figure 3.7 Typical effects of temperature and moisture on the mechanical behaviour of an unmodified cold curing epoxy in tension.

case of a stiff cold cure epoxy formulation with a relatively low strain-to-failure.

3.3.6 Testing

Testing represents a large and important subject in its own right and falls outside the scope of this book. A full discourse on the testing of adhesives in bulk form and of testing adhesive bonded joints is given in Mays and Hutchinson (1992). Some key test procedures are outlined elsewhere (Mays and Hutchinson, 1988).

The tensile stress/strain behaviour depicted in Fig. 3.4 was derived from tests conducted using dumb-bell specimens. Values of tensile strength, modulus and strain-to-failure feature in Table 3.3. Reliable values of adhesive material shear strength can be derived from the thick adherend shear test (TAST), as described by Mays and Hutchinson (1988, 1992), over a range of temperatures. Values for fracture energy can be obtained from tests conducted using double cantilever specimens as described by Mays and Hutchinson (1992). One classical method of obtaining T_g is to use dynamic mechanical thermal analysis on a small beam of adhesive. A more approximate technique involves deformation of a larger prism of adhesive in order to determine the heat distortion temperature (HDT).

3.4 Adhesion and surface preparation

3.4.1 Introduction

A full general discussion on the topic of adhesion and surface preparation has been given by many authors, including Mays and Hutchinson (1992). Particular discussion on adhesion and surface preparation of FRP surfaces is given by ASTM (1984), Clarke (1996) and by Hutchinson (1997). Some discussion on adhesion and surface preparation of concrete surfaces is given by ASTM (1983), Sasse and Fiebrich (1983), Gaul (1984), Sasse (1986), Edwards (1987) and Hutchinson (1993).

It has been noted that the most important construction difficulty in bonded joint design is surface preparation (e.g. Clarke, 1996). A major barrier to the more widespread use of adhesive bonding in construction is a lack of understanding about adhesion, appropriate surface preparation techniques and their effects on both initial bond strength and long term bond durability. Hot/wet environments are generally deleterious for adhesive bonds although there is little evidence to date that such environments are particularly harmful for FRP/concrete interfaces (see Chapter 6, Section 6.7). Nevertheless adequate surface preparation is still a vital prerequisite for obtaining and maintaining joint integrity.

3.4.2 Adhesion and interfacial contact

Adhesives join materials primarily by attaching to their surfaces within a layer of molecular dimensions, that is, of the order of 0.1–0.5 nm. The term ‘adhesion’ is associated with intermolecular forces acting across an interface and involves a consideration of surface energies and interfacial tensions. Being liquid, adhesives flow over and into the irregularities of a solid surface, coming into contact with it and as a result, interacting with its molecular forces. The adhesive then solidifies to form the joint. The basic requirements for good adhesion are therefore intimate contact between the adhesive and substrates, and an absence of weak layers or contamination at the interface.

Adhesive bonding involves a liquid ‘wetting’ a solid surface, which implies the formation of a thin film of liquid spreading uniformly without breaking into droplets (Fig. 3.8). Fundamentally, the surface tension of the adhesive should be lower than the surface energy of the solids involved, in this case, the treated surface of FRP and the exposed constituents of concrete. Because of the similarity in adhesive and composite matrix composition, values of surface tension and surface energy are similar. Both compositions contain polar molecular groups which are mutually attractive and chemically compatible. Thus good adhesion is assured, provided that

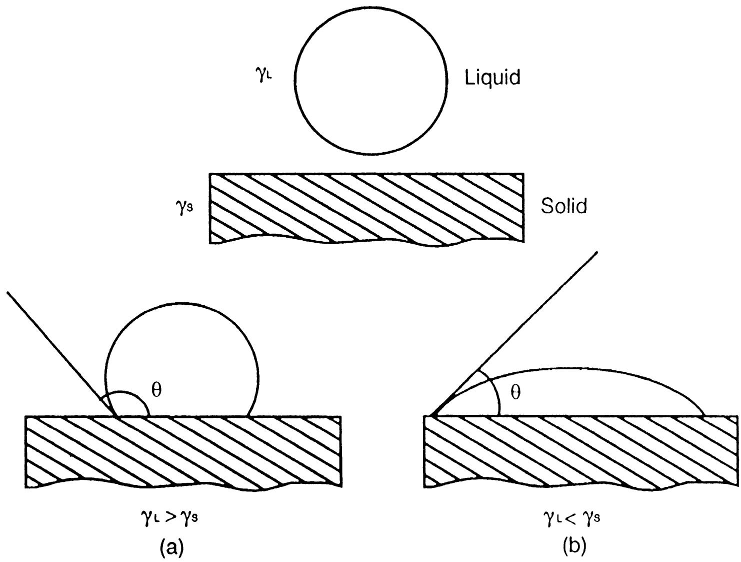


Figure 3.8 Simple representation of non-wetting (a) and wetting (b). γ_L is the surface free energy (surface tension) of a liquid, γ_S is the surface free energy of a solid and θ is the contact angle between solid and liquid.

contamination is removed by surface preparation. Some typical values of FRP surface energies and epoxy adhesive surface tension are given in Table 3.5.

The chemical and physical nature of the surface of concrete is complex and variable. The surface of this multiphase material may contain exposed aggregate, sand, unhydrated cement particles and cement gel, together with cracks and voids; the surface moisture content may also be variable. Surface treatments should remove significant contamination, cement-rich layers and traces of mould release agents. Mutual atomic or molecular attractions between an adhesive and the exposed (and probably heavily hydrated) constituents may exist, but mechanical keying or interlocking into the irregularities and pores of the surface probably play a significant role too. It is impossible to measure the surface energy of concrete, although it should be possible to measure the energies of its constituents separately. For example, Table 3.5 indicates a value for silica.

Finally, the rheological characteristics of the adhesive (or a primer) are also important and low viscosity materials are beneficial for penetrating into, and binding together, surfaces such as concrete which are porous and which may be fractured at a microscale. The interaction of surface energetics, tensions and rheology is a complex one.

Table 3.5 Typical values of surface free energies

Surface	Surface free energy, γ (mJ m^{-2})
Ferric oxide	1350[4]
Aluminium oxide	630[4]
Silica	290[4]
Water	72[5]
CFRP[1] (peel ply)	64
CFRP[2] (heavily abraded)	58
GFRP[3] (corona treated)	57
GFRP[3] (heavily abraded)	55
CFRP[1] (solvent degreased)	51
Epoxy adhesive	45[5]
GFRP[3] (as moulded)	38

[1] CFRP, vinylester matrix.
[2] CFRP, epoxy matrix.
[3] GFRP, polyester matrix.
[4] Under ideal conditions *in vacuo*; values measured in air are 50–100 mJ m^{-2}.
[5] Surface tension, rather than the surface free energy, of liquids is more usually quoted. Tension and energy are numerically identical but dimensionally different.

3.4.3 Surface preparation

The purpose of surface preparation is to remove contamination and weak surface layers, to change the substrate surface topography and/or introduce new surface chemical groups to promote bond formation. An appreciation of the effects of surface preparation may be gained from surface analytical or mechanical test techniques. Surface preparation generally has a much greater influence on long term bond durability than it does on initial bond strength, so that a high standard of surface preparation is essential for promoting long term bond performance.

Surface preparation represents part of the bonding operation discussed in Section 3.5. However a detailed discussion is provided in this section because of the importance of understanding the effects of preparation on bond integrity.

3.4.4 Surface preparation of concrete

For concrete strengthening applications the material must be treated *in situ*, sometimes under less than ideal conditions. The plane of the surface(s) to

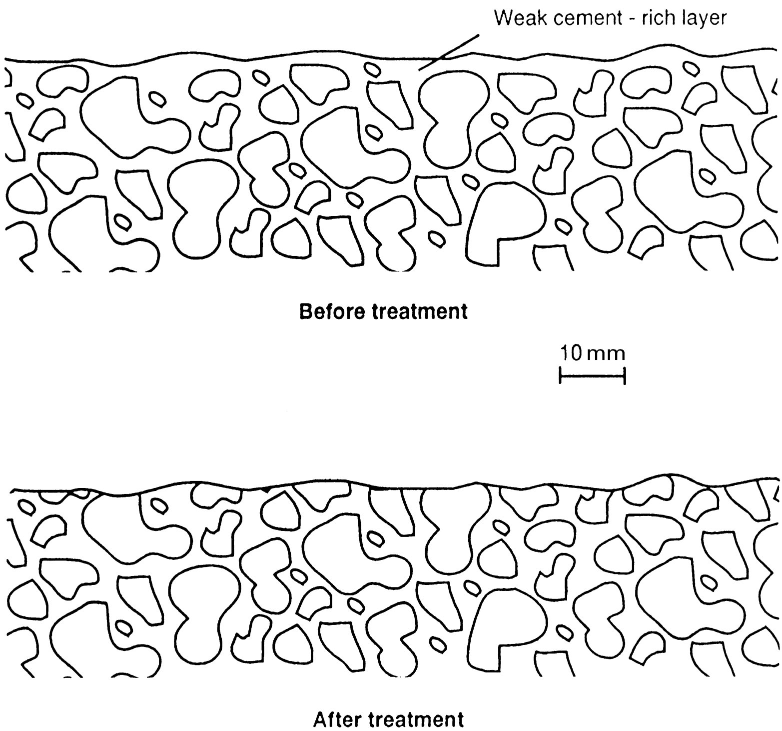


Figure 3.9 Schematic idealisation of concrete following surface preparation.

be treated (horizontal, vertical, overhead, etc.) has a large bearing on the selection of an appropriate method. The choice of method, or combinations of methods, depends upon the costs, the scale and location of the operation, access to equipment and materials, and health and safety conditions.

In essence, the purpose of surface preparation is to remove the outer, weak and potentially contaminated skin together with poorly bound material, in order to expose small- to medium-sized pieces of aggregate (Fig. 3.9). This must be achieved without causing microcracks or other damage in the layer behind which would lead to a plane of weakness and hence a reduction in strength of the adhesive connection. Any large depressions, blow holes and cracks must be filled with suitable mortars prior to the application of a structural adhesive. This will ensure a relatively uniform bondline thickness in order to maximise the efficiency of shear stress transfer. The basic sequence of steps in the process of surface preparation should be:

- To remove any damaged or substandard concrete and reinstate with good quality material.

- To remove laitance, preferably by grit blasting. Other techniques such as wire brushing, grinding, bush hammering, acid etching, water jetting or flame blasting are not recommended for plate bonding applications. In particular, the use of pneumatic tools can cause significant damage to the underlying concrete (Hutchinson, 1993).
- To remove dust and debris by brushing, air blast or vacuum.

Additional steps could include:

- Further cleaning, with a suitable solvent, to remove any remaining contaminant.
- Drying the surface to be bonded, if necessary.
- Application of an adhesive-compatible (epoxy) primer, if necessary.

The recommended method of laitance removal, by blasting, is fast, plant intensive and operator dependent. There exist a multiplicity of types of blast media, media sizes, blast pressures and types of equipment. With dry systems, oil and water traps should be used to prevent contamination from air compressors; dry systems may be open, or closed with vacuum recovery and recycling of the blast media. Generally a fairly microrough but macrosmooth surface is generated, together with a lot of dust; particles of blast media may also be left embedded in the surface. Clearly this dust and debris must be removed prior to bonding. Wet blasting is another option which overcomes some of the problems associated with the dust, but of course a wet concrete surface is left which must be dried prior to bonding. Gaul (1984) suggests that the surface water content should be less than 4% prior to the application of coatings.

After preparation, the suitability of the surface should be checked using the pull-off test procedure described in Chapter 6, Section 6.7.2. The time lapse between preparation and bonding should be minimised in order to prevent any further contamination of the surface. Adhesion to damp and wet concrete surfaces was investigated in the ROBUST project, as described in Chapter 6, Section 6.7.2 and Table 6.3.

3.4.5 Surface preparation of fibre reinforced polymer composites

For concrete strengthening applications the material may be treated off-site, enabling a variety of potential techniques to be used. However, relatively large areas of strip material will need to be treated in a reliable and consistent way.

The surface of composite material may be contaminated with mould release agents, lubricants and fingerprints as a result of the production process. Further, the matrix resin may include waxes, flow agents and 'internal' mould release agents which can be left on the surface of a cured composite. Surface preparation not only serves to remove contamination such as fluorocarbon release agents, but may also increase the surface area for bonding, promote micromechanical interlocking and/or chemically modify a surface. Care must be taken to ensure that only the chemistry and morphology of a *thin* surface layer is modified. It is important not to break reinforcing fibres, nor to affect the bulk properties of the composite.

It is recommended in structural bonding that any random fibre mats and surface veils are removed from the surface to ensure stress transfer directly into the main reinforcement fibres. Such a task is rather difficult to control using mechanical removal methods such as grinding or grit blasting. For this reason, careful consideration should be given to the fabrication of the composite material in the first place in order to make it ready for adhesive bonding with minimal disturbance. The main methods of surface preparation for composites are solvent degreasing, mechanical abrasion and use of the peel-ply technique, and these methods are often used in combination. A fuller discussion of these and other techniques is considered by Hutchinson (1997). The effect of such treatments is to make the composite surface more 'wettable', as shown in Table 3.5.

The basic sequence of steps in the process of surface preparation should be either:

- To remove grease and dust with a suitable solvent such as acetone or methyl ethyl ketone (MEK).
- To remove release agents and resin-rich surface layers by abrasion. This can be accomplished by careful grinding, sanding (e.g. ASTM, 1984; EXTREN® Design Manual, 1989), light grit blasting using very fine alumina (e.g. Bowditch and Stannard, 1980), or cryoblasting with solid carbon dioxide pellets. The resultant surface should be textured, dull and lustreless.
- To remove dust and debris by solvent wiping.

or:

- To strip off a peel-ply layer. A peel-ply is a sacrificial layer about 0.2 mm thick which is laid-up on the outermost surfaces of a composite material and co-cured with it. During cure, the peel-ply layer becomes consolidated into the surface of the composite.

In the ROBUST system of structural strengthening, the CFRP strips were pultruded with a clean scrubbed nylon peel-ply layer incorporated into their surfaces. This provides a uniquely practical surface preparation method such that unskilled operators can achieve a very clean and bondable surface on site. This technique was first advanced for use for such applications by Hutchinson and Rahimi (1993).

There are several special considerations associated with peel-ply technology which are outside the scope of this book, but the subject is discussed more fully elsewhere by Hutchinson (1997). Suffice to say that combinations of composite resin matrix, peel-ply materials and composite processing conditions need to be assessed carefully. When the dry peel-ply material is pulled off, the top layer of resin on the FRP component is fractured and removed, leaving behind a clean, rough, surface for bonding. The resultant surface topography is essentially an imprint of the peel-ply fabric weave pattern (Fig. 3.10).

Finally, all FRP materials absorb a small amount of moisture due to ambient humidity levels or proximity to a wet environment. In CFRP it is the matrix resin which absorbs the moisture, but in GFRP and AFRP it is both the fibres and the matrix resin which can absorb moisture. For ambient temperature curing adhesive systems the presence of small amounts of moisture is unlikely to pose a problem although the EUROCOMP Handbook (Clarke, 1996) recommends that the adherends be dried; this may not, of course, be practical! However with elevated temperature curing adhesive systems the FRP components *must* be dried as far as possible (say to $<0.5\%$ moisture content by weight). This is because the heat curing process draws

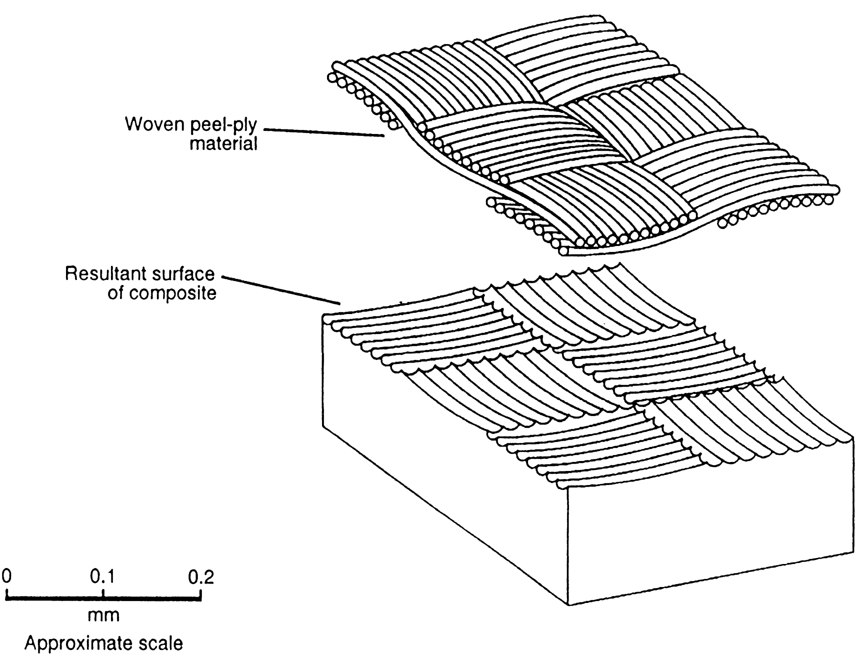


Figure 3.10 Typical surface topography arising from woven peel-ply material.

moisture out of the adherend(s) and into the bondline, resulting in voiding and weakening of the adhesive itself.

The suitability of surfaces for bonding following any treatment can be assessed using wettability tests and mechanical tests. Single lap shear joints of the type described in Chapter 6, Section 6.7.3 may be used to assess adhesion and joint strength.

3.5 The bonding operation

The satisfactory fabrication of joints relies upon the manufacture and provision of high quality composite materials to specified tolerances, suitable detailed designs and agreed methods of erection and completion of the work. Training of site operatives and their supervisors is essential. More detailed commentaries on the aspects detailed below are given by Mays and Hutchinson (1992), Department of Transport (1994), Clarke (1996) and Hutchinson (1997).

3.5.1 Handling, finishing and storage of materials

FRP components are relatively light and long lengths may be rolled up for convenience. They should be stored with suitable protection and supported off the ground in well-ventilated dry conditions. A peel-ply surface layer, if on both sides of the composite, will provide protection against scratching, chafing and damage to the bonding surface.

Any machining, trimming and drilling can give rise to problems of splitting, delamination and fibre pull-out. Thus FRP components must be supported rigidly with a suitable backing piece to enable these operations to be carried out without causing damage. Large amounts of dust may be generated which are best extracted directly by vacuum. Ideally, any sawn or drilled surfaces should be sealed with a compatible resin to protect the fibre reinforcement from the direct effects of moisture in service.

Two-part epoxy adhesives are solvent free and simply require protection from extreme conditions. Aspects of shelf life and the effect of temperature on viscosity and application characteristics were discussed in Section 3.3. Cleaning solvents (and primers) contain flammable materials and they require appropriate storage facilities at the place of fabrication.

3.5.2 Protection of the working environment

The working environment should be as dry and as clean as possible. Many adhesives will not cure below about 5 °C, so that attention must be paid to controlling both the ambient temperature and the temperature of the component surfaces. For example, a warm adhesive applied to a very cold surface, even if properly prepared, may not adhere to it adequately because of the tendency to 'skin' due to thermal shock. Infrared heaters, space heaters and heating blankets can be used both to warm the components prior to bonding and to assist with curing postbonding. Dehumidifiers may also need to be used if the dew point is in excess of 10 °C.

The surfaces of components to be bonded must be dry and this can be achieved by proper attention to storage, an enclosed working environment and by physically drying or preheating the components.

3.5.3 Surface preparation

A high standard of surface preparation is essential, as discussed in Section 3.4. However, surface preparation represents probably the most difficult part of the bonding process to control and this often relates to the location and scale of the strengthening operation. Thus simple, reproducible, surface preparation methods are required such as the use of the peel-ply technique for composite materials. It has been stated that the time elapsed between component surface preparation and bonding should be kept as short as possible, principally to avoid subsequent contamination. Whatever the method of surface preparation used, the resultant 'qualities' such as wettability, soundness and freedom from contamination can be measured and compared against recommended criteria. A piece of adhesive tape can be used to detect the presence of dust and debris. If moisture is picked up by absorbent paper pressed against the concrete it is likely to be too damp for bonding.

3.5.4 Adhesive mixing, dispensing and application

Two-component adhesives must be mixed thoroughly and modern packaging generally removes the need for weighing or measurement, thus removing the potential for errors. Resin and hardener components are frequently of dissimilar colours so that complete mixing can be judged when a uniform colour has been achieved. With some systems supplied in tins, the hardener component is poured into the container partially full of resin and mixing can then take place. Alternatively, the resin and hardener may be supplied in co-axial or twin cartridges which can be mounted in hand-held guns; the pistons in the cartridges are advanced by a trigger mechanism and mixing takes place within special disposable nozzles.

Where two-component adhesives are mixed together in a can or container, there is a finite usable life or pot life as described in Section 3.3.4. It was also noted that the viscosity, and therefore ease of application, of adhesive material can vary significantly with temperature. Adhesives with the correct characteristics at 20 °C may be impossibly viscous at 5–10 °C, or too fluid at 30 °C.

It is important to spread the adhesive soon after mixing to dissipate the heat generated and extend its usable life. Adhesive may be applied either to both concrete and the FRP, or to the FRP only. Common practice in plate bonding is to spread the adhesive slightly more thickly along the centreline of the FRP plates than at the sides of the plate. This can be achieved by using a curved adhesive spreader tool to result in a convex adhesive layer profile. This procedure reduces the risk of forming voids when pressing the plate against the concrete surface. Excess adhesive squeezed out of the bondline can be scraped away and a fillet formed at the plate edge.

3.5.5 Component fit-up and bondline thickness control

Good design and sequencing of the bonding operation should facilitate the positioning of the plates, as well as controlling bondline thickness. Where end-anchorages are used, precise positioning of the plates may be achieved by using anchor bolts placed through special plate end-tabs and into predrilled holes in the concrete.

Thin FRP plates are so light that no temporary clamps or fixtures are required. The plates are simply pressed against the concrete surface using hard rubber rollers and the ‘tack’ of the uncured adhesive is sufficient to hold the plates in position whilst it cures.

A target adhesive thickness of 1–2 mm is recommended. A thick bondline may result in reduced joint strength and increased consumption of relatively expensive material. Conversely a very thin joint may result in ‘brittle’ joint behaviour. Control can be achieved by the insertion of small thermoplastic discs within the bondline or by the addition of glass spheres called ballotini to the mixed adhesive.

3.5.6 Curing

Curing and hardening can sometimes represent a significant process in scheduling and completing the bondline operation, giving rise to the need for enclosure and artificial heating of the working environment (Section 3.5.2). The actual rate of cure development depends upon the adhesive formulation and the ambient temperature, with some scope often being provided by adhesive manufacturers who offer ‘normal’ and ‘rapid’ grades. A typical comparison is provided for Sikadur 31PBA, the adhesive used in the ROBUST project, as shown in Fig. 3.11 and 3.12. A significant degree of cure can be expected between 6 h and 12 h over a range of temperatures between 5–20 °C. Products will be almost fully cured after 24 h.

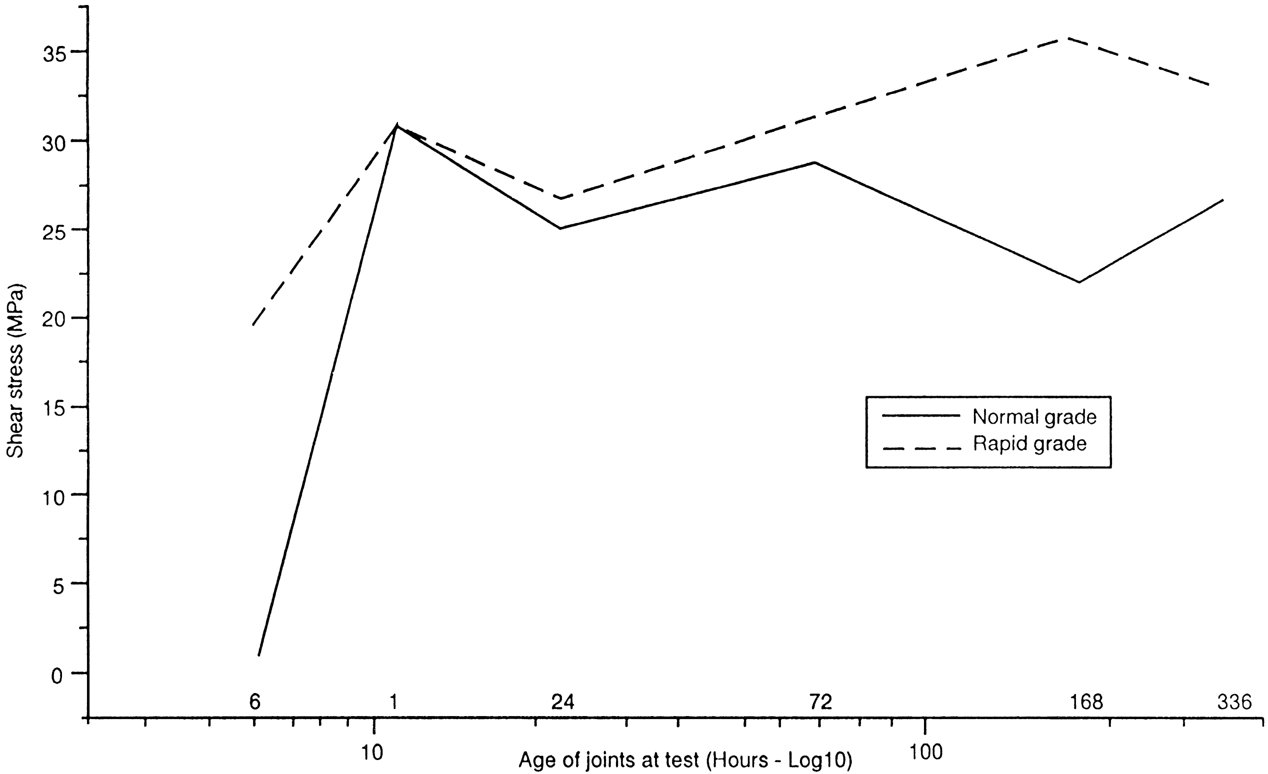


Figure 3.11 Shear strength (MPa) of Sikadur 31PBA normal and rapid grades from thick adherend shear test (TAST) joints cured and tested at 23°C.

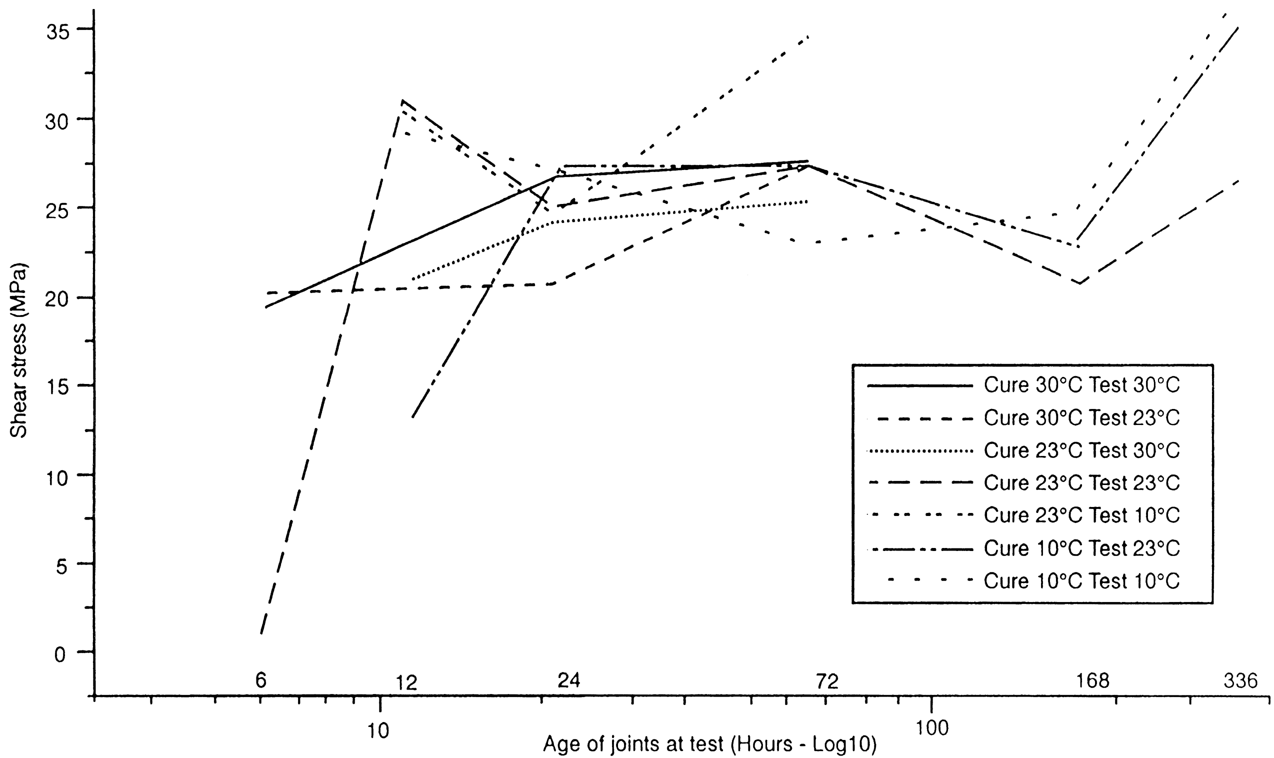


Figure 3.12 Shear strength (MPa) of Sikadur 31PBA normal grade, from thick adherend shear test (TAST) joints cured and tested at different temperatures.

3.6 Durability and fire

Low temperatures generally have little effect on the strength and stiffness performance of composite materials other than to make the polymer less flexible. This could result in a tendency to damage by fatigue and in extreme cases it should be considered. High temperature on the other hand is more problematic, as all resins soften when heated sufficiently. It is therefore important that a composite is designed and specified such that its design temperature is compatible with its glass transition temperature (see also Section 3.3.5).

Carbon fibre is a conductor of electricity and composites in which it is used may be highly noble relative to some metals. If such materials are in contact, galvanic corrosion can take place, resulting in corrosion of the metallic element. Contact between the two materials must be avoided by the incorporation of a suitable barrier.

Polymers absorb moisture to a certain extent and this has the effect of altering the properties of the polymer and of course a composite of which it is the matrix while wet. Mechanical properties and glass transition temperature tend to be reduced as a function of the amount of water absorbed. The effect may be trivial in many circumstances but nevertheless should be taken into account by the designer.

The fire resistance of polymer composites ranges from 'highly resistant' to 'burns readily'. Fire resistance may be enhanced by high fibre content, a high level of filler in the resin mix and by the inclusion of fire retardant additives if necessary. The fire performance characteristics which are most appropriate for structural bonding applications are 'surface spread of flame' and 'ease of ignition'. Both must be designed to be appropriate for the circumstances.

3.7 Painting

It is generally not necessary to paint composites, but if desired, painting can be carried out successfully; paint suppliers' advice should be sought on specific applications. Primers are available specifically for use with composites which allow subsequent painting with cellulose and polymer lacquers. Two-pack polyurethane is a tough, long lasting paint system which does not generally need to be used in conjunction with a primer.

3.8 Summary

Structural adhesives have a long history of use in construction, with epoxy adhesives having been exploited for externally bonded steel plate reinforcement since the mid-1960s. Adhesive bonding represents the natural method of joining together dissimilar materials such as concrete and polymer composites. Fibre reinforced polymer composites are being used increasingly because of their obvious advantages of high strength/weight ratio, design freedom, ease of handling and corrosion resistance. The vast majority of composites used in construction are reinforced with glass fibres, but the modulus and strength requirements for externally bonded reinforcement generally dictate the use of carbon fibre reinforced composites. Composites produced by the pultrusion and prepreg routes are of high fibre volume fraction, excellent mechanical properties in the fibre direction and of high reliability and quality. They must, however, be handled with care and due precautions should be taken when machining and drilling them.

Control of the adhesive bonding operation is crucial to the satisfactory fabrication of all reliable and durable bonded joints. There are many aspects which must be considered including the storage of materials, protection of the working environment, surface preparation of the components, adhesive mixing, dispensing and application, joint fit-up, bondline thickness control and curing. When bonding metals in particular, one of the most difficult aspects to control on site is surface preparation. By substituting plastics for steel not only is the reliability of bonding improved, but by using a peel-ply layer on the surface of the composite plates (as in the ROBUST project) a reliable and practical site-based preparation technique is assured.

The durability of the materials and of the bonded interfaces is considered in Chapter 6.

3.9 References

ASTM D2903 (1984) *Standard Practice for Preparation of Surfaces of Plastics Prior to Adhesive Bonding*, American Society for Testing and Materials, Philadelphia.

ASTM D4258 (1983) *Standard Practice for Abrading Concrete*, American Society for Testing and Materials, Philadelphia.

Bowditch M R and Stannard K J (1980) ‘Bonding GRP with acrylic adhesives’, in *Adhesion 5*, ed. K W Allen, London, Applied Science Publishers, Chapter 6.

Clarke J L (ed.) (1996) *Structural Design of Polymer Composites*, EUROCOMP Design Code and Handbook, London, E & F N Spon.

Department of Transport (1994) *Strengthening of Concrete Highway Structures using Externally Bonded Plates, BA 30/94*, London, HMSO, Vol. 13, Section 3, Part 1.

Edwards S C (1987) ‘Surface coatings’, in *Repair of Concrete Structures*, ed. R T L Allen and S C Edwards, Blackie & Son, London, Chapter 10.

EXTREN® (1989) *Design Manual*, Morrison Molded Fiber Glass Company, Bristol VA, USA, Section 13.

Gaul R W (1984) ‘Preparing concrete surfaces for coatings’, *Concrete Int* **6**(7) 17–22.

Hollaway L C (1993) *Polymer Composites for Civil and Structural Engineering*, Glasgow, Blackie Academic and Professional.

Hutchinson A R (1993) 'Review of methods for the surface treatment of concrete', in *Characterisation of Surface Condition*, MTS Project 4, Report No 2 on the Performance of Adhesive Joints, Dept. of Trade and Industry, London.

Hutchinson A R (1997) *Joining of Fibre-Reinforced Polymer Composite Materials*, Report 46, CIRIA, London.

Hutchinson A R and Rahimi H (1993) 'Behaviour of reinforced concrete beams with externally bonded fibre-reinforced plastics', *Proc. 5th Int. Conf. on Structural Faults and Repair*, ed. M C Forde, Engineering Technics Press, Edinburgh, Vol. 3, pp 221–228.

Mays G C and Hutchinson A R (1988) 'Engineering property requirements for structural adhesives', *Proc Inst Civil Eng* **85**(2) 485–501.

Mays G C and Hutchinson A R (1992) *Adhesives in Civil Engineering*, Cambridge University Press.

Sasse H R (ed.) (1986) 'Adhesion between polymers and concrete', *Proceedings International Symposium in Aix-en-Provence*, Chapman and Hall, London.

Sasse H R and Fiebrich M (1983) 'Bonding of polymer materials to concrete', *RILEM Materials and Struct* **16**(94) 293–301.

4 Structural strengthening of concrete beams using unstressed composite plates

L C HOLLAWAY AND G C MAYS

4.1 Introduction

Various geometric and loading parameters will be discussed in this chapter in an attempt to gain a basic structural appreciation and understanding of the loading characteristics and failure mechanism for fibre reinforced polymer (FRP) composite plated beams, in particular, the effects which may initiate overall collapse. However, some of the basic parameters have been investigated numerically using the finite element analyses; these are discussed in Chapter 8.

This chapter will examine the effects of varying as wide a range of parameters as possible to obtain knowledge of how the strengthening system may be optimized and thus improve the overall performance of the beam. A discussion will be given of the effect of the different parameters on overall load/deflection and strain responses up to failure, the mode of failure and the transfer of strain between the concrete and the external plate. These discussions will draw upon the results and observations made during the ROBUST project.

The following procedure adopted for the preparation of the plated 1 m, 2.3 m, 4.5 m or 18 m long beams, is the one recommended for the ROBUST system. The concrete bond surface was gritblasted, typically using copper slag and the surface was then cleaned by a vacuum process. Oil or grease may be removed by cleaning with detergent, caustic soda or trisodium phosphate followed by thorough rinsing with water to remove any residue. The composite plate of the required dimensions was then bonded in place after removal of the peel ply. A 2 mm thickness of adhesive was used for all external plate bonding and for laboratory beams this was applied to both the concrete and the composite plate. For some of the 18 m site beams the adhesive was applied to the concrete surface and to the composite plate, whilst for other beams the adhesive was applied to the concrete surface only. To control the 2 mm thickness of adhesive in the laboratory, ballotini of 2 mm diameter were placed sparingly in the adhesive; however, this method was not followed on site. After compressing the two surfaces together, excess adhesive was carefully removed and a pressure was applied to the plate to maintain complete contact; the adhesive used was Sikadur 31 PBA.

Part A Laboratory tests

4.2 General form and behaviour of loaded beams

4.2.1 Unplated beam response

In the following discussions in this chapter, it will be assumed that the beam is loaded in four point bending and that the shear span to beam depth ratio lies between 3.5 and 8.0. The significance of this ratio is discussed in Section 4.3.1 of this chapter.

The load deflection response of unplated beams exhibits three regions of behaviour, as shown in Fig. 4.1. At low applied loads, the stiffness of the reinforced concrete (RC) beam is relatively high, indicating that the concrete behaves in a linear elastic manner. As the load increases the bending stresses in the extreme fibres at the top and bottom of the section increase until the tensile strength of the concrete is reached at the base of the beam,

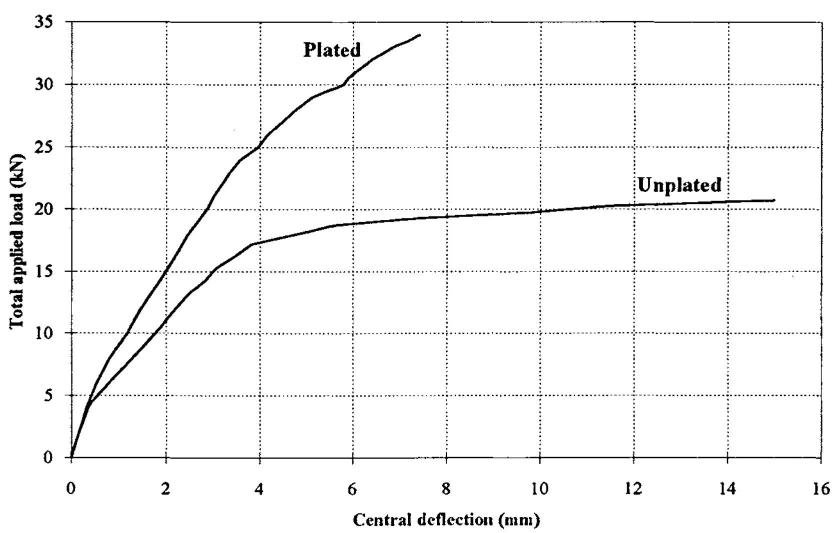


Figure 4.1 Comparison of typical load/deflection responses of unplated and plated 1 m long beams.

causing flexural cracks to form, initially in the constant moment region. The occurrence of flexural cracking causes a marked reduction in the member stiffness, as shown by a sudden change of gradient in the response. The response after the cracking load is approximately linear, at a gradient which will be referred to in this book as the postcracking stiffness.

After the concrete cracks in tension, a greater proportion of the tensile bending component is carried by the reinforcing steel at the base of the section. As the beam rotation increases further, the tensile stress in the reinforcing steel increases, theoretically in a uniform distribution throughout the constant moment region. In practice, however, it would reach maxima where flexural cracks are bridged. Eventually, the yield stress of the steel is reached at one or more points; this loss of material stiffness results in a reduction in overall beam stiffness (defined as the total load/central deflection) as the ability of the section to support the tensile component of the bending moment is reduced. This is shown by the second marked change in gradient of the load/deflection response at a load, referred to as the yield load. It is assumed in the above discussion that the beams are designed to be under-reinforced so that yielding of the steel precedes crushing of the concrete in compression. It would be expected that failure would be characterized by large strains in the reinforcing steel and that the deflection of the beam at collapse would be substantial and would be accompanied by excessive cracking.

Flexural cracks formed in the constant moment region will extend vertically upwards and become progressively wider as the applied load is increased. Cracks will also initiate in the shear span as the load increases. At collapse these will typically cover half to two-thirds of the length of the shear span; these cracks initially extend vertically, then tend to progress towards the load points in a diagonal fashion.

4.2.2 Plated beam response

In the following discussion in this section, shear span/beam depth ratios of 3.0 will be considered and the widths of the composite plate will cover or nearly cover the full width of the soffit of the beam (for a discussion on the effect of plate width, see Section 4.3.2). The shear span/beam depth ratio for the plated beams in this book are based on the full beam depth since this takes into account the depth of concrete to the lowest level of tensile reinforcement (i.e. the composite plate). The general form of the load/deflection response of a typical plated beam is shown in Fig. 4.1 and is very similar to an unplated beam but with several key differences.

The initial stiffness before cracking is almost identical for the two cases. This is to be expected since, before the concrete has cracked, all of the section is effective and the plate has relatively little effect on the second moment of area, and hence flexural rigidity of the section, unless a very stiff plate of high cross-sectional area is utilized. In addition, since the load which causes the concrete to crack initially is dependent on the second moment of area of the section, this will also be altered little by the addition of an external plate.

The postcracking stiffness of the plated beams is significantly higher than that of the unstrengthened beams. After cracking, when the concrete beneath the neutral axis is no longer as effective at supporting load, the addition of an external plate causes the second moment of area of the section, and hence the flexural rigidity of the strengthened member, to be increased significantly; the amount depends upon the plate material and cross-sectional area. When externally bonded to the tension face of the member, the plate is positioned most advantageously in terms of increasing the beam stiffness, since its lever arm to the neutral axis of the section is a maximum. In addition, the external plate also provides a mechanism by which tensile stress can be distributed to intact concrete between cracks, resulting in improved performance of the concrete in the tension zone. This enables the section to work more efficiently and produce a tension stiffening effect, whereby the concrete can contribute to the second moment of area and thus to the flexural rigidity of the section. The location of the plate on the tension face is again the most advantageous for this purpose, as well as for restraining crack opening.

Before yielding, the tensile component in flexure is shared between the internal steel reinforcement and the bonded plate. The concrete beneath the neutral axis also contributes to some extent as a result of tension stiffening, described above, from both the steel and plate. As a result of the increased beam stiffness and since the internal steel is carrying a lower proportion of the tensile component for a given applied load when the beam is plated, the yield stress of the steel is reached at a higher applied load. The fact that the external plate relieves some of the tensile stress carried by the internal steel reinforcement is structurally significant since the reinforcing bar strains control the crack widths in concrete.

As the internal steel yields, so the overall member stiffness decreases, as for the unplated beams. However, after yielding, the plate continues to support the tensile component of the moment couple acting on the section. Hence the reduction in flexural rigidity is significantly less than when the beam is unplated. This stiffness after yielding is referred to in this book as the postyielding stiffness and is taken as the gradient of a line joining the yield point to the point of collapse. The plate is able to support an increasing tensile bending component and the plated beams can sustain considerably higher applied loads before collapse occurs. One further characteristic of a plated beam response compared with unplated beams is the reduction in ductility to collapse of the system.

Figure 4.2 shows a typical progression of cracking and collapse for plated beams under a shear span/beam depth ratio of 3.0. Flexural cracks are initiated in the constant moment region as the tensile strength of the concrete is reached (see Fig. 4.2(a)). The cracks propagate upwards as loading progresses, but remain very narrow throughout the loading history, being far smaller than those in the unplated beams. This demonstrates the restraining effect which the plate has on crack opening, an important consideration from a serviceability viewpoint. Further flexural and flexural/shear cracks initiate at locations progressively further along the shear spans as the load level increases (see Fig. 4.2(b)). Inclined cracks propagate towards the loading points, becoming shallower in angle closer to the supports. This diagonal cracking is a result of the increase in shear loading on the beams. They widen as the applied load increases, but remain narrow at the base of the beam, demonstrating the confining effect of the external plate. Eventu-

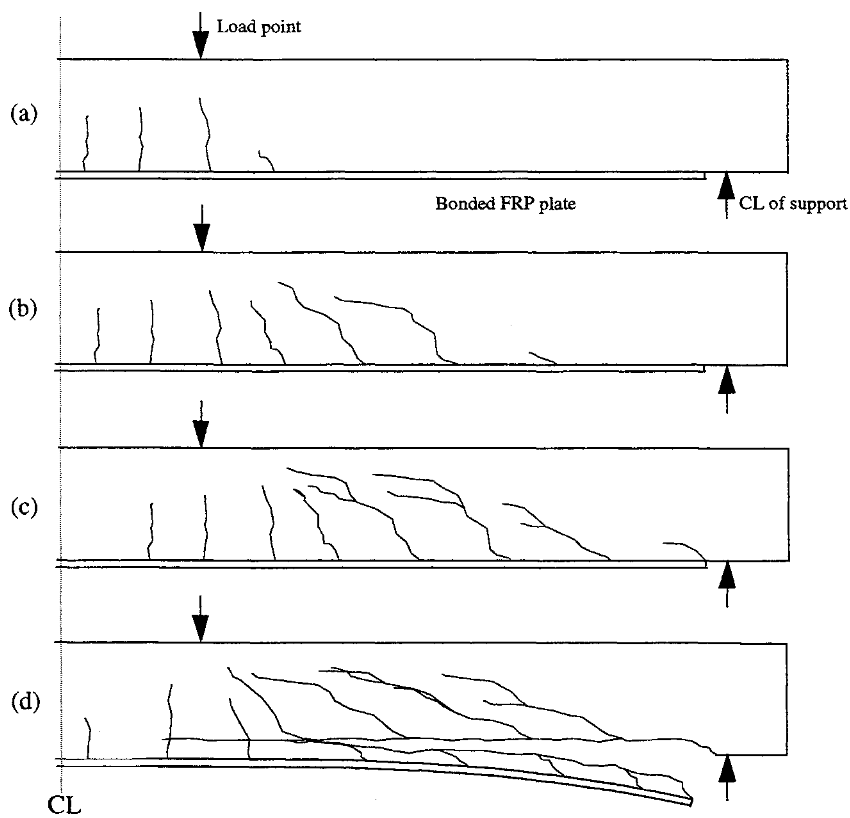


Figure 4.2 Typical progression of cracking and collapse for a plated unanchored beam (CL = centreline).

ally, diagonal cracks will initiate at, or close to, the end of the plate (see Fig. 4.2(c)) and these would propagate roughly to the level of the internal steel reinforcement, at which point the plate would suddenly separate from the beam, peeling downwards, taking with it the cover concrete and causing the collapse of the beam (see Fig. 4.2(d)). This separation often extends for the length of the shear span and occasionally past the beam centreline. In the above case it is assumed that the plates are not anchored to the beam during the loading operation.

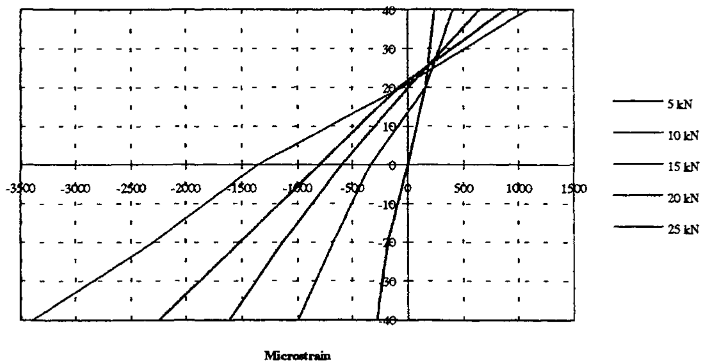
Although the diagonal shear cracks which propagate towards the load can be seen as failure approaches, collapse itself is generally very brittle in nature. In the above discussion it is assumed that the beams have been designed in the unplated state to fail in flexure with adequate shear reinforcement to support this ultimate load, as would be the case for existing members in service. When plated, the applied load can exceed that of the unplated case by a considerable amount and consequently the beam becomes under-reinforced in shear for this higher load, with the occurrence of widespread shear cracking.

Figure 4.3 illustrates typical effects of plating on the longitudinal strain distribution through the depth of the section about the centre of the beam; this illustration is typical for any shear span/beam depth ratio. The vertical axes represent the depth through the section, with the datum being the horizontal mid-plane of the beam. For the case of the strengthened beams the longitudinal strain distribution is extended to include strains at the centre of the composite plate. This demonstrates whether the plate and concrete beam are acting compositely by showing the degree of strain compatibility between the two materials across the adhesive bondline.

This comparison indicates that for any given load stage, the tensile strains towards the base of the section, which represents the strains in the concrete and the accumulation of crack widths, were considerably higher for the unplated beams (Fig. 4.3(a)) than for the identical beams externally strengthened (Fig. 4.3(b)). This demonstrates the effect of the plate in restraining the opening of cracks and maintaining the general integrity of the section. In addition, by the utilization of externally bonded reinforcement the mean crack height, as well as the crack widths, will reduce. This is shown in Fig. 4.3 which reveals that, for a given load, the neutral axis is lower for the strengthened beams and thus the tensile cracks are reduced in height. The preservation of composite action between the external plate and reinforced concrete member is one of the most important structural requirements of the plate bonding technique if it is to be successful as a repair method.

Figure 4.4(a) shows typical load/strain responses obtained for one-half of a strengthened 1.0m long beam, where the gauge positions refer to those in

(a)



(b)

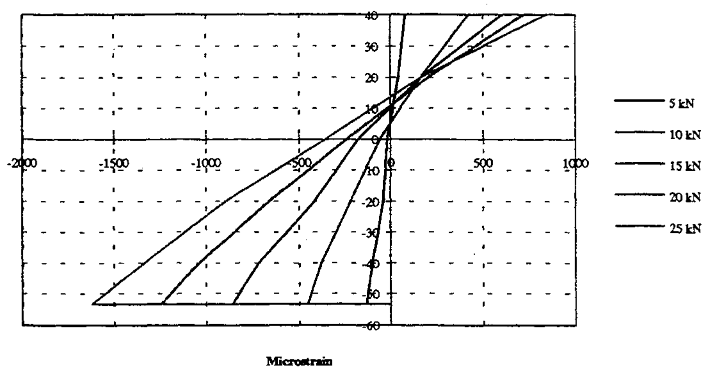
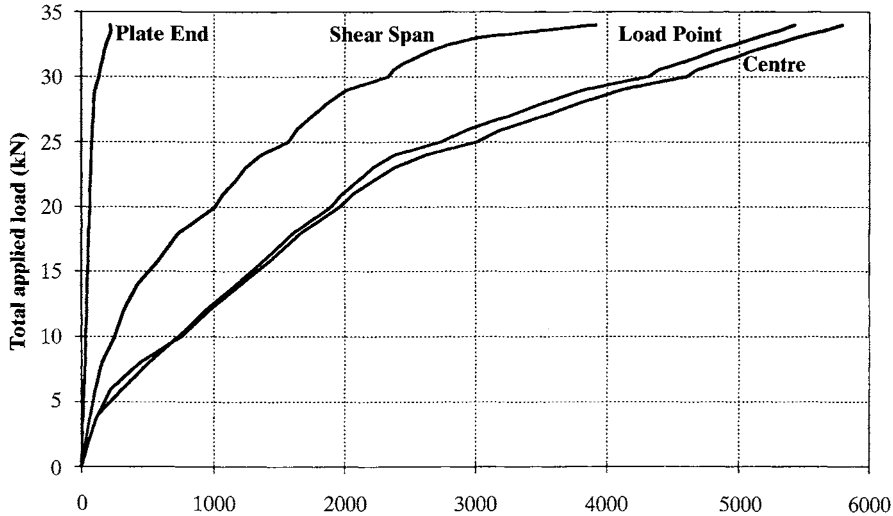
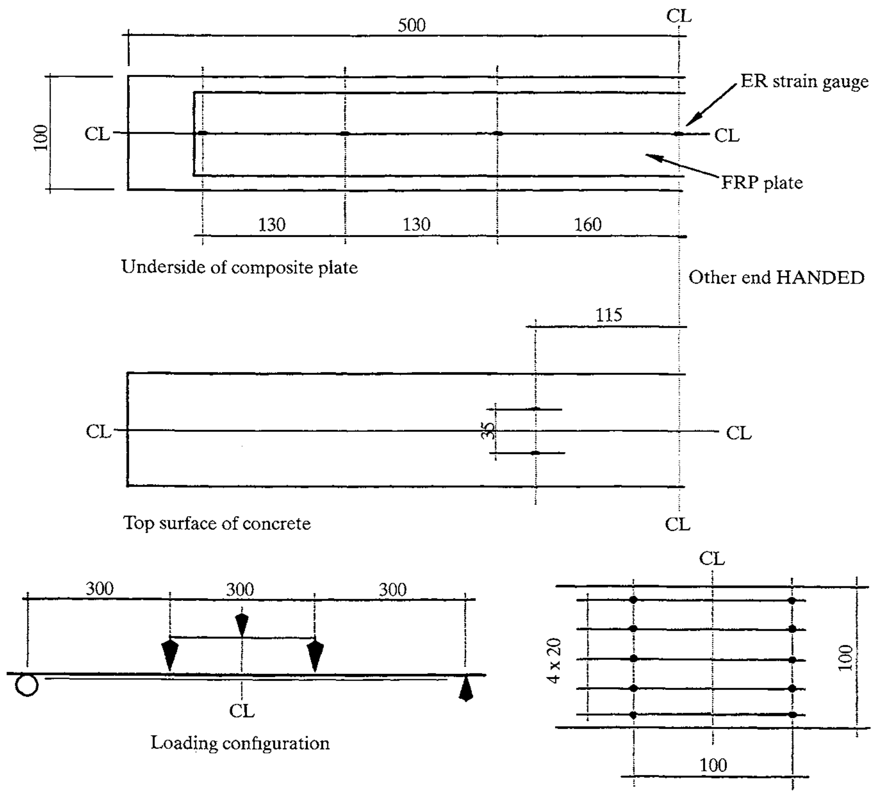


Figure 4.3 Comparison of typical section strains obtained for unplated and plated beams at various loads.

Fig. 4.4(b). This relationship was obtained for a four point loading situation. The load/strain values at the beam centre and close to the load point are of similar form, the central values being slightly higher than those recorded just outside the constant moment region. The load/strain behaviour shows that initially, before cracking of the concrete occurs, the rate of strain developed in the plate is low, indicating that the concrete is supporting a large proportion of the tensile bending component. As cracking initiates and develops, the strain rate increases markedly and then remains approximately constant until yielding of the internal steel occurs, at which point the strain rate increases further, then again remains approximately constant thereafter until collapse. These linear characteristics in the postcracking and post-yielding regions reflect the linear nature of the FRP in tension. The gauges in



(a)



(b)

Figure 4.4 (a) Typical load/strain response obtained at gauge positions along plate. (b) Instrumentation and loading arrangement for 1 m beams. Dimensions are in mm, not to scale.

the shear span and especially close to the plate end retain their initial high rates of strain up to higher loads than the values closer to the beam centre. This demonstrates how cracking progressively initiates further along the shear span towards the support as the level of applied loading increases. Considerably more strain is indicated in the centre of the shear span as the beam approaches collapse, demonstrating that flexural/shear cracking becomes more prevalent in this region as the load increases.

It will be observed from Fig. 4.5 that outside the constant moment region, the longitudinal strain distribution appears initially to follow the bending moment diagram. A linear variation of the axial stress in the external plate between the plate end and the point of load application can theoretically be shown to exist at the ultimate state, although this assumes that the position of the neutral axis does not change along the plated length for a given level of loading. An implication of linear variation in axial strain is that there exists a uniform distribution of shear between the external plate and the concrete. However, whilst such a distribution appears to exist throughout the plate for the majority of the loading range, at higher loads the longitudinal strains in the shear spans increase above those of a linear variation, showing that strains are not proportional to the applied moment at these locations.

Figure 4.5 also indicates that as the applied load increases towards its maximum value, the distribution of strain in the plate becomes slightly more unsymmetrical. It is possible that the longitudinal strain may attain higher values on one side of the centreline than on the other. This type of behaviour may be accounted for by defects in the bondline, unsymmetrical distribution of cracking and general inhomogeneity of concrete.

Typical compressive strain responses, shown in Fig. 4.6, measured on the compression face of the concrete close to each of the loaded points, are very similar, as would be expected for symmetrical loading conditions. These responses are of a similar pattern to the tensile behaviour for the external plate in the constant moment region. The occurrence of initial cracking is detected as a change of gradient in the response, which then continues at an approximately constant rate up to yield of the internal steel reinforcement; this is again marked by an increase in the rate of strain as the overall member stiffness is reduced.

4.3 Geometric parameters

4.3.1 Effects of various shear span/beam depth ratios

The loading configurations clearly affect the way in which a plated beam will fail and the shear span/beam depth ratio is an important parameter. Therefore, a short discussion will be given here on this topic. Only a limited

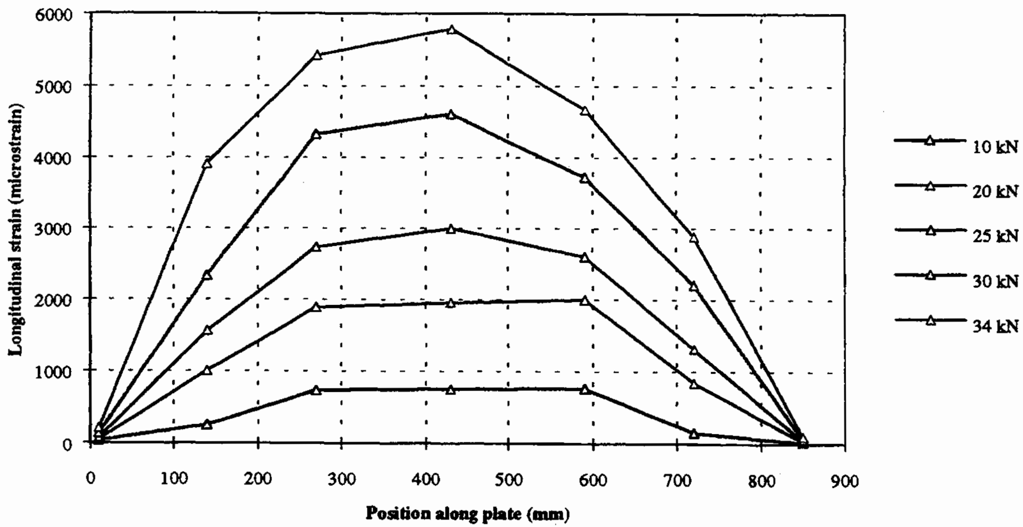


Figure 4.5 Typical distribution of longitudinal strain along the plate length at various applied load levels.

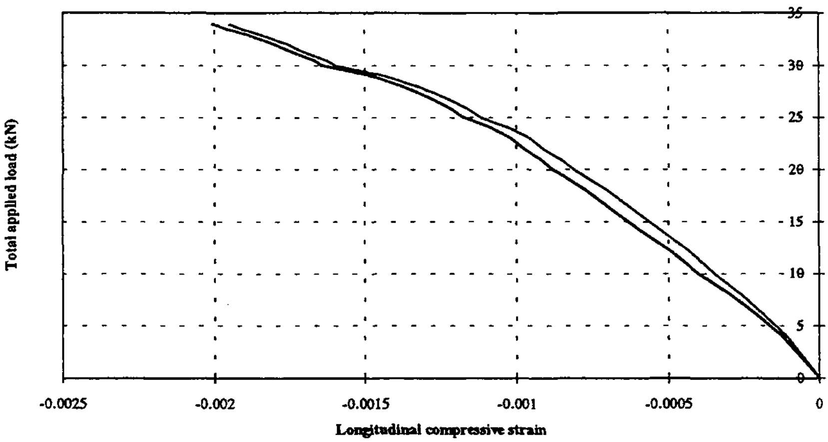


Figure 4.6 Typical compressive strain responses for strengthened 1 m long beam.

amount of work in this area has been undertaken during the ROBUST project. To enable a discussion to be given on the mode of plate separation, both composite and steel plates will be considered in this section in order to make general observations on this topic.

Jones *et al.* (1980) confirmed that very low shear span/beam depth ratios will result in catastrophic shear failure, as are also typical of unplated reinforced concrete beams under such loading (Kotsovos, 1986, 1984). Unplated shear span/effective depth ratios (the depth of concrete to the lowest level of tensile reinforcement) greater than 6.0 usually result in flexural failures. Ratios in the range 2.5 to 6.0 may be associated with flexure-shear cracking (Kong and Evans, 1987) in which a vertically rising flexural crack changes direction towards a loading position to become a shear crack. This was also noted by Garden and Hollaway (1996) in plated beams with a shear span/effective depth ratio of 4.76 (or a shear span/beam depth ratio of 4.0), indicating similarity to unplated beam behaviour. Shear bond failure of unplated beams, characterized by the destruction of the concrete/rebar bond due to the rebars being 'pressed down' under the applied shear force (Kong and Evans, 1987), occurs under lower shear span/beam depth ratios of 2.5–4.0. The action of the vertical step in Fig. 4.7 may be likened to a shear bond failure.

It has been noted in the ROBUST project, that as the shear span/beam depth ratio increases in value beyond 3.5, the position at which plate separation initiates shifts away from the free end of the plate (point *C* in Fig. 4.7). Investigative work at the University of Surrey showed that the plate generally would separate first at the location of the widest shear crack in the shear span (*AC* of Fig. 4.7) of potential failure; the initiation of plate

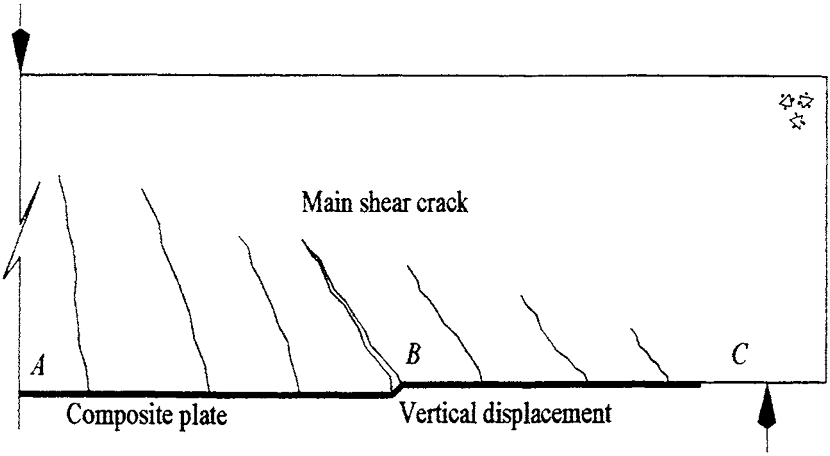


Figure 4.7 Initiation of plate separation for a beam of shear span/beam depth ratio of 3.5.

separation is shown at point *B* in Fig. 4.7. The base of the crack widens vertically to an extent which causes the layer of cover concrete to be pulled away from the external reinforcement at this position; Fig. 4.8 depicts this, where *AD* is the location of the main shear crack, *B* is the position of a subsidiary crack and point *C* is the position of the reaction. Furthermore, for shear span/beam depth ratio of greater than 3.5, the flexural and shear cracks would generally be more numerous but would be narrower than in a corresponding unplated beam. Under this condition the location of plate separation initiation would generally move further from the free end of the plate in the shear span as the shear span/beam depth ratio increases. Typically Fig. 4.9 shows that for a shear span/depth ratio of 4.0, the widest concrete crack, marked X, occurs near one of the loading positions; this crack would appear to be a flexural–shear crack. The whole thickness of the cover concrete separates from the internal rebars over a short length of plate adjacent to crack X. The damage at this location is likely to be due to the widening of the crack leading to the isolation of a triangular piece of concrete. Beyond this short length of exposed rebar the thickness of the separated concrete diminishes to a thin layer and would be composed of cement and weak aggregate/cement interfaces. At the end of the composite plate only traces of adhesive remain on the surface of the plate; the composite plate remains undamaged during the various stages of failure. At the position of the main crack, Fig. 4.9, the width of the separated concrete will generally be equal to the full width of the beam irrespective of the width of the composite plate. However, with reduced composite plate widths, the width of the separated concrete will tend to decrease to approximately the plate width with distance from the main crack; this is illustrated in Fig. 4.10 which presents the state of concrete cracking immediately before plate

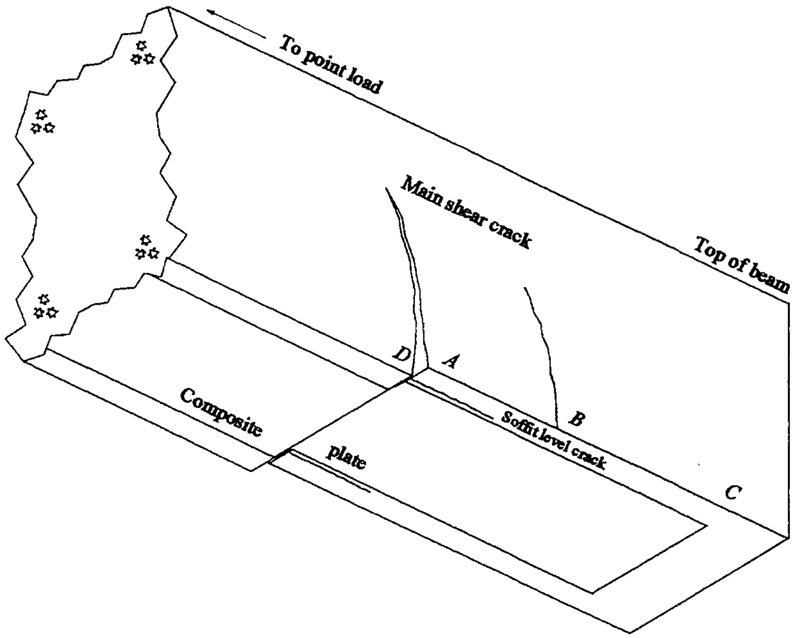


Figure 4.8 Exaggerated view of the concrete cracking immediately before plate separation (shear span/beam depth ratio of 3.5).

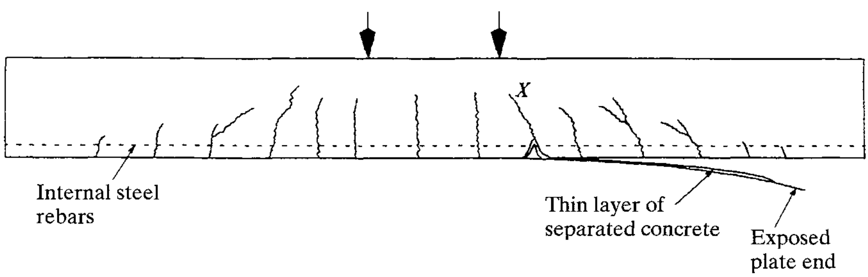


Figure 4.9 Typical mode of plate separation for a shear span/beam depth ratio of 4.0.

separation. The narrowing of the cracked width of concrete at the soffit level leads to the crack path marked A-B-C-D. Figure 4.11 shows a typical failure of the adhesive beyond the main flexural crack; the positions A-B-C-D in Fig. 4.11 correspond to those in Fig. 4.10. The investigative work at Oxford Brookes University under the ROBUST project (Rahimi, 1996) showed that for a shear span/beam depth ratio greater than 6, the failure mechanism is dependent upon the initial design of the reinforced concrete beam and that providing the shear loading capacity of the plate bonded beams is sufficient, the beam would tend to collapse by a sudden catastrophic failure, characterised by plate debonding followed by detachment

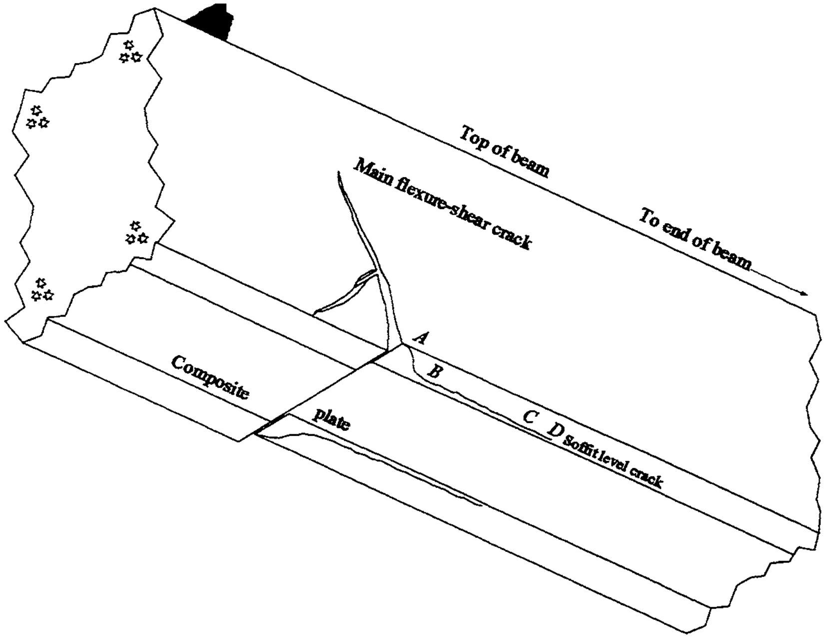


Figure 4.10 Exaggerated view of the concrete cracking immediately before plate separation (shear span/beam depth ratio of 4.0).

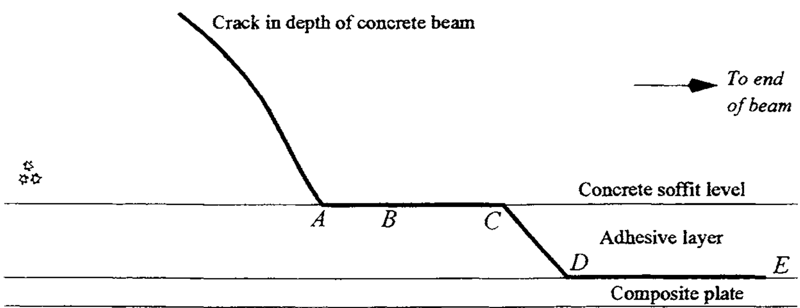


Figure 4.11 Appearance of cracking in the adhesive layer.

of the concrete cover in particular areas of the shear span and into the pure moment region.

In addition to the cracking of the concrete, cracks will be present in the adhesive layer but they will be of a random nature and will have been formed as a result of concrete shear cracking, itself a random nature phenomenon. The application of plate end anchorage does not appear to have any clear effect on the locations of the adhesive cracks although the beams with anchorage did experience a greater number of adhesive cracks in either the failed or non-failed shear span due to the greater opportunity for cracking by the higher ultimate capacities with anchorage. The point C, in Fig. 4.10, represents the soffit level crack being diverted into the adhesive. The crack CD then propagates further along the line DE, the adhesive/plate interface.

Under ROBUST, the University of Surrey (Garden *et al.*, 1998) undertook a series of tests using cantilever specimens to allow the mode of plate separation to be observed under shear span/beam depth ratios up to a value of 8.00; this enabled confirmation of the influence of plate end anchorage under low and high ratios. Figure 4.12 shows the effects of bolted plate end anchorage under a low and a high shear span/depth ratio. The maximum applied load on the cantilever becomes lower with increasing shear span/depth ratio but the maximum bending moment (i.e. at the encastre support) becomes greater as in the four point bending tests. The figure shows the effect of the bolted plate end anchorage under a low and a high shear span/beam depth ratio and the anchorage clearly has a much greater influence under a low ratio. The increases in capacity due to bolted anchor-

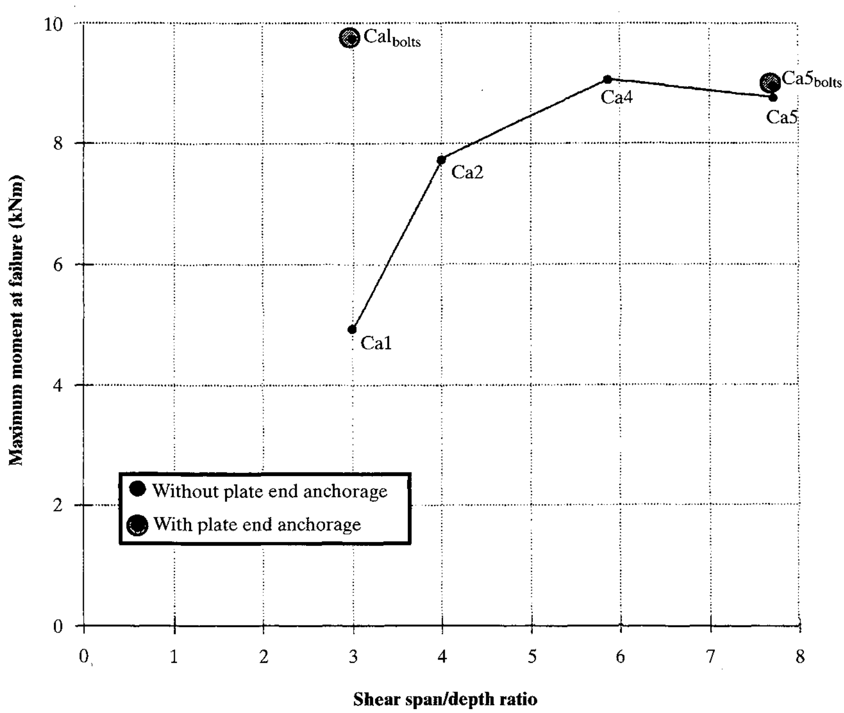


Figure 4.12 Variation of ultimate bending moment with shear span/beam depth ratio in 1 m long beams.

ages are shown to be 97% and 2% under the ratios of 3.00 and 7.72, respectively.

It may be observed that under a shear span ratio greater than 4.00, plate end anchorage serves no purpose other than to provide a layer of material (i.e. a net) that retains broken concrete after plate separation. However, under the lowest shear span/beam depth ratio of 3.00, the anchorage serves this purpose, but in addition and more importantly, considerably enhances the ultimate capacity and member stiffness. Shear span/depth ratios greater than 4.00 are associated with randomly located adhesive cracks. Based on the cantilever results given in Garden *et al.* (1998), the following are given:

- $a_v/h = 3.00$ plate end peel initiated by the occurrence of a shear crack at the end of the plate;
- $a_v/h = 3.40$ shear-bond separation of the plate under the action of a shear crack;
- $a_v/h = 4.00 - 7.72$ shear-bond separation of the plate under the action of a flexure-shear crack, perhaps better referred to as flexure bond.

Table 4.1 summarizes the ratios that have been related to the modes of plate separation by other researchers. The references have provided sufficient detail regarding the loading configuration and the mode of plate

Table 4.1 Modes of plate separation

Reference	Shear span/beam depth ratio	Plate material
Failure initiated at the end of the plate		
Van Gemert and Maesschalck (1983)	2.22	Steel
Swamy *et al.* (1987)	3.00	Steel
Swamy and Jones (1990)	3.00	Steel
Ritchie *et al.* (1991)	3.00	Composite
Oehlers (1992)	3.14	Steel
Sharif *et al.* (1994)	2.62	Composite
Hussain *et al.* (1995)	2.67	Steel
Quantrill *et al.* (1995)	3.00	Composite
Failure initiated in shear span		
Jones *et al.* (1988)	3.41	Steel
Takeda *et al.* (1996)	3.40	Composite
Failure initiated near load position		
Irwin (1975)	4.02	Steel
Jones *et al.* (1982)	5.00	Steel
Macdonald and Calder (1982)	4.52	Steel
Meier and Kaiser (1991)	4.40	Composite
Rahimi (1996)	>6.0	Composite
Takeda *et al.* (1996)	4.0	Composite

separation to enable comparisons to be made with the ROBUST failure modes; these references are related to tests of both steel and composite plated beams.

4.3.2 Effect of plate geometry

The ROBUST project investigated the effects that the plate sectional geometry and the plate end geometry had on the strength of the plated beam. To undertake this task prepreg tapes of unidirectional carbon fibre reinforced polymer (CFRP) composites were used. The main reason for this choice of composite was the versatility of the material for the experimental fabrication; in the uncured condition, the thickness and shape of the material could readily be altered. The thickness of the CFRP laminate was 0.2 mm and these could be built up to any desired thickness. The shaping of the uncured material could be undertaken by cutting.

4.3.2.1 Plate sectional geometry

The plate aspect ratio was investigated to determine the effect of varying the plate thickness and width whilst keeping the plate area constant (Garden *et al.*, 1997).

The tests were conducted on a number of identical 1 m long reinforced concrete beams with shear span/beam depth ratio of 3.0, 3.4 and 4.0. Table 4.2 presents stiffness, ductility and strengthening data for the beams. Ductility is defined as the ratio of ultimate deflection to deflection at internal

Table 4.2 Beam stiffness, ductility and strengthening data

Beam series	Beam	Plate size (mm)	Plate anchor method	Ductility	Postcrack stiffness (kN mm^{-1})	Postyield stiffness (kN mm^{-1})	Strengthening at failure: % by moment
1	Control	None	None	6.76	4.17	0.20	0.0
3	Control	None	None	9.30	3.35	0.22	0.0
1	$1A_u$	0.5 × 90	None	3.15	9.09	2.25	0.0
	$1B_u$	0.7 × 65	None	2.92	6.92	3.07	114.7
	$1C_u$	1.0 × 45	None	1.78	5.23	2.62	87.6
2	$2A_u$	0.5 × 90	None	3.17	5.88	2.00	156.7
	$2A_a$	0.5 × 90	Supports	6.23	5.97	2.73	237.7
	$2B_u$	0.7 × 65	None	2.50	5.57	2.37	126.7
	$2B_a$	0.7 × 65	Supports	5.65	6.05	3.21	231.0
	$2C_u$	1.0 × 45	None	2.78	5.59	2.91	137.0
	$2C_a$	1.0 × 45	Supports	5.76	5.88	1.96	174.0
3	$3A_u$	0.5 × 90	None	5.49	6.28	2.45	205.9
	$3B_u$	0.7 × 65	None	4.41	5.60	2.29	170.6
	$3C_u$	1.0 × 45	None	2.52	4.96	2.46	140.8

reinforcement yield. This quantity is appropriate because yielding of internal reinforcement marks the onset of relatively large section rotations and, consequently, large and widespread cracks which are associated with failure of the beams. All percentages of strengthening quoted in Table 4.2 are based upon taking an ultimate moment of 2.55 N m, which was achieved by the control beams. The plate types were designated A, B and C for plates of 90 mm × 0.5 mm, 65 mm × 0.7 mm and 45 mm × 1.0 mm, where the first and second dimensions are the width and thickness, respectively.

From Table 4.2 it can be seen that the ductility generally increases with increasing shear span/beam depth ratio for a given plate width. Furthermore, the failure load falls as the plate width decreases for the beam series 1, 2 and 3 whether the beams have end anchorage or not.

4.3.2.2 Plate end geometry

In coupon single lap joints specimens, the tendency for the system to fail by peel stress is reduced by tapering the adherends. ROBUST, therefore, undertook an investigation into composite plate end geometry and studied the effect on the overall behaviour of the plated beams.

In the study unidirectional carbon fibre prepreg composite plates were utilised to facilitate the reduction in width and the fabrication of the stepped section. Two separate investigations were undertaken (Rahimi, 1996); the first used plates 150 mm wide reduced to 50 mm over a distance of 300 mm and the second investigation allowed for the tapering of the 150 mm wide plates from 0.78 mm to 0.2 mm thick in four equal stages over a distance of 30 mm at the plate ends.

It was shown that the strength of the plated beams with tapered (in plan) bonded laminates were not significantly different from those with square-ended plates; on average the beams were slightly weaker than those with square ends, all other geometric parameters for the two types of beams being equal. Furthermore, the beams strengthened with composite plates which were tapered (in section) over their thickness performed in a similar way to the strengthened beams without tapers and with square-ended plates.

Further work at Oxford Brookes University investigated the characteristics of the plated systems as the thickness of the CFRP composite plate increased. Rahimi (1996) showed that the strengths of the beams as the plates were increased in thickness did not vary linearly. The tests showed that there is a diminishing return in the beam strength as the thickness (and therefore area) of the composite plate increases.

The type of failure associated with the 2 ply (0.2 mm thick) CFRP plate was quite different from the 6 ply (1.2 mm thick) CFRP plate when both were bonded to similar RC beams. In the former case, failure occurred with no visible cracks appearing at the plate ends and when based on observed failure patterns and numerical predications it is likely that failure initiated near the point loads. In the latter case, cracks extended to the plate ends and ultimately the plate delaminated with lumps of concrete being pulled away from the plate end and also at the middle of the shear spans. The failure prediction in this case is that the CFRP plated beam might have been initiated either at the plate end or near the point load.

4.3.3 Effect of anchorage on plated beams

In this section the effects of using bolts as an anchorage system at the ends of composite plates in plated beams will be discussed. The use of bolts as anchorage is common when steel is used as the plating medium in strengthening applications, both to resist forces tending to peel the plate away from the beam and to support the plate should the adhesive bond fail. Deblois *et al.* (1992) is currently the only reported published literature on the use of FRP for external RC strengthening which utilises bolts for plate end anchorage. The effect of introducing holes into the unidirectional FRP section requires careful design.

The ROBUST project investigated four plate end anchorage systems, these were:

- continuation of the CFRP plate under the support,
- a bolted system based on steel bolts inserted through the CFRP plate and into the concrete, Fig. 4.13,
- a clamping force at the ends provided by mechanically attached steel clamps, Fig. 4.14,
- improved attachment of the CFRP plate to the concrete member using bonded glass fibre reinforced plastic (GFRP) angle sections, Fig. 4.15.

Although all of the above plate end anchorage systems, except the mechanical clamps, displayed the ability to secure the end of the CFRP plate and thus prevent plate end movement after plate separation, the bolted system was the one that was found to be both practical and effective. Figure 4.16 gives the response of total applied load against midspan deflection for bolted, and under support-anchored conditions and a non-anchored situation.

The design of bolted connections in composite materials is much more complex than with standard structural materials such as steel, since fibre reinforced materials can be weakened to a greater degree by the introduction of holes. As with other mechanically fastened joints, the introduction of discontinuities reduces the cross-sectional area, thereby lowering the load required to fail the material in tension. Furthermore, reinforcing fibres become severed and high stress concentrations are introduced in the region of such discontinuities, as a result of the anisotropy of the material; these

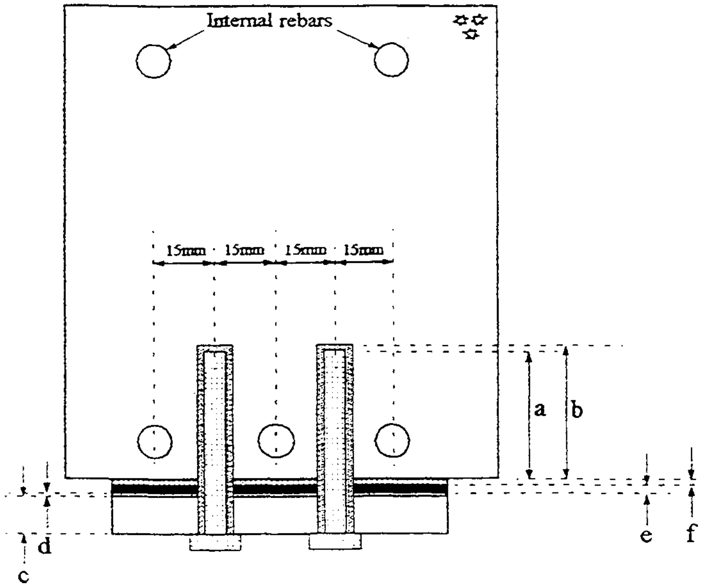


Figure 4.13 Dimensions of the bolted anchorage in the 2.3m beams. (a) 61 mm bolt penetration, (b) 65 mm drilled hole for bolt, (c) 15 mm thick GFRP anchorage block, (d) 1 mm thick layer of 3M adhesive, (e) 12 mm thick CFRP plate, (f) 2 mm thick layer of Sikadur 31 PBA.

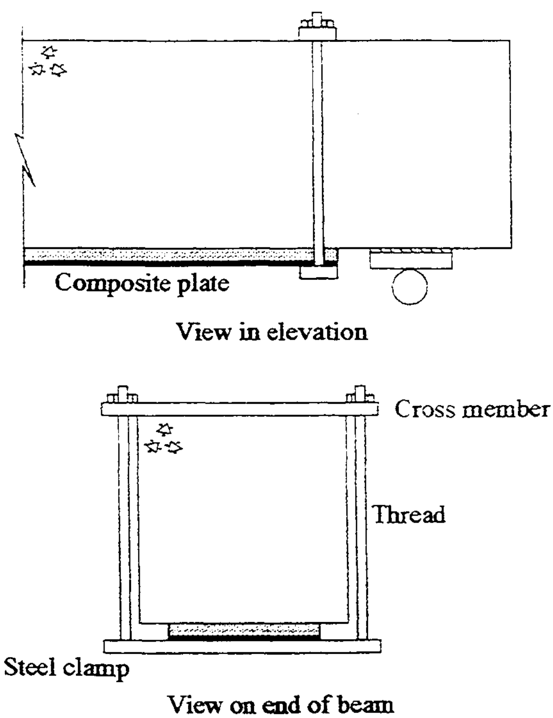


Figure 4.14 Mechanical clamping system.

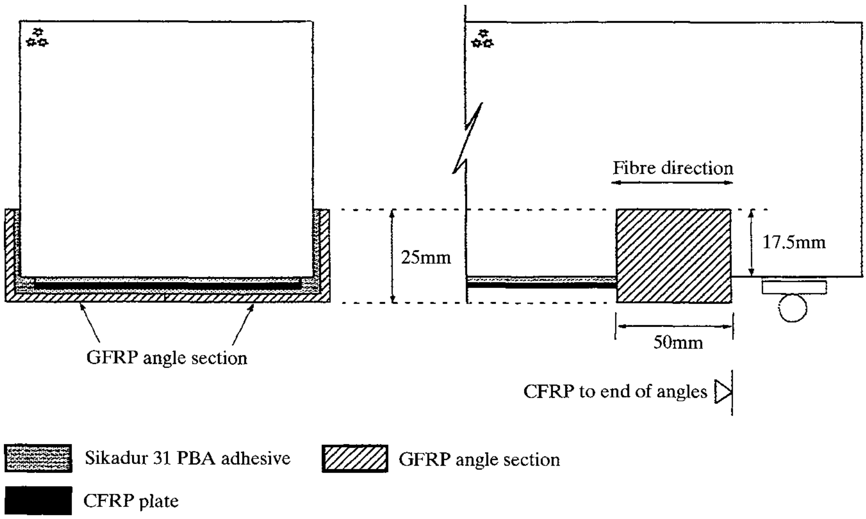


Figure 4.15 Plate end anchorage by externally bonded angle sections.

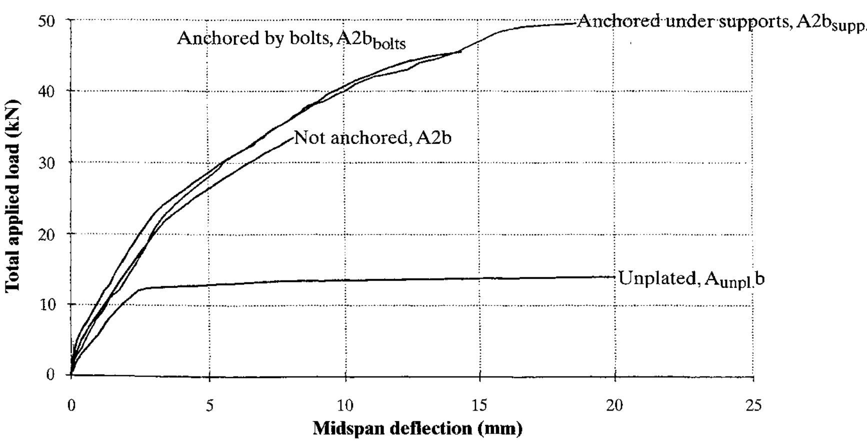


Figure 4.16 Deflection responses for 1 m long beams with 0.82 mm × 67 mm plate.

promote fracture. Although FRP materials exhibit some stress concentration relief in the form of matrix cracking, such as fibre matrix debonding, fibre breakage and fibre pull-out, all resulting in behaviour which is not perfectly elastic, no yielding capability is present. As a result of this lack of plasticity, the tensile elastic stress concentration due to a circular hole in a unidirectional sheet can be more than three times that associated with isotropic materials, increasing with the ratio of member width to hole diameter. A detailed description of the strength of mechanically fastened

FRP joints is given by Collings (1987), with a theoretical stress analysis presented by Matthews (1987). The design of such joints was examined by Hart-Smith (1987). Shear pull-out failure can occur at relatively low loads, particularly in pultruded materials due to the high percentage of longitudinal fibres and appears as splitting of the composite in the fibre direction behind the holes.

Clearly, drilling holes through FRP is undesirable, consequently it is necessary to bond an endplate on to the composite soffit plate, on to which the bolt heads would bear and would aid the redistribution of the tensile and the longitudinal stresses in the composite soffit plate away from the holes. The use of a combined bonded and bolted anchorage system reduces the stress concentrations that exist around the bolt holes in the laminate and also reduces the possibility of fatigue problems for the bolt fasteners alone. The length of overlap of the endplate with the composite soffit plate must be sufficient to prevent the latter 'pulling out' as a result of shear pull-out failure of the laminate beyond the bolts. The load transferred to the endplate is supported by the bolts through dowel action; the bolts also resist normal stresses acting away from the surface of the beam, thought to be associated with the peel stresses at the plate ends. The length of bolt which penetrates the concrete must be sufficient to provide a sound anchorage, preventing pulling out of the bolts normal to the beam as the load increases.

The ROBUST anchorage system consists of bonding pultruded carbon fibre plates on to the soffits of RC beams and then making an informed engineering judgment about whether the system will need to be anchored at the extreme ends of the plate, bearing in mind the comments made in Section 4.3.1.

All anchor plates, if used, on 2.3 m, 4.5 m and 18 m beams in the ROBUST project were manufactured from GFRP composites. These anchor plates were laminates with ±45 glass fibre lay-up and were roughened and degreased with solvent before bonding. They had a thickness of 14 mm and had a width of the same value as the composite plate; thus when bonded in place they covered the full width of the soffit plate. They were bonded in the correct location at the composite plate ends using an adhesive manufactured by 3M (1990) after having been predrilled with holes of 12 mm diameter. Figure 4.13 shows the arrangement of the anchorage for the 2.3 m long beam used in the laboratory.

After the 3M adhesive was cured sufficiently the bolt holes were drilled into the concrete using a masonry drill bit of 10 mm diameter. The anchorage plate thus acted as a template for drilling the bolt holes in the concrete. After drilling, the resulting holes were blown out to remove dust and debris and were then filled with adhesive. The high yield bolts of 10 mm diameter were thoroughly degreased with solvent to remove contaminants and pushed into the holes whilst being rotated to ensure the threads were covered with adhesive. The bolts, therefore, were not used in the normal sense since no tightening against a thread was carried out; the same adhesive was used as that to bond the composite plate to the soffit beam.

A comparison of the deflection responses of beams with and without plate end anchorage for each plate cross-section considered in ROBUST showed improvement in stiffness beyond yield over the unanchored plates. The anchorage system which was developed by extending the plate under the supports involved a normal force equal to the support reaction acting against the CFRP plate sufficient to prevent plate slippage. Although this anchorage system applied a high clamping force and the bolted system applied none, the difference in ultimate capacity was not large and there was no difference in the deflection response per unit applied load. The remaining two systems (namely, a clamping force at the plate ends and the attachment of GFRP angles at the plate ends) were not as efficient as the above and consequently the results will not be discussed here.

The following observations are made from test results derived from the ROBUST project regarding the plate end anchorage:

- When the plate ends are anchored, the shear strength of the concrete may become the limiting factor governing the ultimate capacity of the beam under a low shear span/depth ratio, due to the elimination of plate peel and the associated large increase in applied load.
- The failure mode is not affected by plate end anchorage when separation is governed by a vertical step at soffit level, either at the location of a shear crack or a flexural-shear crack.
- The plate end anchorage must take the form of a system securely anchored to or in the concrete member itself, rather than merely being bonded to the outside of the member.
- The ROBUST bolted anchorage system is practical and has the same effect on the failure mode as anchorage under the beam supports associated with a clamping force equal to the support reaction (namely the shear force).
- Under a low shear span/beam depth ratio, plate end anchorage is effective because it suppresses the normal component of plate end peel and prevents the shear component from damaging the concrete prematurely.
- Confidence can only be placed in the ability of the bolted anchorage to restrain the shear component if the bolts are placed through the composite plate via a bonded anchorage block. When the bolts are outside the plate width, there will be no guarantee that the plate will not slip.
- The bonded anchorage block ensures the composite plate sustains no

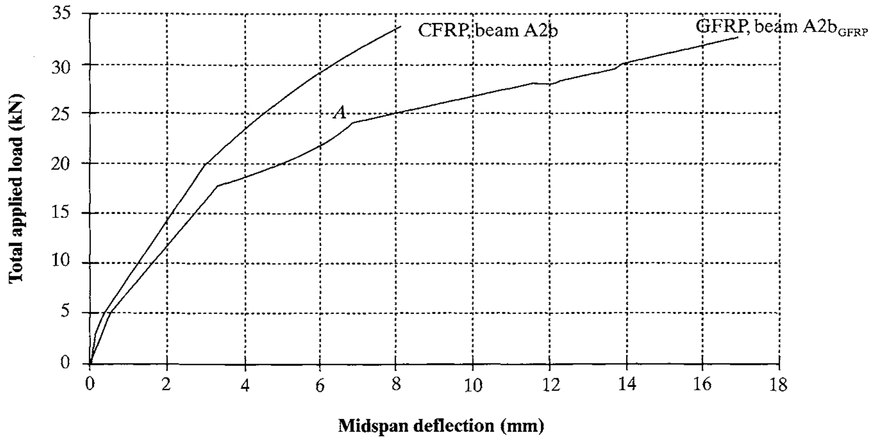


Figure 4.17 Deflection responses of the CFRP and GFRP plated beams.

damage due to the introduction of the drilled hole which is associated with high localised stress concentrations.

4.3.4 Effect of increasing the plate stiffness

The ROBUST project showed that GFRP plated beams were associated with lower neutral axis depths compared with CFRP plated ones. Figure 4.17 shows a typical comparison of the midspan deflection responses of the CFRP and GFRP plated beams, indicating the relatively high structural stiffness created by the greater modulus CFRP plate. In this particular test, the concrete began to fail in compression at an applied load of 24 kN in the GFRP plated case, causing a reduction in member stiffness at the point marked A in the figure, but this compressive failure did not propagate further until shortly before plate separation, allowing the beam to maintain its current stiffness up to failure. Both members failed by the same mode of plate separation (namely, initiated by the vertical step at the base of a shear crack, at similar applied loads).

4.3.5 Effect of adhesive thickness

The effects of adhesive thickness were assessed by comparing the responses of both a high strength (28 day cube strength, 70 N mm^{-2}) and a lower strength (28 day cube strength, 40.0 N mm^{-2}) concrete beam. It was found that for both sets of beams, the flexural responses obtained were very similar for a 1 mm and 2 mm adhesive thickness bondline which suggests that this parameter has little effect on the overall structural behaviour of the plated beams. A greater thickness of bondline than would normally be recommended to bond two similar or dissimilar adherends together is required in plate bonding in order to even out the irregularities in the soffit of the concrete beams.

4.3.6 Effect of concrete compressive strengths

The effects of concrete compressive strengths on structural behaviour were investigated by comparing the results from beams which were manufactured from two different strength concretes; both types were plate bonded. The results showed that strengthening the high strength concrete increased the serviceability limit, the yield load and the maximum load carried in comparison to the lower strength beams. The strengths over a comparable unplated control beam were also enhanced at each load level; the plated beams manufactured from the higher strength concrete resulted in serviceability load increases of around 40% over the unplated beam, compared to 35% for the plated beams with the lower strength concrete, whilst the yield load was increased by 131% compared to the 115% increase obtained from the plated beams manufactured from the lower strength concrete.

For a given applied load, beams made from the higher strength concrete deflected less than those of the lower strength concrete; the post-cracking stiffness of the plated beams was almost 50% higher than those of the lower strength and gave increases compared to the unplated response of over 40% as opposed to 24% for the lower strength concrete beams.

4.4 Discussion

There are a number of important general observations, regarding the strengthening and stiffening of beams, derived from the laboratory investigations:

(a) when plate end anchorage is not utilised:
- It is advantageous to use a plate with as high a stiffness as practicable (Figure 4.17 and Rahimi (1996)).
- It is advantageous to use a wide plate, when plate separation is an issue, to improve the ultimate capacity of the beam and to avoid early failure caused by peeling of the plate. It should be noted, however, that the failure will be more brittle than for a narrower plate width (Table 4.2).
- The plated beams are associated with lower maximum deflections and lower strains throughout the section due to increase in the flexural rigidity of the section (Fig. 4.3).

- The plating of beams improves their stiffness, their yield and their ultimate capacities.
- The plated beam compared to an unplated one will support a higher serviceability load and the overall deflections of the beam at this load will be lower.

(b) when plated anchors are used, there are improvements over and above those in item (a); these improvements relate mainly to low shear span/beam depth ratios below the value of 4.0:

- The plate end anchorage increases the strength of the beam, at low shear span/beam depth ratios, by raising the postyield structural stiffness (defined by applied load on beam/central deflection of beam), a greater applied load is required to reach the critical deformation associated with plate separation.
- The structural stiffness will only be raised by plate end anchorage if the composite action, between the beam and plate, is maintained throughout the shear span.
- The end plate anchorage increases the failure load and increases the ductility in comparison to corresponding unanchored specimens allowing higher maximum overall deflections (Fig. 4.16) and maximum plate and concrete strains to be attained, and in addition higher longitudinal strains to be developed towards the plate ends compared with the unanchored case, thus producing a more uniform distribution of strain along the length of the external plate.
- Although the anchorage has a negligible influence on the member stiffness under a high shear span/beam depth ratio, the anchorage could act as a 'net' under the beam in the event of failure.

Further observations are that beams, under four point loading and without plate end anchorage, will generally have a premature failure, by plate separation due to plate end peel occurring under low shear span/beam depth ratios only; this is not a typical failure mode. The failure mode changes with increasing shear span/beam depth ratio by a shift in the position at which plate separation is initiated away from the plate end and towards the constant moment region.

The concrete will suffer horizontal cracking along the level of the soffit at elevated loads, under the relatively high shear span/beam depth ratios which lead to plate separation being initiated at a principal shear crack or at a flexure-shear crack, and the soffit crack will penetrate the adhesive at apparently random locations. The cracks through the thickness of the adhesive may propagate as adhesive/plate interfacial cracking, a clearly detrimental effect for which the governing mechanism remains to be fully understood.

The greatest ultimate capacity of a plated beam, with a given cross-sectional area of plate, is achieved using as wide a plate as possible, in which case the brittle failure mode of plate separation will be delayed until a late stage in the loading range. Under high shear span/beam depth ratios, wide plates are still preferable since, to maximise the ultimate capacity, wide plates distribute the flexural strain over the whole width of the section, unlike narrow plates where the strain in the plate is suddenly converted into tensile cracks in the adjacent soffit concrete not covered by the plate.

With respect to the anchored system at the ends of the plates, it is not necessary to apply a force against the plate since the principal objective, which is to prevent the shear and normal disturbances of the plate ends, is met using the bolted system. The global stiffening influence of plate end anchorage, due to the improved concrete/plate composite action, comes into greatest effect after yield of the internal rebars, well beyond the serviceability load. Since the applied load on a plated member will generally be limited to the serviceability value, little static benefit of anchorage appears to be gained.

Part B Field investigation

As has been described in Part A, the materials technology developed during the research phase of the ROBUST project has been applied, in the laboratory, to enable parameter studies of the strengthening of numerous reinforced concrete beams to be undertaken. However, for the future exploitation, implementation and specification of the technique by potential clients the viability of the technique was demonstrated on full scale beams. Furthermore, such full scale testing would provide the opportunity to validate a theoretical model which had been developed for the analysis of beams strengthened in flexure by the bonding of composite plates; this numerical work is discussed in Chapter 8. The results of such work can then be applied in the formulation of a design method and specification.

4.5 Testing programme for 18m beam

4.5.1 Test beams

Ten 18m long prestressed concrete beams were selected from a number of such beams which had been removed from a deteriorated highway bridge in Oxfordshire. The original three span bridge was constructed using about 50 rectangular prestressed concrete beams per span placed side-by-side in a simply supported manner. Composite action was achieved by using an *in situ* reinforced concrete top slab and transverse prestress. Dismantling of the bridge involved removal of the top slab and cutting longitudinally between adjacent beams. Selection of suitable beams for the test programme was based largely upon their condition, the main criteria being no evidence of tendon movement at end anchorages, as little damage to the concrete as possible and suitable faces for subsequent placing of instrumentation.

A typical end elevation and midspan section through the test beams are shown in Fig. 4.18. The beams were 710 mm deep and 380 mm wide at the base, reducing to 340 mm at the top. The original bridge drawings dated 1957 specify an A6000 concrete mix (41.4 N mm^{-2}), although for analysis purposes a compressive strength of 50 N mm^{-2} was assumed to allow for the effects of ageing. Conventional unstressed reinforcement comprised three 12 mm diameter mild steel bars on top and bottom faces with 10 mm diameter stirrups at 450 mm centres for shear reinforcement.

Each beam was prestressed longitudinally using five Gifford Udall 12 wire grouted tendons, their centroid being located 125 mm from the base at

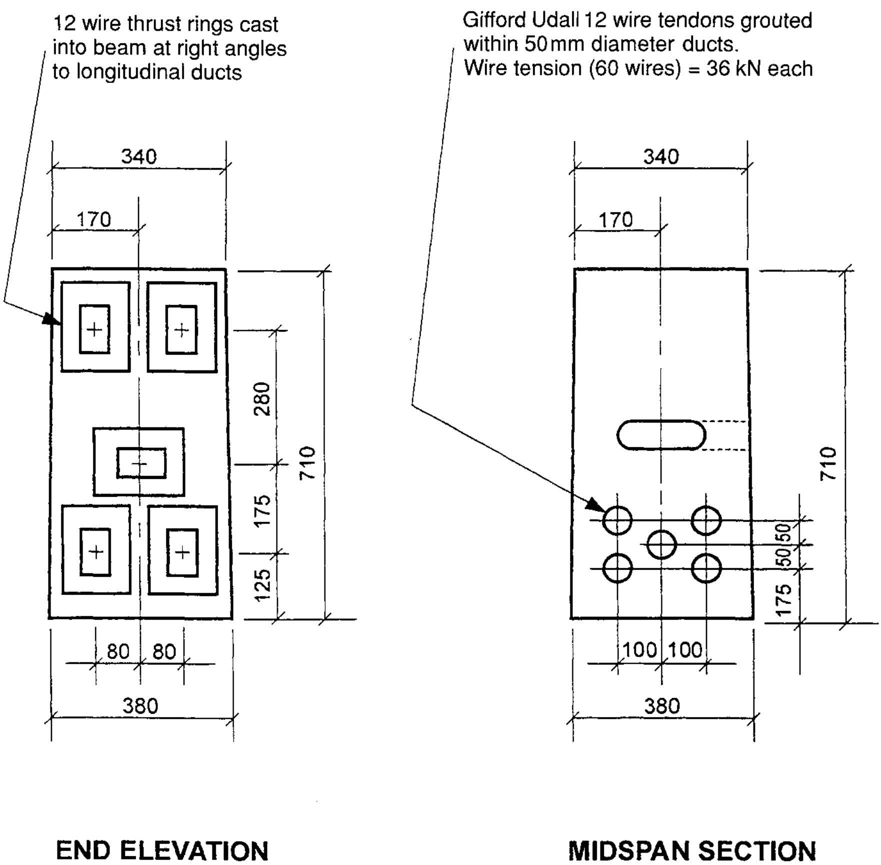


Figure 4.18 End elevation and midspan section of 18 m beams.

midspan. The nominal wire tension specified on the drawings was 8083 lb (36 kN), giving an overall prestressing force of 2160 kN.

Analysis of the midspan section using conventional theory for prestressed concrete showed that the prestressing tendons had not yielded when the ultimate moment capacity was reached, that is, the beams were significantly over-reinforced. This is not unexpected since the beams had originally been designed to function compositely with an *in situ* concrete top slab. To allow these beams to be used as a suitable test bed for the ROBUST strengthening system, it was necessary to attempt to destress the lower three tendons. This would have the effect of reducing the ultimate moment capacity by about 60% and hence allow for the use of up to three layers of composite plates during the strengthening operations.

The lower three tendons were destressed by coring through the beams horizontally from both sides at four locations. The core locations are shown in Fig. 4.19 in relation to the four point bending loading configuration to be used in the tests. Their positions were chosen to provide destressing over the complete length of the constant moment zone but to avoid the loading positions. The 1.5 m longitudinal spacing of the cores was based upon the calculated transmission length of the tendons.

4.5.2 Testing rationale

The variables to be investigated by the test programme were selected as follows:

(a) Plate area: variation in plate area was achieved by using between one and three layers of ROBUST CFRP composite plates, each layer consisting of three plates each of dimensions 90 mm × 1 mm. Plates were bonded in place using either the adhesive Sikadur 31 PBA or Sikadur 30.

(b) Unstressed/stressed plates: eight beams were strengthened with unstressed CFRP plates and two with stressed plates. The performance of the beams with prestressed plates will be discussed separately in Chapter 5.

(c) Plate length: four plate lengths, nominally 6.0 m, 9.25 m, 12.5 m and 15.8 m were used.

(d) Stressed state prior to plating: in five of the beams with unstressed plates and in two beams with stressed plates, three prestressing tendons were removed by coring prior to plating as described above. In these beams, load was applied to the unplated beams within the elastic range to establish the unplated stiffness. In the remaining three beams the three tendons were severed after plating and the plates monitored for stress transfer. This latter scenario was designed to study the

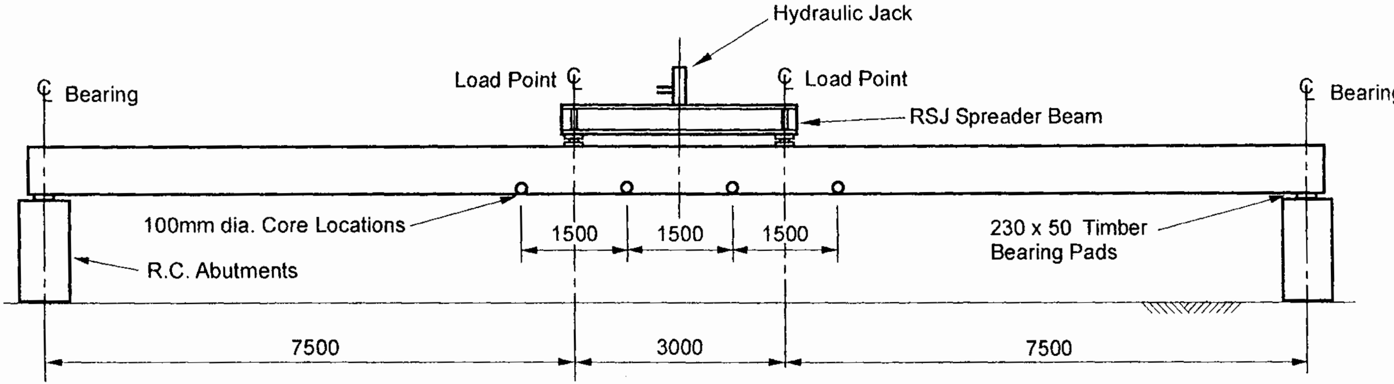


Figure 4.19 Core locations in relation to loading configuration.

Table 4.3 Test parameters for 18 m beams (unstressed plates)

Beam no.	Overall plate dimensions (mm)	Free/ Anchored plate ends	Plate length (m)	Maximum unplated loading (kN)	Maximum plated loading (kN)	Monitored during coring
1	270 × 1	F	12.5	62.5	77	
2	270 × 2	F	15.8	62.5	118[1]	
3	270 × 1	F	6.0	62.5	60[1]	
4	270 × 1	F	15.8	62.5	72	
5	270 × 1	A	6.0	62.5	75	
6	270 × 2	A	6.6	—	87	✓
9	270 × 3	F	15.3	—	130[1]	✓
10	270 × 1	F	9.25	—	75	✓

[1] Beams failed under core holes by plate debonding.

ability of the external CFRP reinforcement to maintain the structural integrity of the beam in the event of tendon damage due to corrosion or accidental impact.

(e) End anchorage conditions. For the short unstressed plates, both free and anchored ends were provided in order to investigate control of possible shear-bond failures. For medium and long plates, anchors were not provided in order to establish whether shear-bond failures occurred.

The test parameters are summarised in Table 4.3.

4.5.3 Testing arrangements

Load testing of these full scale beams was conducted at a purpose built site in Oxfordshire by the Royal Military College of Science. Reinforced concrete abutment walls 1.5 m high and 1.0 m wide were constructed at 18 m centres as illustrated in Fig. 4.20 and the beams placed in position side-by-side on hardwood boards to the top of the support walls. A 1.25 m square reinforced concrete anchor block was cast with its surface at ground level along the centre of the testing bay. For each beam to be tested, four Dividag bars were cast into the anchorage block at 0.5 m centres and provision made for couplers at ground level to extend these to the complete height of the loading frame. Load was applied through a hydraulic jack which reacted against the frame and hence via the Dividag bars into the anchor block at ground level. The entire loading frame and tension anchorage system was designed for a capacity of 40 tonnes.

Load from the jack was applied to the beams through a steel spreader beam to two loading points at 3.0 m centres as shown in Fig. 4.19. Thus

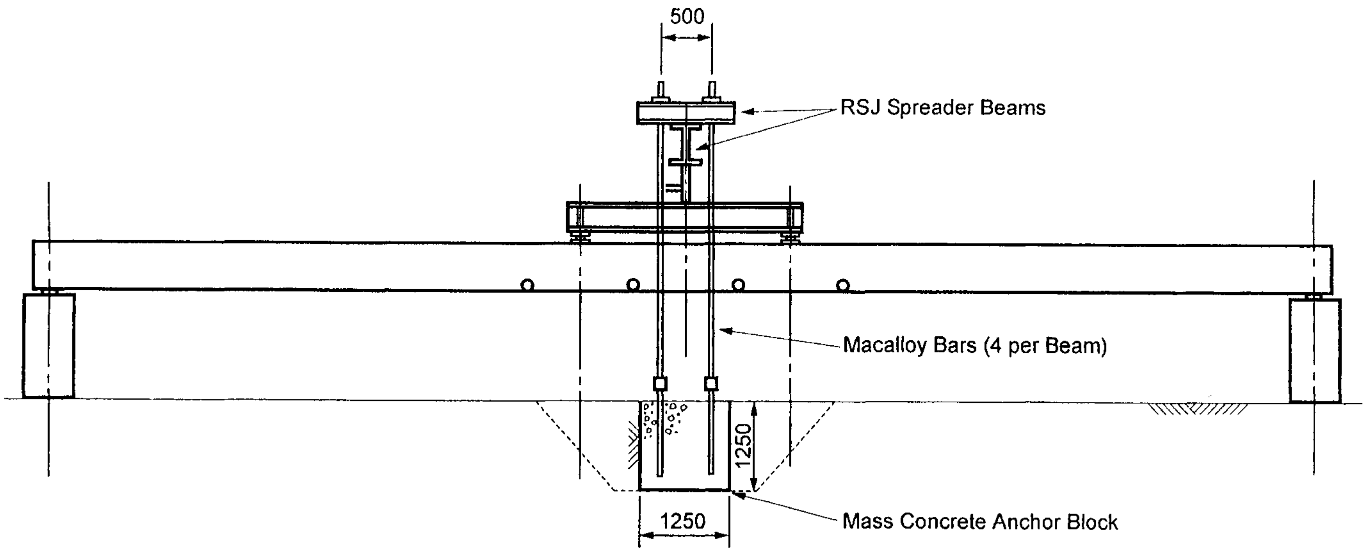


Figure 4.20 Load testing arrangement.

the beams were loaded in four point bending with a shear span to overall depth ratio of 10.5. The entire loading frame, jack and spreader beam were capable of being winched from one beam to the next as the testing programme progressed.

The instrumentation attached to each beam is illustrated in Fig. 4.21 and is summarised below:

(a) Load: this was measured using a load cell located between the jack and the spreader beam.
(b) Deflections: these were measured at quarter and midspan points using linear variable displacement transducers (LVDT).
(c) Concrete strains: vibrating wire gauges (VWG) were positioned on one side face of the beam to monitor horizontal concrete strains and hence to determine the distribution of strain over the depth of the section and the location of the neutral axis. These gauges would not be able to span a number of cracks and hence a pair of LVDTs connected by a hollow steel bar were attached on the opposite face to provide a long length strain monitoring device in the tension zone. These automatic strain monitoring devices were supplemented by the use of a crack width gauge and a vernier gauge to measure the relatively large strains across the cracks which developed at the core locations.
(d) CFRP plate strains: these were measured using electrical resistance strain gauges (ERS) attached to the external surface of the plates. Typically ERS gauges were placed at the end, quarter and midspan points and under the core locations on each plate.

The output from the load cell, LVDTs, VWGs and ERSs was channelled via a datalogger to a computer programmed to provide for continuous monitoring throughout each load cycle.

4.5.4 Results and discussions

Figure 4.22 shows the load–deflection plot for beam 4 prior to plating. It can be seen that after the first load cycle there was a residual deflection of 26 mm which is believed to be due to further debonding of the tendons between core positions during loading. A second load cycle was conducted at the end of which there was very little further residual deflection.

The corresponding load–deflection plot for the same beam 4 after plating is shown in Fig. 4.23. This beam had been strengthened by a single layer of plates over 15.8 m. Three cycles of load were applied up to a maximum of 72 kN, this representing 80% of the predicted ultimate load. The repeatability of behaviour between the three cycles is clear.

The unplated and plated plots have been superimposed in Fig. 4.24 and compared with theoretical predictions from three-dimensional finite

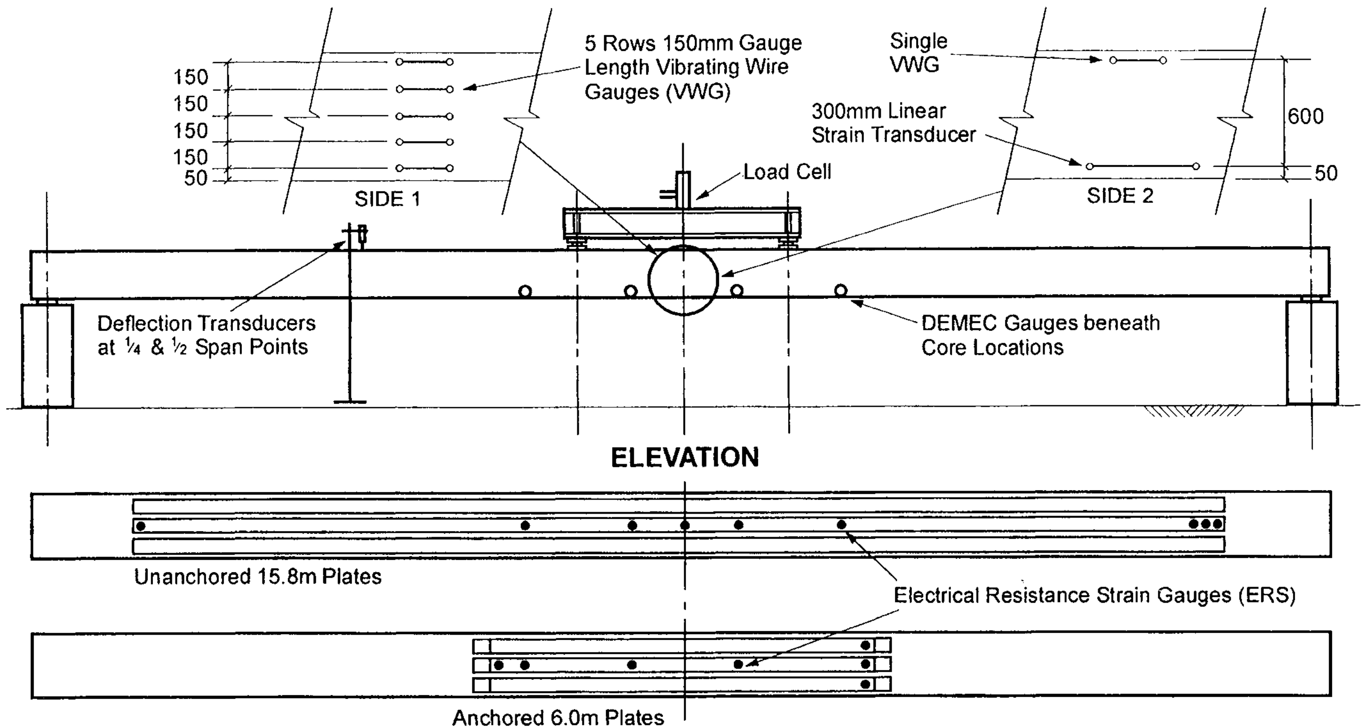


Figure 4.21 Typical plans on beam soffit showing plate gauges and instrumentation attached to beams.

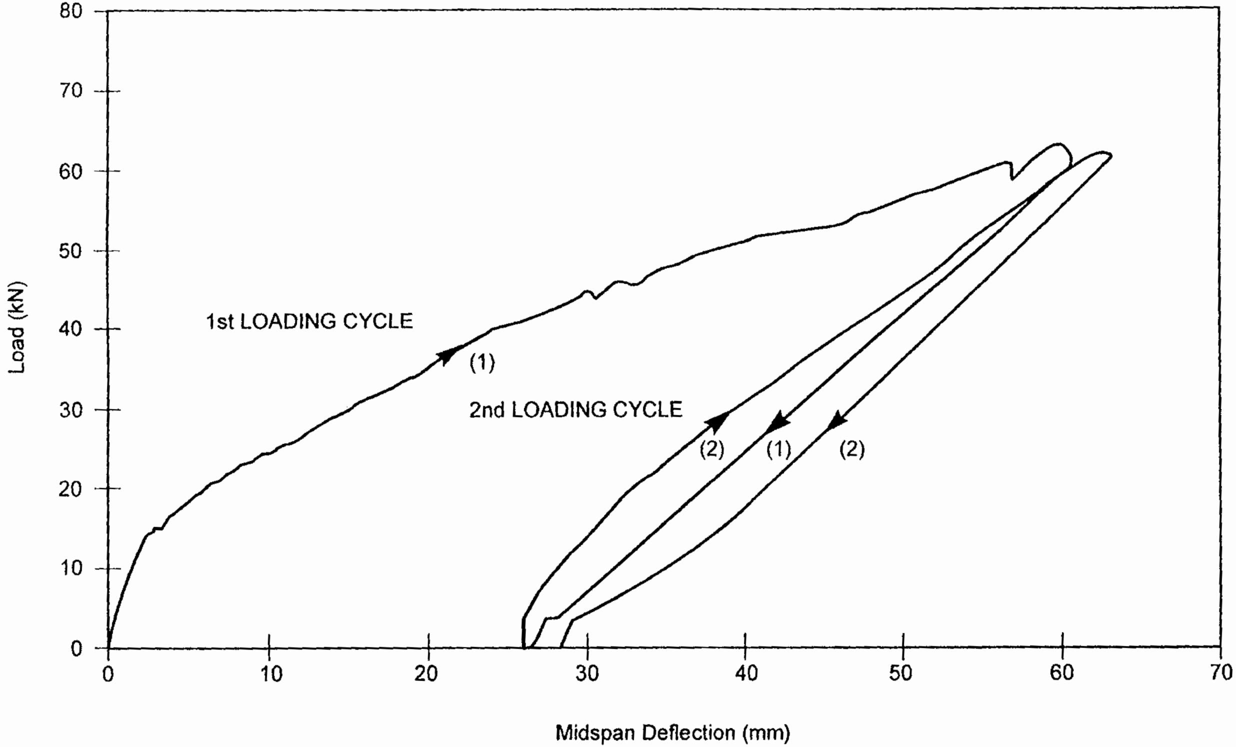


Figure 4.22 Load–deflection curve for beam 4 prior to plating.

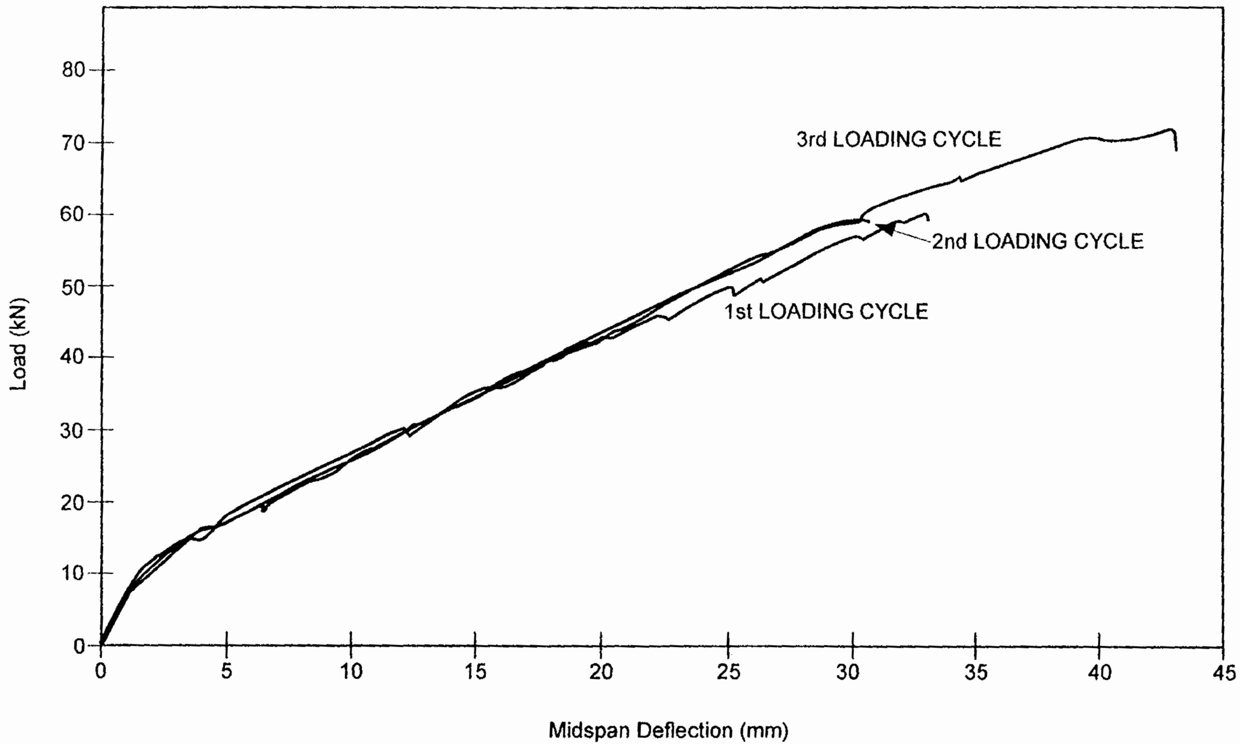


Figure 4.23 Load–deflection curve for beam 4 after plating.

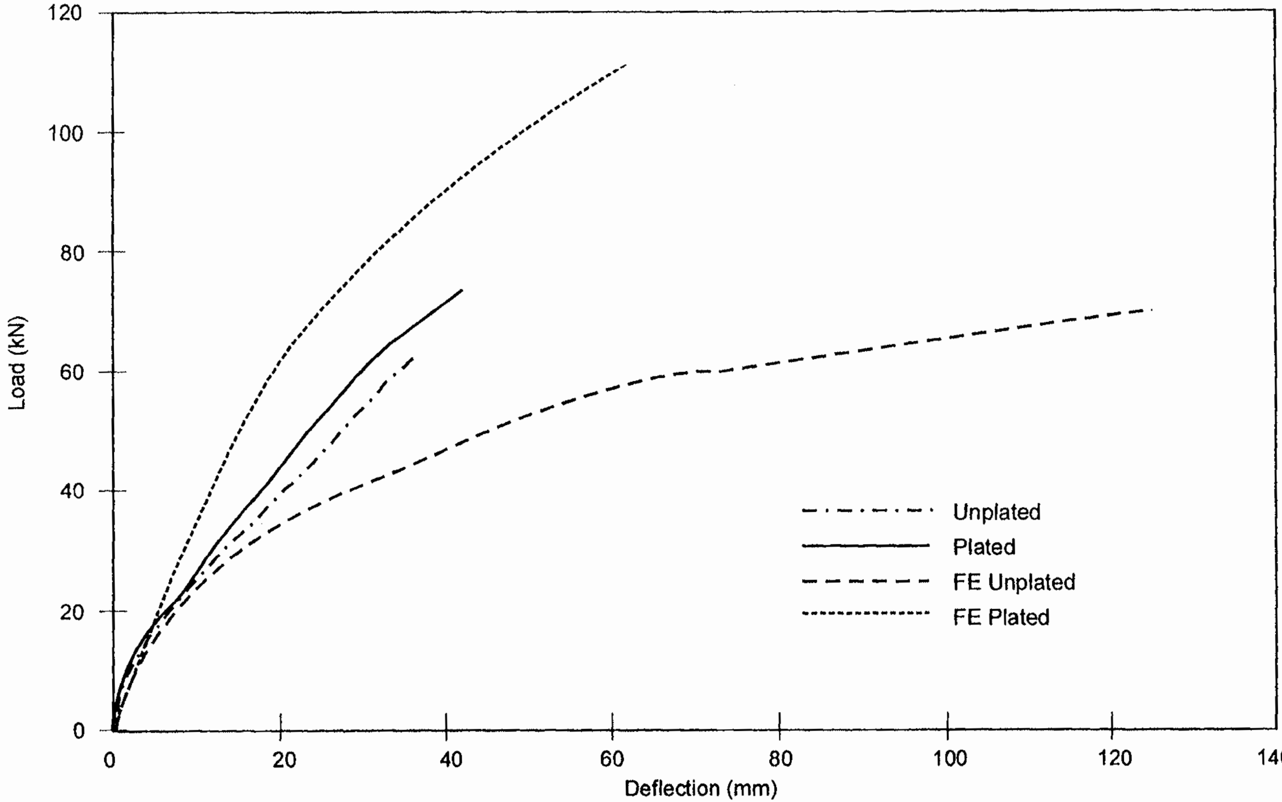


Figure 4.24 Superimposed load–deflection curves for beam 4.

element (FE) analysis. The results of the FE analysis represent the lower and upper bounds of predicted performance, bearing in mind the uncertainties involved in estimating the effectiveness of the attempts to destress three of the prestressing tendons.

A comparison of deflections, strains and crack widths in the unplated and plated beam 4 at various load levels is presented in Table 4.4. Taking the results at 60 kN as an example, it may be observed that although the effect of plating on deflections is relatively small, tensile strains at midspan and crack widths have been approximately halved.

A typical distribution of stress along the plate for the same beam is shown in Fig. 4.25 with the FE predictions. The discontinuities associated with the core locations and the corresponding wide cracks observed at these locations are responsible for many of the variations between observations and theory in these tests.

The load–deflection curves for the remaining beams with unstressed plates are shown in Fig. 4.26 to Fig. 4.32. With the exception of beam 3, the tendons of which were damaged during the plating process, all beams show an increase in stiffness over their unplated counterparts. Beam 3, strengthened by a single layer of 6.0 m long unanchored plates, exhibited a near failure by debonding under one of the core locations (see Fig. 4.33) and within only 0.75 m of the plate end at a load of 60 kN. However, there was no evidence of any anchorage failure of the plate ends.

Table 4.4 Deflections, strains and crack widths for beam 4

	Load (kN)	Unplated	Plated
Midspan deflection (mm)	15	4.4	3.5
	30	14.0	12.5
	60	34.6	33.0
Quarterspan deflection (mm)	15	3.1	2.2
	30	9.3	8.3
	60	22.2	21.0
Midspan plate strain (tension) ($\mu\varepsilon$)	15	35[1]	22
	30	93[1]	60
	60	203[1]	115
Midspan concrete strain (compression) ($\mu\varepsilon$)	15	30	28
	30	80	80
	60	170	180
Crack width (mm)	15	—	—
	30	0.40	0.19
	60	1.02	0.43

[1]On concrete at same location prior to plating.

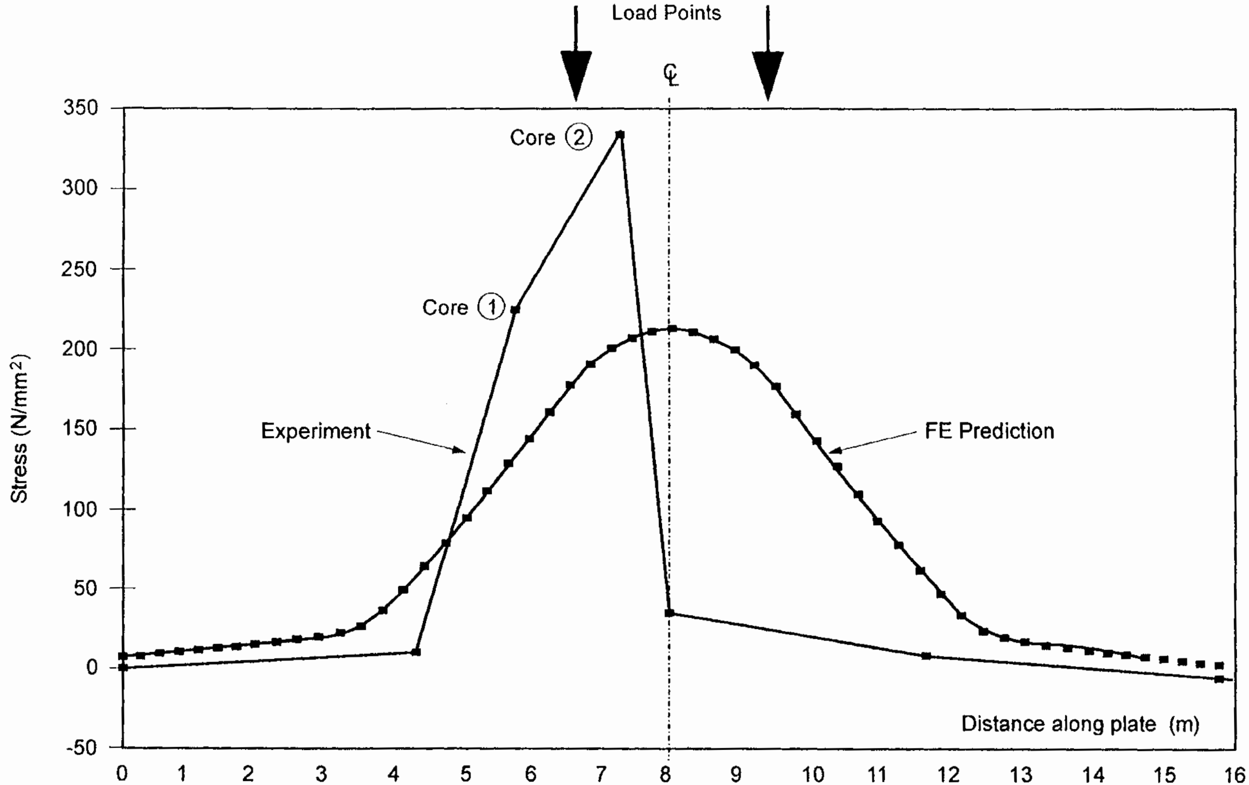


Figure 4.25 Stress distributions along plate for beam 4.

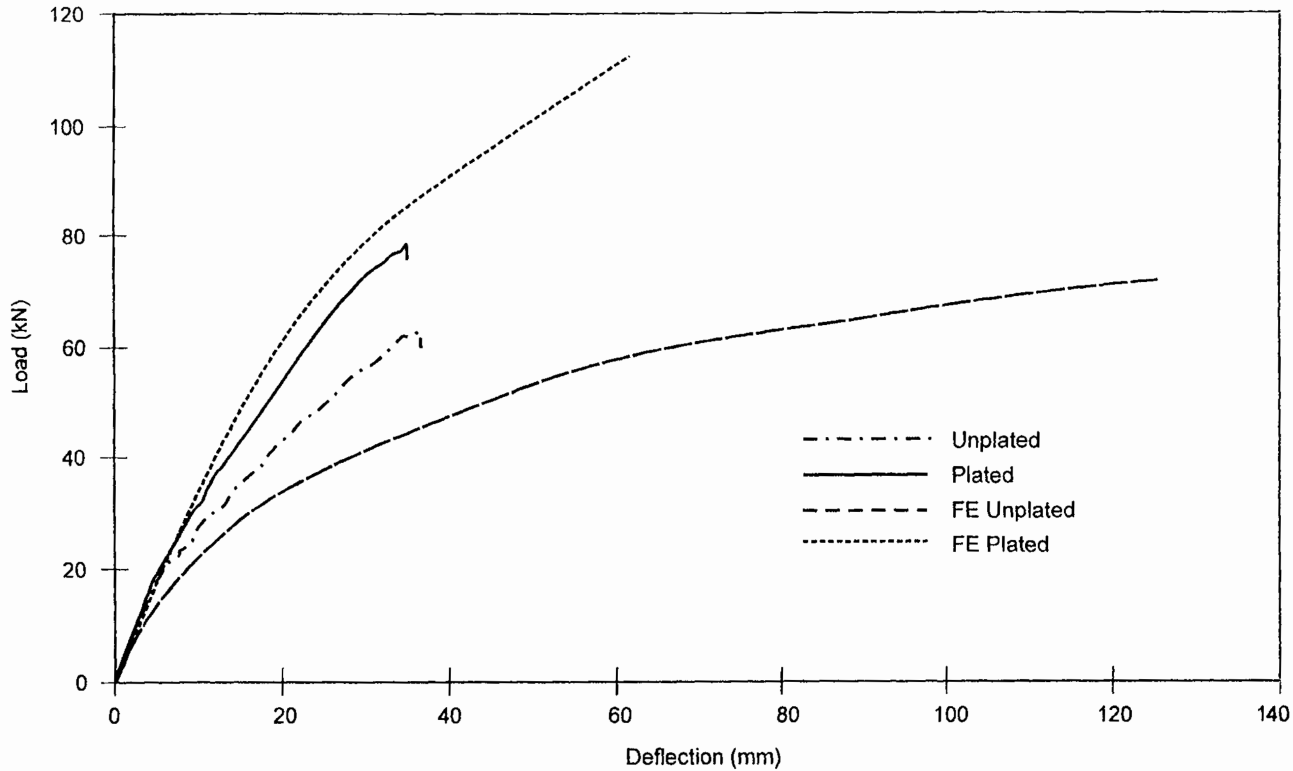


Figure 4.26 Load–deflection curves for beam 1.

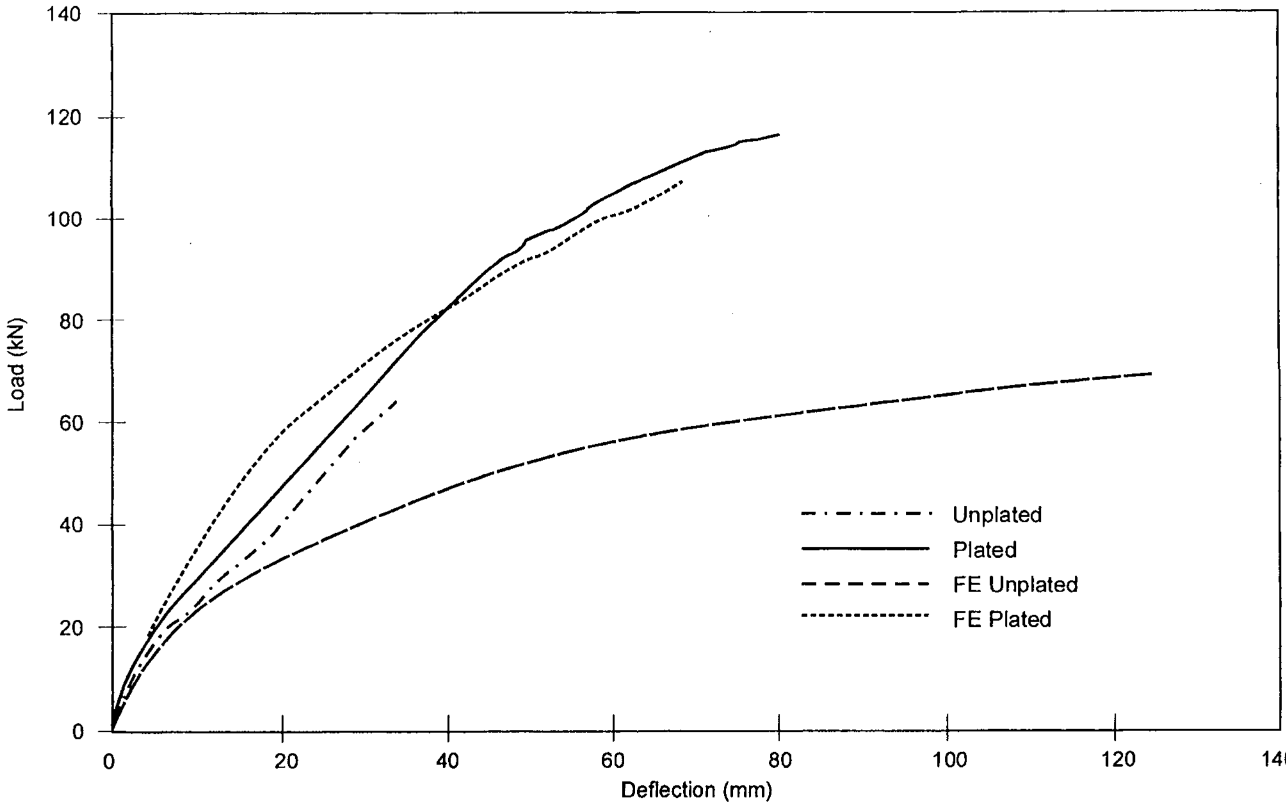


Figure 4.27 Load–deflection curves for beam 2.

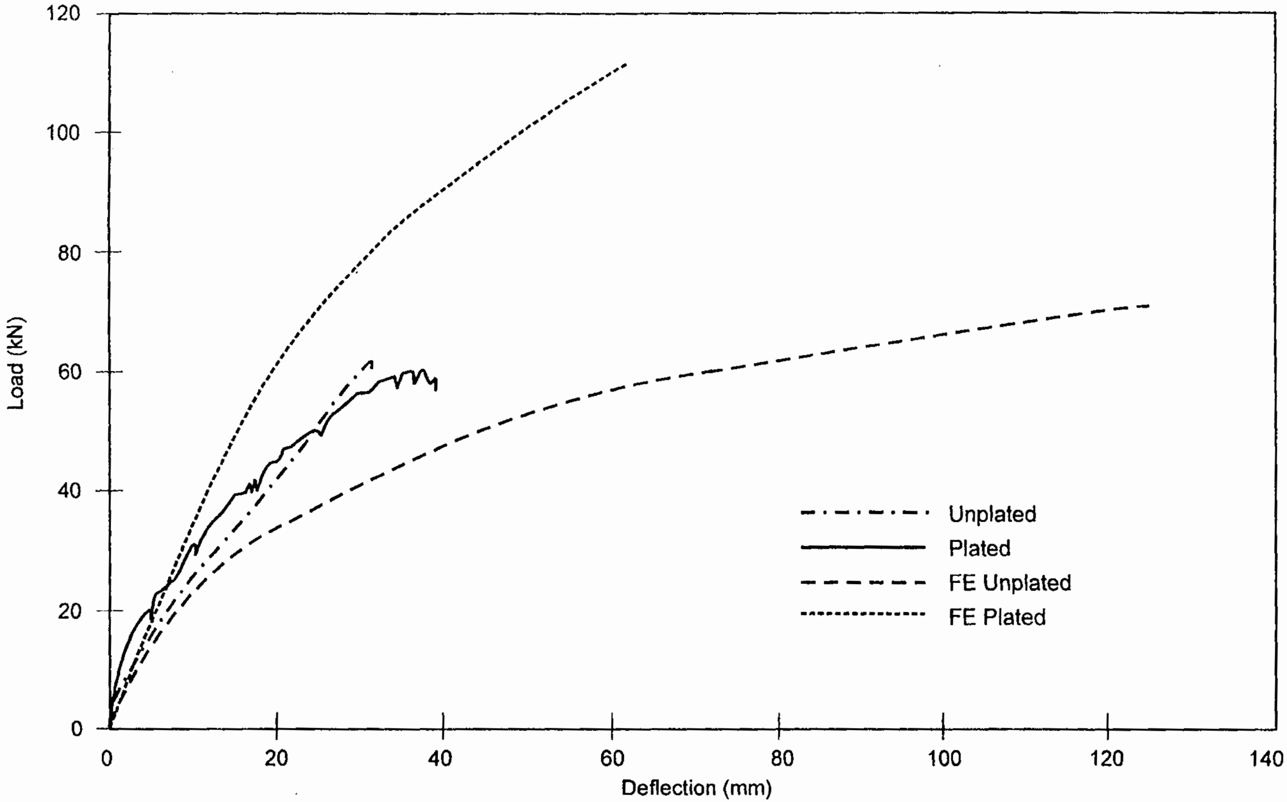


Figure 4.28 Load–deflection curves for beam 3.

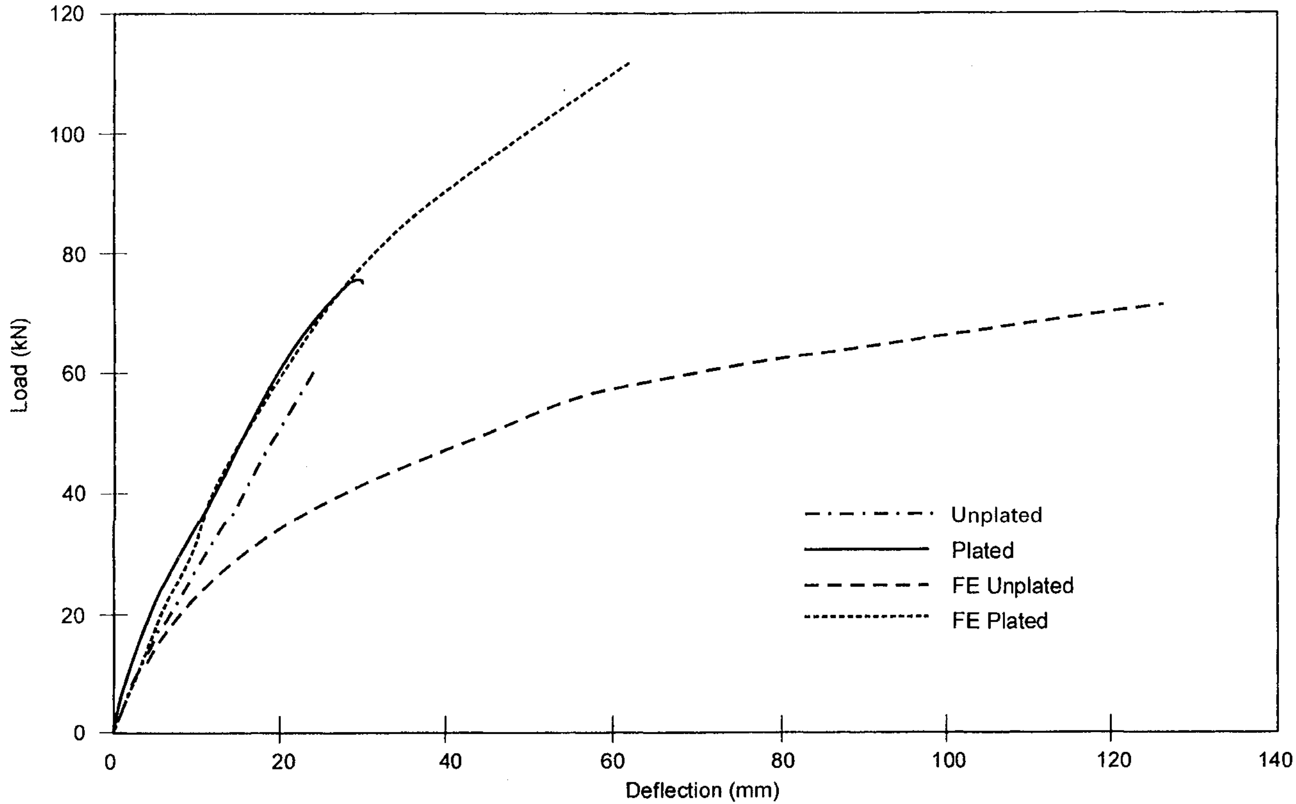


Figure 4.29 Load–deflection curves for beam 5.

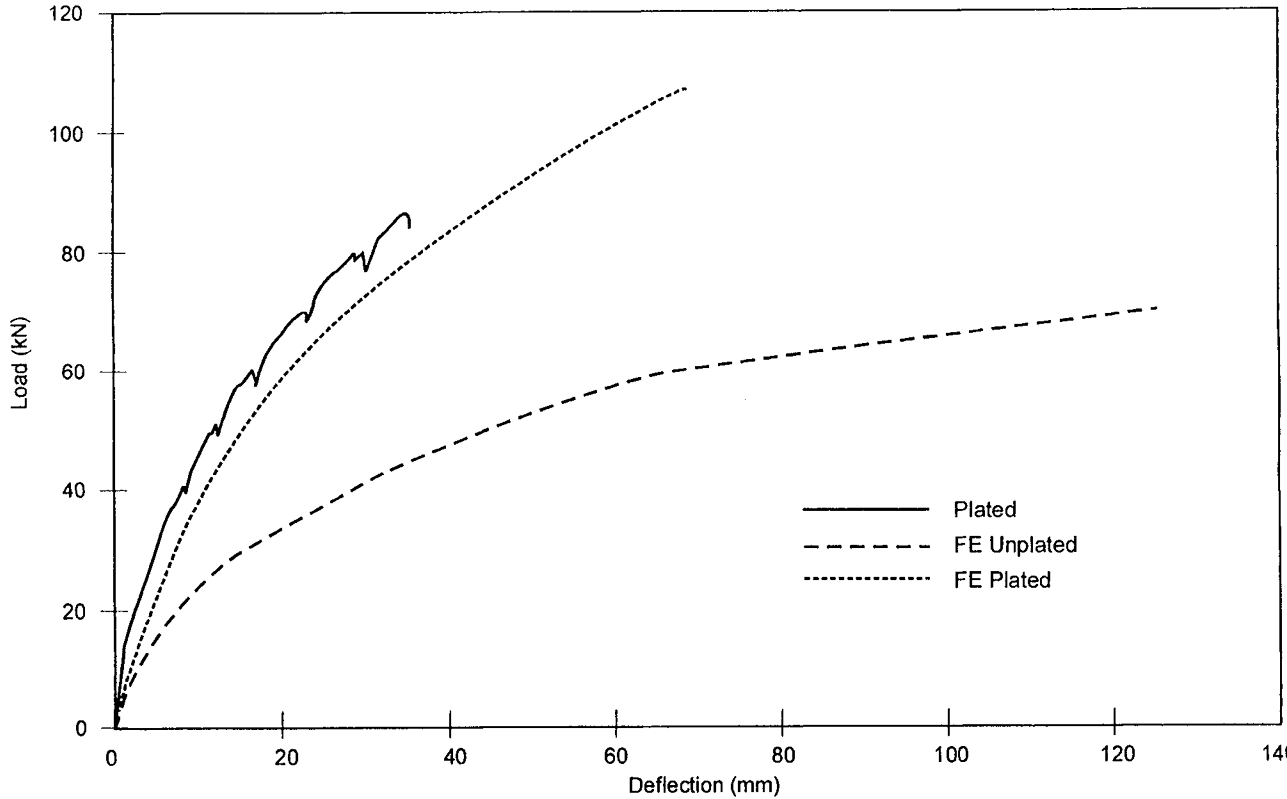


Figure 4.30 Load–deflection curve for beam 6.

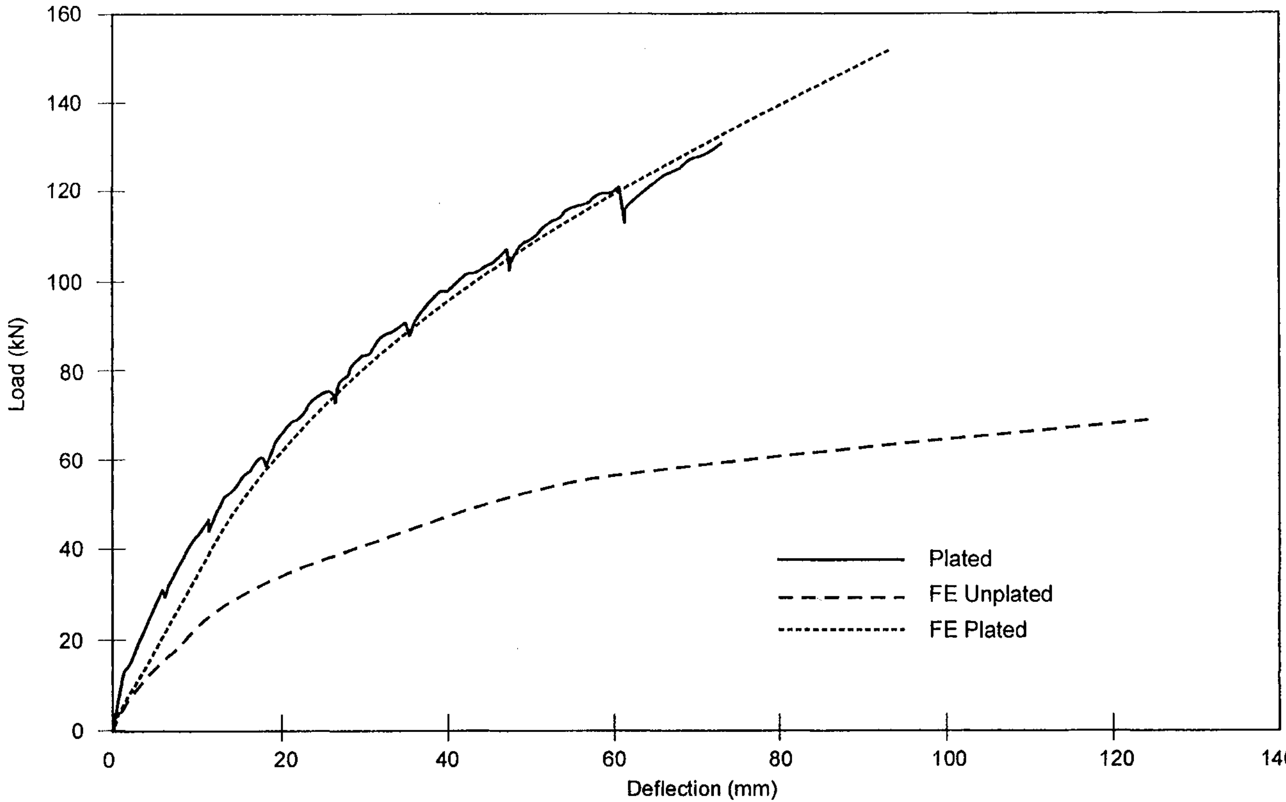


Figure 4.31 Load–deflection curve for beam 9.

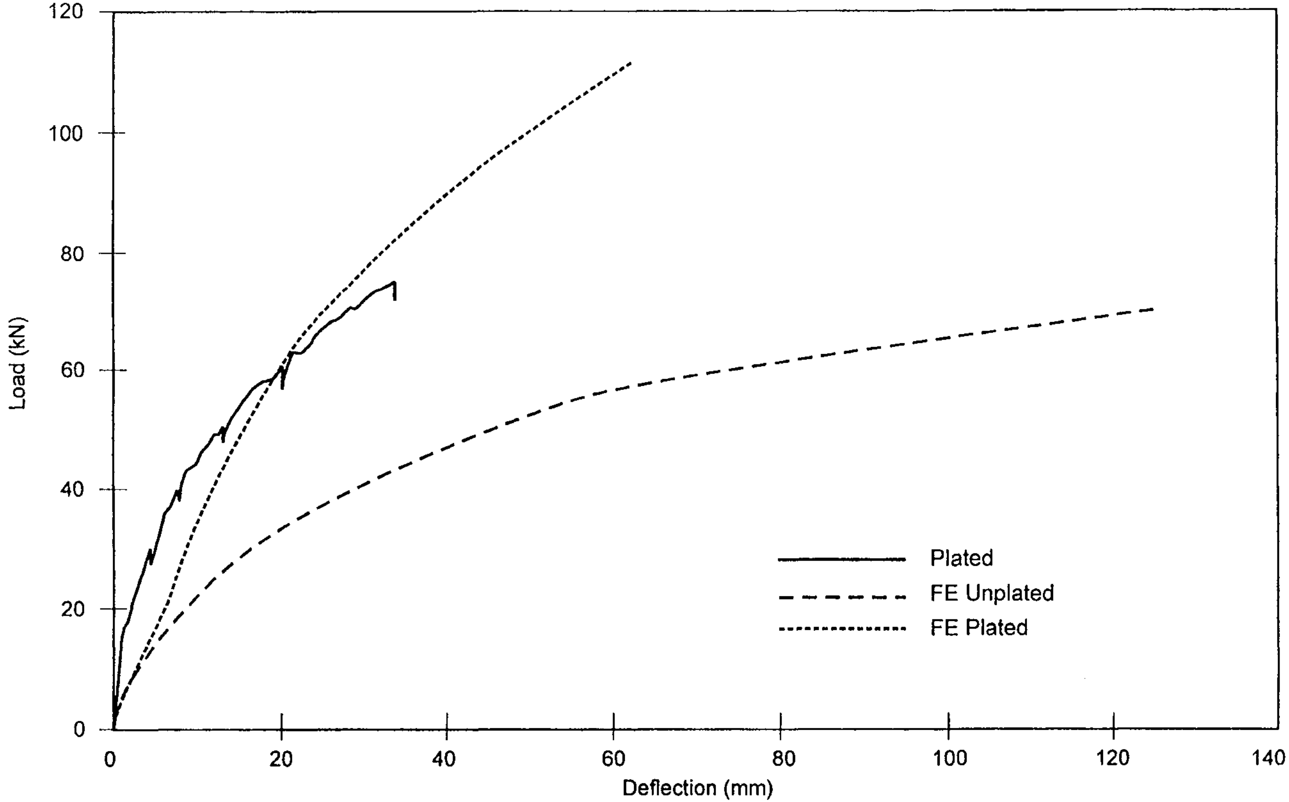


Figure 4.32 Load–deflection curve for beam 10.

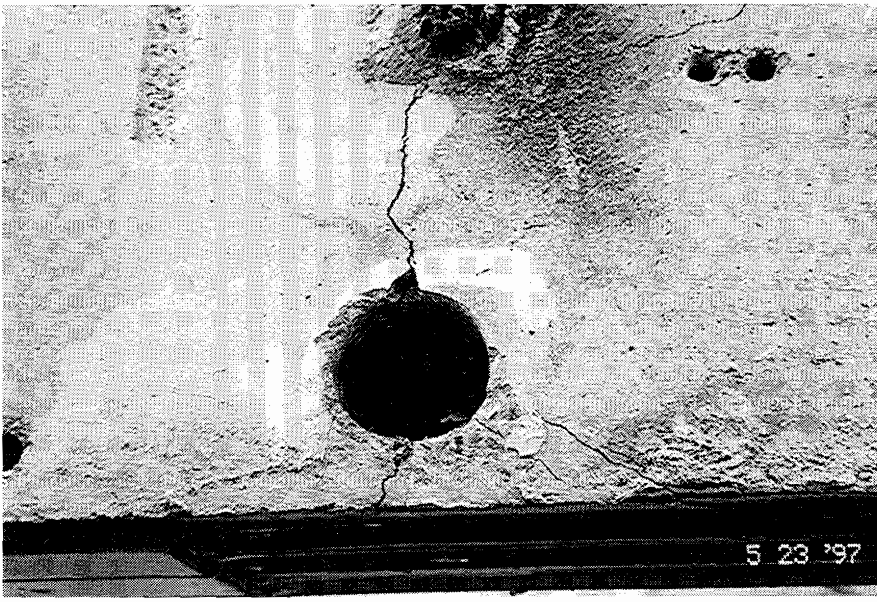


Figure 4.33 Failure mode for beam 3 (photo).

Beam 2, strengthened by a double layer of plates over 15.8 m, was loaded to near failure. This occurred at a load of 118 kN compared with a predicted value of 104 kN. It represented a 60% strength increase over the predicted failure load of an unplated beam. The failure mechanism is illustrated in Fig. 4.34.

Beam 9, strengthened by three layers of 15.8 m plates, was also loaded to near failure. The plates in this beam had been monitored for stress transfer as the three tendons were severed after plating. The beams failed at a load of 130 kN, which is in the middle of the predicted range of 110–150 kN, depending upon the assumptions made about the residual effectiveness of the severed tendons. This failure load represents an increase of strength of 80% over the predicted failure load of an unplated beam. The failure mode is illustrated in Fig. 4.35, again showing debonding following the development of wide inclined cracks originating from the core locations.

Examination of the load–deflection curves for beams 6, 9, and 10 reveals stiffer behaviour than that predicted by the finite element analysis. This is because these beams were plated before coring and the plate has helped in reducing the crack widths at the core positions and hence the deflections.

In those beams which were taken to near failure, the near failure mechanism was relatively ductile and did not involve a catastrophic collapse. In all cases, plate debonding was adjacent to the core which was a particular

Figure 4.34 Failure mode for beam 2 (photo).

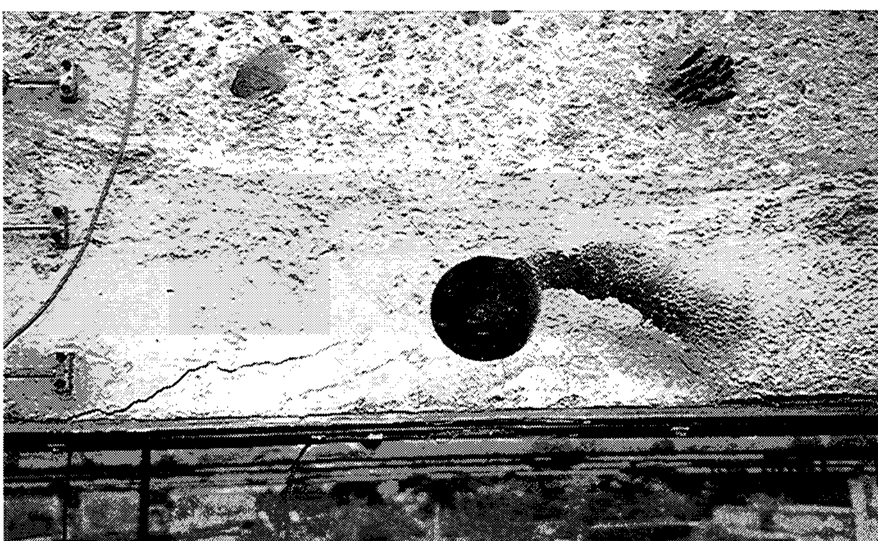


Figure 4.35 Failure mode for beam 9 (photo).

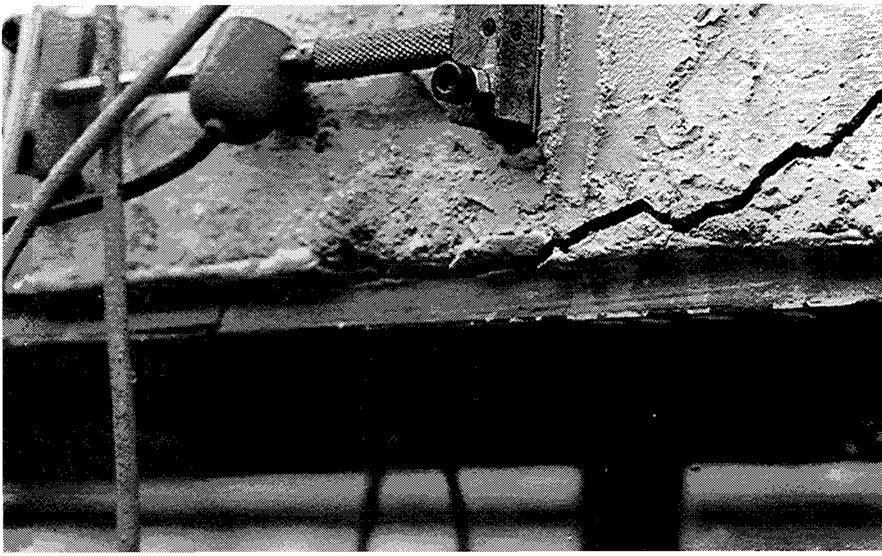


Figure 4.35 Continued.

feature of this test programme. Such discontinuities would not normally be encountered on real structures which, inherently, would be under-reinforced at the outset of strengthening.

4.6 Observations

The important observations derived from the 18 m beam tests under real site conditions are:

- The load–deflection behaviour of beams plated with CFRP pultruded plates is repeatable and predictable.
- CFRP plating provides a significant improvement in crack control, as a consequence of which the beam is stiffer in flexure.
- Failure modes were characterised by debonding under one of the core holes used initially partially to destress the beams.
- No anchorage failures occurred at the ends of the CFRP plates, irrespective of plate length or the presence, or otherwise, of anchor bolts.
- The use of two or three layers of plates produced increases in ultimate capacities of about 60% and 80%, respectively, compared with that predicted for an unplated beam.
- Bonded unstressed CFRP plates can serve to maintain the structural integrity of a prestressed beam in the event of internal tendon damage.

- The viability of strengthening concrete beams using externally bonded unstressed CFRP plates has been successfully demonstrated at full scale.
- As tendons were destressed, stresses were transferred into the bonded plate.

4.7 Concluding remarks

The ROBUST project has demonstrated that composite plate bonding generates significant improvements in ultimate capacity of reinforced concrete members and that the stiffness of a member is increased, causing reductions in the maximum overall deflection and strains throughout the cross-section of the beam. It is clear that as the plate strength and stiffness are required along the length of a flexurally loaded member, the most appropriate choice of material is a unidirectional reinforced composite with the fibres parallel to the span of the beam; there must, however, be sufficient stiffness transversely for handling purposes.

4.8 References

3M (1990) 'Scotch-weld structural adhesive 9323-B/A: product data sheet', *3M Adhesives, Coatings and Sealants.*

Deblois M, Picard A and Beaulieu D (1992) 'Renforcement de poutres en beton armé à l'aide de matériaux composites: études théoretique et expérimentale', in *Advanced Composite Materials in Bridges and Structures*, eds K W Neale and P Labossière, Proc. 1st. International Conference, Sherbrooke, 1992, pp 165–175.

Collings T A (1987) 'Experimentally determined strength of mechanically fastened joints', in *Jointing Fibre-Reinforced Plastics*, ed. F L Matthews, London, Elsevier Applied Science, pp 9–64.

Garden H, Hollaway L C, Thorne A M and Parke G A R (1996) 'A parameter study of the strengthening of reinforced concrete beams with bonded composites', in *Bridge Management 3 – Inspection, Maintenance, Assessment and Repair*, eds J E Harding, G A R Parke and M J Ryall, E & FN Spon, London, pp 400–408.

Garden H, Hollaway L C and Thorne A M (1997) 'A preliminary evaluation of carbon fibre reinforced polymer plates for strengthening reinforced concrete members', *Proc. Instn Civil Eng; Struct Buildings* **123** 127–142.

Garden H, Quantrill R J, Hollaway L C, Thorne A M and Parke G A R (1998) 'An experimental study of the anchorage length of carbon fibre composite plates used to strengthen reinforced concrete beams', *Construct Building Mater* **12** 203–219.

Hart-Smith L J (1987) 'Design of adhesively bonded joints', in *Jointing Fibre-reinforced Plastics*, ed. F L Matthews, London, Elsevier Applied Science, pp 227–270.

Hussain M, Sharif A, Basunbul I A, Baluch M H and Al-Sulaimani G J (1995) 'Flexural behaviour of precracked reinforced concrete beams strengthened externally by steel plates', *ACI Struct J* **92**(1) 14–22.

Irwin C A K (1975) '*The Strengthening of Concrete Beams by Bonded Steel Plates*', Supplementary Report 160UC, Transport and Road Research Laboratory, Crowthorne.

Jones R, Swamy R N and Ang T H (1982) 'Under- and over-reinforced concrete beams with glued steel plates', *Int J Cement Compos* **4**(1) 19–32.

Jones R, Swamy R N, Bloxham J and Bouderbalah A (1980) 'Composite behaviour of concrete beams with epoxy bonded external reinforcement', *Int J Cement Compos* **2**(2) 91–107.

Jones R, Swamy R N and Charif A (1988) 'Plate separation and anchorage of reinforced concrete beams strengthened by epoxy-bonded steel plates', *The Struct Eng* **66**(5) 85–94.

Kong F K and Evans R H (1987) *Reinforced and Prestressed Concrete*, 3rd edn, London, Chapman and Hall.

Kotsovos M D (1983) 'Mechanisms of "shear" failure', *Magazine Concrete Res* **35**(122) 99–106.

Kotsovos M D (1984) 'Behaviour of reinforced concrete beams with a shear span to depth ratio between 1.0 and 2.5', *ACI J* **81** 279–298.

Macdonald M D and Calder A J J (1982) 'Bonded steel plating for strengthening concrete structures', *Int J Adhesive Adhesion* **2** 191–207.

Matthews F L (1987) 'Introduction', in *Jointing Fibre-reinforced Plastics*, ed. F L Matthews, London, Elsevier Applied Science.

Meier U and Kaiser H (1991) 'Strengthening of structures with CFRP laminates', *Proc. International Seminar on Structural Repairs and Strengthening by the Plate Bonding Technique*, University of Sheffield, September 5th–7th, 1990.

Oehlers D J (1992) 'Reinforced concrete beams with plates glued to their soffits', *J Struct Eng* **118**(8) 2023–2038.

Quantrill R J, Hollaway L C, Thorne A M and Parke G A R (1995) 'Preliminary research on the strengthening of reinforced concrete beams using GFRP', in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, ed. L Taerwe, London, E & FN Spon, pp 541–550.

Rahimi H (1996) *Strenghtening of Concrete Structures with Externally Bonded Fibre Reinforced Plastics*, PhD Thesis, Oxford Brookes University.

Ritchie P A, Thomas D A, Lu L W and Connelly G M (1991) 'External reinforcement of concrete beams using fiber reinforced plastics', *ACI Struct J* **88**(4) 490–500.

Sharif A, Al-Sulaimani G J, Basunbul I A, Baluch M H and Ghaleb B N (1994) 'Strengthening of initially loaded reinforced concrete beams using FRP plates', *ACI Struct J* **91**(2) 160–168.

Swamy R N and Jones R (1990) 'Strengthening of concrete structures by the plate bonding technique: structural behaviour and design aspects', *Proc. International Seminar on Structural Repairs and Strengthening by the Plate Bonding Technique*, University of Sheffield, September 5–7, 1990.

Swamy R, Jones R and Bloxham J W (1987) 'Structural behaviour of reinforced concrete beams, strengthened by epoxy-bonded steel plates', *The Struct Eng* **65A**(2) 59–68.

Takeda K, Mitsui Y, Murakami K, Sakai H and Nakamura, M (1996) 'Flexural behaviour of reinforced concrete beams strengthened with carbon fibre sheets', *Composites A* **27**(10) 981–987.
Van Gemert D and Maesschalck R (1983) 'Structural repair of a reinforced concrete plate by epoxy bonded reinforcement', *Int J Cement Lightweight Concrete* **5**(4) 247–255.

5

Structural strengthening of concrete beams using prestressed plates

H N GARDEN AND G C MAYS

5.1 Introduction

The aim in prestressing concrete beams may be either to increase the serviceability capacity of the structural system of which the beams form a part or to extend its ultimate limit state. External prestressing can be applied effectively by the use of unbonded tendons or bonded composite plates. In both cases, it is essential that the existing structure can accommodate the mounting of additional elements for the ends of the prestressing components, but the use of tendons usually necessitates the installation of additional deviation supports for tendons to form the longitudinal profile of the tendons. In both cases, it will be essential to investigate fully the magnitude and form of any local stresses which may be induced into the existing structure as a result of the application of additional prestress.

The method of prestressing with externally bonded composite plates combines the benefits of excellent plate durability and structural improvement, due to the plate tension at the greatest possible distance from the neutral axis of the beam. It has been suggested that prestressing with composite plates is a more economical alternative to conventional prestressing methods used in new construction (Triantafillou and Plevris, 1991).

A significant advantage is gained by prestressing in segmental construction (Trinh, 1990). This is due to the compressive strains generated by prestress at the joints between segmental units, these locations experiencing high tensile strain in the absence of prestress. This high tensile strain is associated with high compressive strain at the top of a segmentally constructed member, resulting in failure by concrete crushing. Depending on the prestress magnitude, the likelihood of crushing at joints is either reduced or eliminated. Leeming *et al.* (1996) reported the ability of externally bonded composite plates to restrict the opening of segmental joints.

The long term shortcomings of conventional prestressing using tendons is that the tendon prestress does not remain at its initial value throughout the life of the structure because various factors bring about prestress losses. The principal losses are due to the following causes which contribute the proportions shown in brackets to the total prestress loss in post-tensioning construction:

- relaxation of the tendons (5%),
- immediate elastic deformation of the concrete that occurs when prestress is transferred into the beam (2–3%). Tendons that have already been prestressed will experience a loss of prestress due to the shortening of the beam upon the prestressing of subsequent tendons,
- creep and shrinkage of the concrete under the compressive prestress over the service life of the structure (10–20%),
- slippage of the tendons at their end anchorages that occurs when the prestress is transferred into the anchorages,
- friction between the tendon and its duct in post-tensioned members, but this being less of a problem in simply supported beams with fairly flat tendon profiles (1–2%) but more severe in continuous beams with their more profiled tendons (10% or more).

Prestressing with externally bonded plates is also subjected to prestress losses by immediate elastic deformation and long term creep of the concrete, although friction losses do not apply. As with tendon losses due to settlement and deformation at the end anchorages, bonded plates suffer a prestress loss due to the shear transferred through the adhesive and into the concrete by the plate tension. This shear action is sufficient to fracture the concrete even at low prestress levels so it is necessary to install anchorages at the ends of the plate to resist this action.

One of the benefits of applying prestressed composite plates is that the effect of the original internal tendon prestress can be restored. If the prestress loss is severe, the nominal tensile reinforcement will be unable to contain concrete crack widths to within the serviceability limit and the permissible applied load level will be reduced.

The application of externally bonded prestressed plates involves four main stages:

- the prestressing of the plate to that initial force which provides the required long term prestress after losses have been taken into account,
- the bonding of the plate to the concrete member,
- the installation of plate end anchorages that will resist the forces tending to fracture the concrete when the prestress is subsequently transferred into the member and when the member is subjected to an externally applied load,
- the transfer of the plate prestress into the concrete.

The prestressing force is not released until after the plate end anchorages have been installed, in order to ensure that the cover concrete is not damaged due to the high shear and peel stresses caused by the plate tension. Investigative work for the ROBUST project, undertaken at the University of Surrey, included interface tests on a concrete/CFRP (carbon fibre reinforced plastic) composite shear specimen in which the applied tensile force was in the plane of the plate. Garden (1997) found that the maximum plate tension that could be transferred into the concrete, before surface delamination of the concrete occurred, was only 6% of the ultimate tensile stress of the plate material. In practice, much higher prestress levels, in the order of at least 25%, will be necessary to achieve a significant improvement in structural stiffness and load carrying capacity of the concrete beam; it has even been suggested that a prestress of as much as 50% of the plate strength may be necessary (Meier *et al.*, 1992)

5.2 Review of previous prestressing studies using composite plates

Triantafillou and Plevris (1991) reported an analytical model developed to describe the maximum achievable level of pretension which can be applied to a composite plate so that the external strengthening system does not fail near the anchorage zones as the pretension is released. It was assumed that failure can occur either by horizontal cracking of the concrete above the fibre reinforced polymer (FRP) plate at the two end zones due to high shear stresses, or by yielding of the adhesive. The analytical model developed is used in a parametric study which suggests that the efficiency of the method is improved by increasing the thickness of the adhesive layer, using a thinner but wider plate of composite material, or by increasing the length over which the plate is bonded, all of these effects theoretically increasing the level of stress which can be applied to the plate, prior to bonding, before failure occurs on release.

Section 2.4.3 of Chapter 2 has given a comprehensive review of the previous prestressing studies using composite plates and it is clear from this review of the work reported in the literature that, whilst this extension of the FRP plate bonding technique is potentially beneficial, given the advantages inherent in prestressing in practice, it remains very much in its infancy. Most of the work carried out to date has merely demonstrated that failure of the system will occur on release of the prestress unless adequate anchorage systems are provided at the plate ends. It has been demonstrated through the programme of beam testing, described in Chapter 4, that for the configurations tested and with shear span/beam depth ratios less than 4.0, failure of a non-prestressed system will occur by separation of the plate from the beam when tested in flexure unless some form of anchorage is provided. It therefore follows that anchorage is an even greater necessity when the plate is given an initial prestress. The provision of anchorages at the plate ends reduces the shear deformation which occurs within the adhesive layer upon pretension release, thereby reducing the shear stresses transferred to the base of the concrete section and minimising the possibility of premature failure. In the ROBUST investigations, therefore, plate end anchorages were used in all cases so that behaviour on application of external load could be studied, as well as the initial response to pretension release.

5.3 Prestressing technique employed in the laboratory

The technique used to pretension the CFRP plates in the laboratory for both the 1.0 m and 2.3 m long beams was as follows; the technique was different from that used on site (see Section 5.5).

Aluminium tabs were epoxy bonded at each end and on both faces of the CFRP plate. These tabs were used to provide stress distribution in the end regions and were 2 mm thick, 130 mm long for both beam sizes and full plate width (80 mm for the 1 m long beams, 90 mm for the 2.3 m). Each end of the CFRP plate was then sandwiched between two steel plates 9 mm thick which were predrilled and provided a jig into which the tabbed ends of the CFRP plate could be held with pins during drilling. After drilling, 12 6 mm diameter bolts were located at each end and tightened to provide frictional as well as shear resistance to the prestressing force to be applied; the CFRP plate with the steel plates bolted on at each end was then loaded into a prestressing frame. The end reaction system is shown in Fig. 5.1.

The sequence of procedures followed in preparing the prestressed beams is shown in Fig. 5.2; methods of gritblasting and cleaning the concrete bond surface, the utilisation of the peel-ply protection on the surface of the composite and the sprinkling of 2 mm diameter ballotini to provide the correct thickness of adhesive were similar to those used in Chapter 4 for the non-tensioned plates. After stressing the plate to the required level, the concrete beam was lifted with jacks up to the level of the taut plate, Fig. 5.2(a). A timber plank was then used to hold the beam at the correct height whilst the adhesive cured. Dead weights equivalent to a pressure of around 7.5 kN m^{-2} were applied to the upper face of the plate so that excess adhesive would be extruded, Fig. 5.2(b); in practice, a vacuum bag technique may be necessary to support the external plate during bonding. After the adhesive had cured, steel clamps were installed at each end of the beam to ensure adequate anchorage of the plate ends upon release of the pretension. The load applied to the CFRP plate was then reduced to zero, the pretension being transferred to the beam by the cured adhesive bond layer and the plate was cut through to isolate the beam.

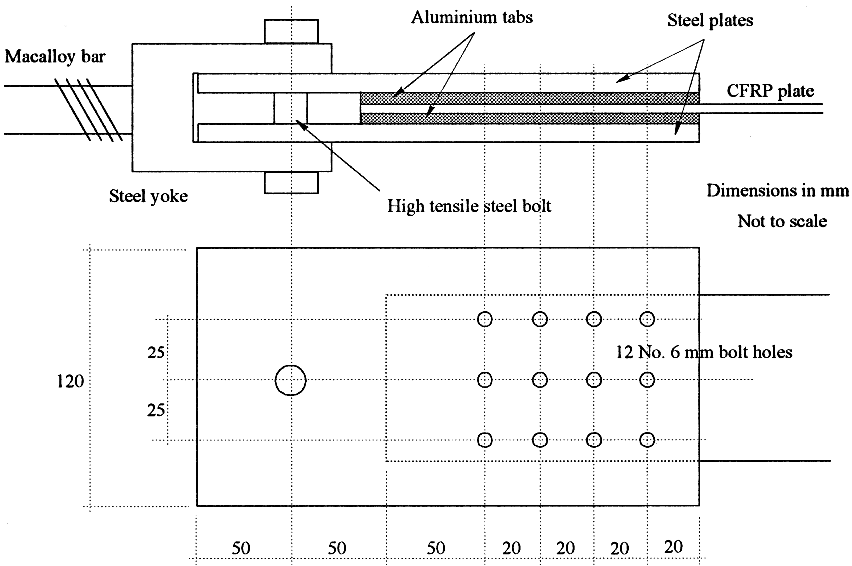


Figure 5.1 End reaction system for prestressing plates.

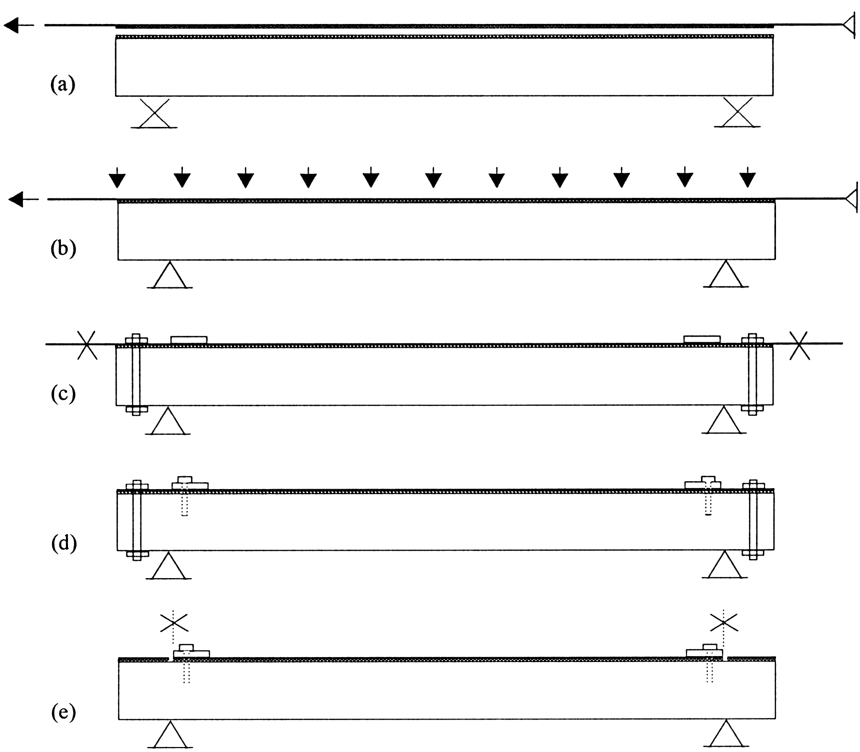


Figure 5.2 Sequence of procedures in preparing prestressed beams.

GFRP endplates 14 mm thick and 40 mm long, covering the full width of the CFRP plate were used in the 2.3 m and 4.5 m beams. These were predrilled and bonded in the correct positions, illustrated in Fig. 5.2(c). After this adhesive had cured and using the glass fibre reinforced plastic (GFRP) endplates as a jig, holes were drilled through the CFRP plate and adhesive into the concrete beam. Steel bolts were then bonded into the holes and allowed to cure, as shown in Fig. 5.2(d); it was necessary for the length of bolt to extend well into the tensile zone of the reinforced concrete beam.

5.4 Results of laboratory tests for concrete beams strengthened with prestressed plates in the ROBUST programme

To illustrate the load/deflection and load/strain responses of reinforced concrete beams strengthened with pretensioned composite plates, typical results from the 2.3 m long beams tested under the ROBUST programme will be discussed. The four point load configuration adopted for these beams provided a 400 mm long constant moment region in the centre of the beam and each shear span was 450 mm long.

The effects of strengthening the 2.3 m beams with various plate tensions are summarised in Tables 5.1 and 5.2, whilst the variations in overall member stiffness are given in Table 5.3.

For all the 2.3 m beams tested during the project, prestressing the composite plates prior to bonding increased the serviceability load but, as the load was governed by the concrete strength and not by the internal steel, the increase over the unplated case was small and pretensioning the plate

Table 5.1 2.3 m Prestressing investigation; strengthening effects of plating; A_s/bd[1] ratio was 1.18% and strength of concrete was nominally 50 N mm^{-2} (prestress as a percentage of the ultimate strength of plate)

Beam	First cracking load (kN)	Increase over unplated (%)	Service-ability load (kN)	Increase over unplated (%)	Yield load (kN)	Increase over unplated (%)
Unplated	13.0	—	47.2	—	76.5	—
Plated 0%	15.0	15.4	53.3	13.0	100.0	30.7
Plated 34.7%	22.5	73.1	53.3	13.0	115.0	50.3
Plated 41.7%	34.0	162.0	53.3	13.0	124.0	62.1

[1] A_s is the area of tensile steel rebars, b is the breadth of the beam and d is the depth of the beam, respectively.

Table 5.2 2.3 m Prestressing investigation; ultimate loads and ductilities; A_s/bd ratio was 1.18% and strength of concrete was nominally 50 N mm^{-2}

Beam	Maximum load carried (kN)	Increase over unplated (%)	Ductility	Proportion of unplated ductility (%)
Unplated	79.9	—	5.15	—
Plated %	125.6	57.2	4.38	85.1
Plated 34.7%	129.3	61.8	4.10	79.6
Plated 41.7%	147.8	85.0	5.49	106.6

Table 5.3 2.3 m Prestressing investigation; stiffening effects of plating. A_s/bd ratio was 1.18% and strength of concrete was nominally 50 N mm^{-2}

Beam	Postcracking stiffness (kN mm^{-1})	Increase over unplated (%)	Postyielding stiffness (kN mm^{-1})	Postcracking stiffness retained (%)
Unplated	7.0	—	—	—
Plated 0%	8.60	22.8	2.34	27.2
Plated 34.7%	9.23	31.9	2.34	27.0
Plated 41.7%	9.30	32.9	2.32	26.5

prior to bonding produced little benefit with respect to the serviceability load over the non-prestressed beam for the given concrete strength.

Figure 5.3 shows a typical relationship between the applied load and maximum beam deflection under further tests at the University of Surrey for the ROBUST programme. In this case the plate was initially pretensioned to 25% of its ultimate strength, as indicated. The curves referred to as ‘unanchored’ and ‘bolted’ in this figure describe the absence of plate end anchorage and the use of plate end anchorage bolts, respectively.

In all cases investigated, a considerable increase in the applied load to cause cracking in pretensioned plated beams was apparent and, from the case considered in Fig. 5.3, a 100% increase over the non-prestressed beam was achieved. The initiation and development of both flexural and shear cracking were much less in evidence for the pretensioned cases than for the non-tensioned beams at comparable loads, demonstrating the ability of the pretensioned plate to limit cracking.

For the beams tested during the ROBUST investigations, it was observed that the serviceability load was governed by the strength of the concrete (σ_u) and by the (A_s/bd) ratio. It should be noted that the results presented in Tables 5.1, 5.2, and 5.3 corresponded to values of (σ_u) equal to 50 N mm^{-2}

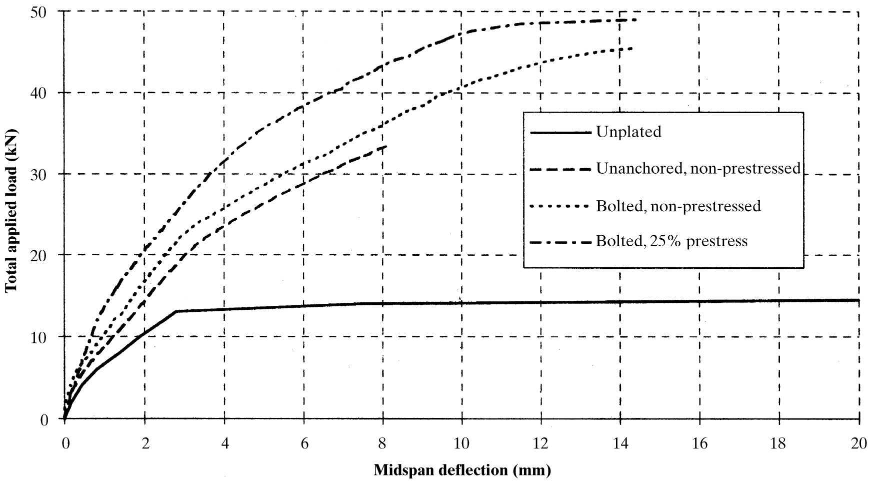


Figure 5.3 Deflection responses to applied load of beams with and without plate prestress (after Garden, 1997).

and values of (A_s/bd) equal to 1.18% whereas the equivalent values for the results shown in Fig. 5.3 were 50 N mm^{-2} and 0.76%, respectively. Consequently, the increases in serviceability over the unplated cases were small in the former case and relatively large in the latter.

From Fig. 5.3, the yield load was found to be 14 kN for the unplated beam, 21 kN for the plated unanchored non-prestressed case and 24 kN for the plated bolted non-prestressed cases, giving increases of 33% and 71.4%, respectively. Prestressing the plate was again found to increase the yield load; an effective prestress of 25% produced a value of 31 kN, an increase of 121.4%.

The CFRP prestressing was also found to produce a moderate increase in the maximum load carried by the plated beams compared with the non-prestressed ones. The nominally 50% prestressed plated case failed by plate tensile fracture, whilst the nominally 25% prestressed plated beam failed by plate separation due to a vertical shear crack opening of the form described in Chapter 4, Section 4.3. Therefore, the lower the ductility the more brittle the failure, as the collapse mechanism changes from plate separation to plate failure in flexural tension.

A significant benefit of prestressing the plate is that the composite action between the plate and the concrete, at the ends of the plate, improves as the prestress increases. This result is illustrated in Figs. 5.4 and 5.5, in which a comparison is made of the increments in plate tensile strain at various points along the plate, under increasing applied load. Both figures represent beams with bolted plate end anchorage with a shear span/beam depth ratio of less than 3.4. However, it should be stressed that all pretensioned plates

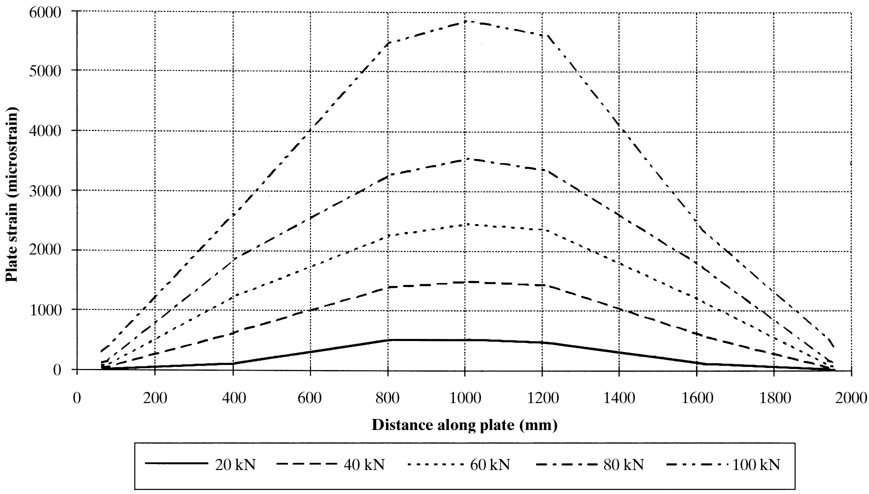


Figure 5.4 Plate strain distributions with no initial plate prestress (after Garden, 1997).

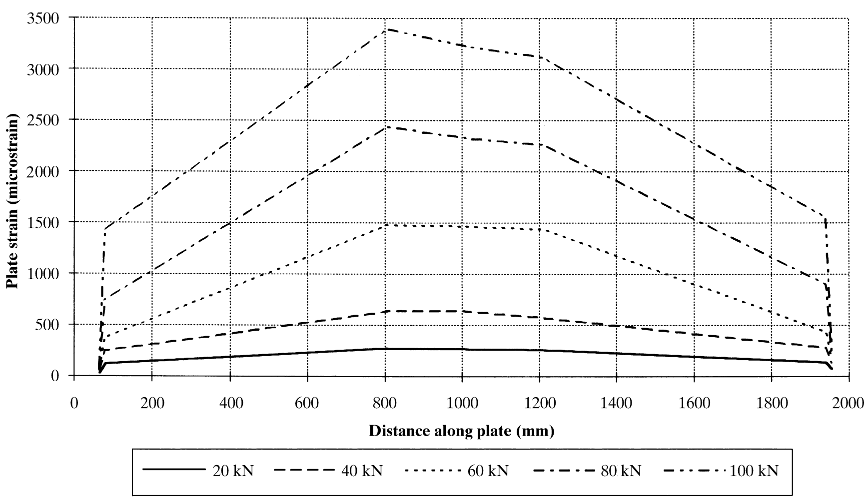


Figure 5.5 Plate strain distributions with a 40% initial plate prestress (after Garden, 1997).

require anchoring at their ends although non-tensioned plates above a shear span/beam depth ratio of 4.0 may not. The increased plate strain at the plate ends indicates an improvement in the ability of the adhesive layer in the vicinity of the plate ends to transfer shear, although the precise mechanism by which this improvement comes about remains the subject of further research.

Figures 5.6 and 5.7 show, for two 4.5m long beams, the strain values at various heights in the plated beams for the non-tensioned and tensioned plates, respectively; the legends in the figures indicate the applied loads. The thick line, marked 0kN in Fig. 5.7, represents the strains in the section after prestress transfer, but before external loading of the beam, for the case in which the plate was prestressed nominally to 50% of its ultimate strength before bonding. Furthermore, when the applied load was 8kN, almost the

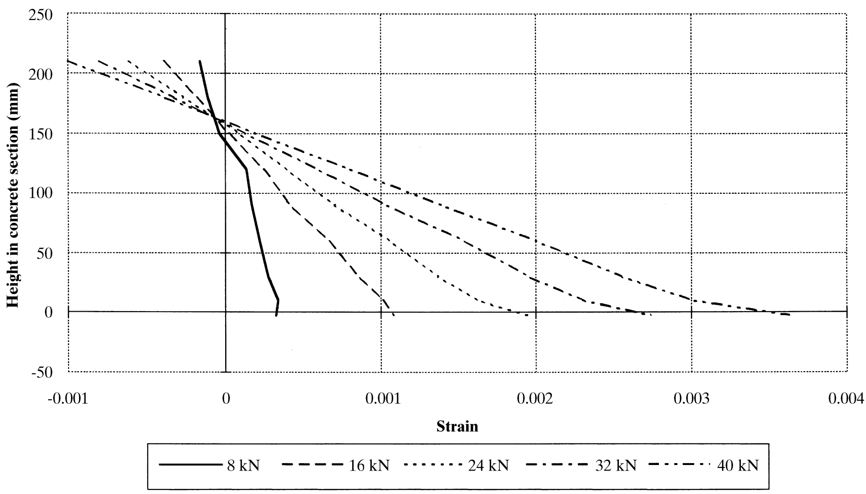


Figure 5.6 Section strains without plate prestress (after Garden, 1997).

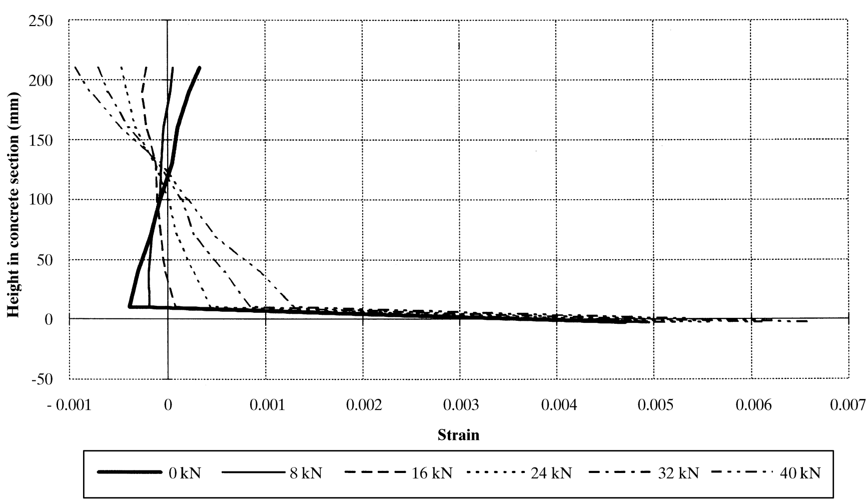


Figure 5.7 Section strains with 50% nominal plate prestress (after Garden, 1997).

full depth of the beam was under a small compressive stress and on subsequently applying external loads greater than 8kN, the neutral axis level remained lower in the prestressed case than the non-prestressed one. The high tensile strains, in the prestressed case, at the level of the composite plate (Fig. 5.7) reflect the prestress tension applied to the CFRP composite plate before the beam was externally loaded; the values at this level are the total strains in the composite plate.

A consistent feature of plated beams with end anchorage under flexural loading is that the curve of applied load against plate strain after yield of the internal rebars is common for non-prestressed and prestressed members (Garden *et al.*, 1998); this is illustrated in Fig. 5.8. It is shown that the non-prestressed member without plate end anchorage exhibited a lower post-yield stiffness than the non-prestressed beam with the bolted plate ends. This is a typical result for beams loaded with a shear span/beam depth ratio equal to or lower than 4.0 due to the stiffening influence of the plate end anchorages (Garden and Hollaway, 1997). This would indicate that the post-yield stiffness of a plated beam with end anchorage is not governed by the plate prestress and that the locus of yield points, with increasing prestress, defines a straight line. These features have been discussed by Garden (1997) in terms of the material properties and curvatures of plated sections under the applied load.

During the investigations with non-prestressed plates, it was noticed that sometimes the adhesive layer suffered cracking through its thickness and along the interface with the composite plate, due to the propagation of shear cracks through the depth of the concrete (Garden, 1997; Garden and

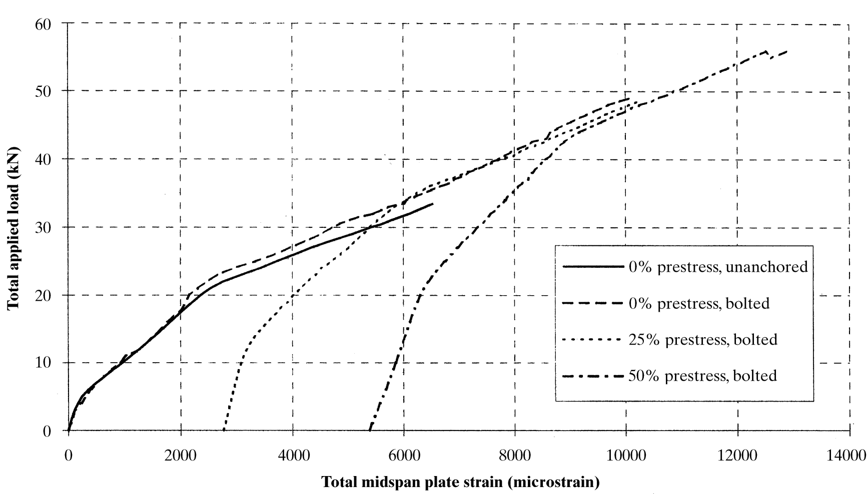


Figure 5.8 Total plate strains of beams with and without plate prestress (after Garden, 1997).

Hollaway, 1997). When the plate prestress is of the order of 20% of ultimate, this adhesive cracking can be considerably retarded or even eliminated. In practice, this characteristic would improve the durability of the system by reducing the moisture ingress at the level of the adhesive and its interfaces.

From a study of the number and distribution of shear cracks in the shear spans of beams loaded under symmetrical four point bending, Garden (1997) demonstrated that, for a given applied load with increasing plate prestress, the number of shear cracks reduces in relation to distance from the loading position. This observation reflects the improvement in durability that may be expected with a prestressed composite plate, compared with a non-prestressed one.

5.5 Results of field investigations of concrete beams strengthened with prestressed plates in the ROBUST programme

Two 18.0 m long prestressed concrete beams were selected from the same source as those used for the field investigations with non-prestressed plates, presented in Chapter 4. The lower three internal tendons in one of these beams were destressed, as described in Section 4.5, prior to the application of the prestressed plate. A four point bending load test, undertaken by the Royal Military College of Science, using the loading configuration detailed previously in Section 4.5.3, was then undertaken within the elastic range to establish the unplated member stiffness. In the second beam, the three lower tendons were destressed after the beam was plated; the prestressed plates were monitored to determine their increase in strain due to the destressing of the internal tendons.

It was intended that both beams should be strengthened with three prestressed CFRP plates in a single layer, each plate being 90 mm wide by 1.0 mm thick bonded with Sikadur 31 PBA adhesive. The plates were 6.0 m long in the first beam and 15.8 m in the second. Table 5.4 summarises the test parameters.

Table 5.4 Test parameters for 18 m beams (stressed plates)

Beam no.	Overall plate dimensions (mm)	Free/ Anchored plate ends	Plate length (m)	Maximum unplated loading (kN)	Maximum plated loading (kN)	Monitored during coring
7	270 × 1	A	6.0	62.5	100[1]	
8	180 × 1	A	15.8	—	83	Yes

[1] Beam failed under core holes by plate debonding.

In order to stress the plates it was necessary to bond a purpose-designed anchor block to each end of the plate. The purpose of anchorage blocks was explained in Chapter 4. The anchor blocks in the 18.0 m beams were 530 mm by 100 mm by 15 mm GFRP tabs, illustrated in Fig. 5.9. The anchor blocks were prebonded to the CFRP under factory conditions using the adhesive, 9323-2B/A, manufactured by 3M. The plates were then transported to site and prestressed to 30% of their ultimate tensile load using a prestressing device mounted on and reacting against the concrete beam, as shown in Fig. 5.10. For safety reasons, the prestressing device and the bolted anchorage system were designed to resist a load equivalent to twice the ultimate tensile strength of the layer of three plates.

Once stressed, the plates were pressed against fresh adhesive, spread to a predetermined thickness on the beam soffit, using a temporary timber propping arrangement. This was necessary to accommodate the upward camber present in the beams which were already prestressed under the action of the internal tendons; the propping system remained in place until the adhesive had cured.

This prestressing procedure was the first of its kind to be used successfully under site conditions. The work proceeded satisfactorily in all respects, except for the sudden and unexpected failure during stressing of one plate in the second beam (no. 8). Subsequent examination of the failed plate revealed a weakening due to an overlap between successive peel-ply sheets which were entrapped within the body of the plate during manufacture. Important quality control lessons were learned from this work, of particular relevance to prestressed plates which experience higher local stresses than non-prestressed composites.

The curve of load against deflection for the plated beam, no. 7, is shown in Fig. 5.11, compared with the comparable unplated member. This shows that there was a considerable increase in stiffness as a result of applying the prestressed plates, consistent with the smaller scale laboratory findings. This increase in stiffness correlated well with the upper bound stiffness which was predicted using three-dimensional finite element analysis. Figure 5.11 also shows the load–deflection behaviour of beam 5 which was strengthened with a similar configuration of non-prestressed plates (see Section 4.4); the greater stiffness, arising from the plate prestress, is clearly seen.

Beam 7 achieved an ultimate load of 100 kN before failing by plate debonding under one of the core locations (see Fig. 5.12(a) and (b)). It is believed that this premature failure arose as a result of some damage to the two remaining prestressing tendons in the beam, which occurred during the drilling operations to provide fixings for the plate prestressing device.

In beam 8, coring to destress the internal tendons was undertaken after the completion of the plate prestressing and plate strains were monitored as the tendons were destressed. The result from the strain gauge immediately

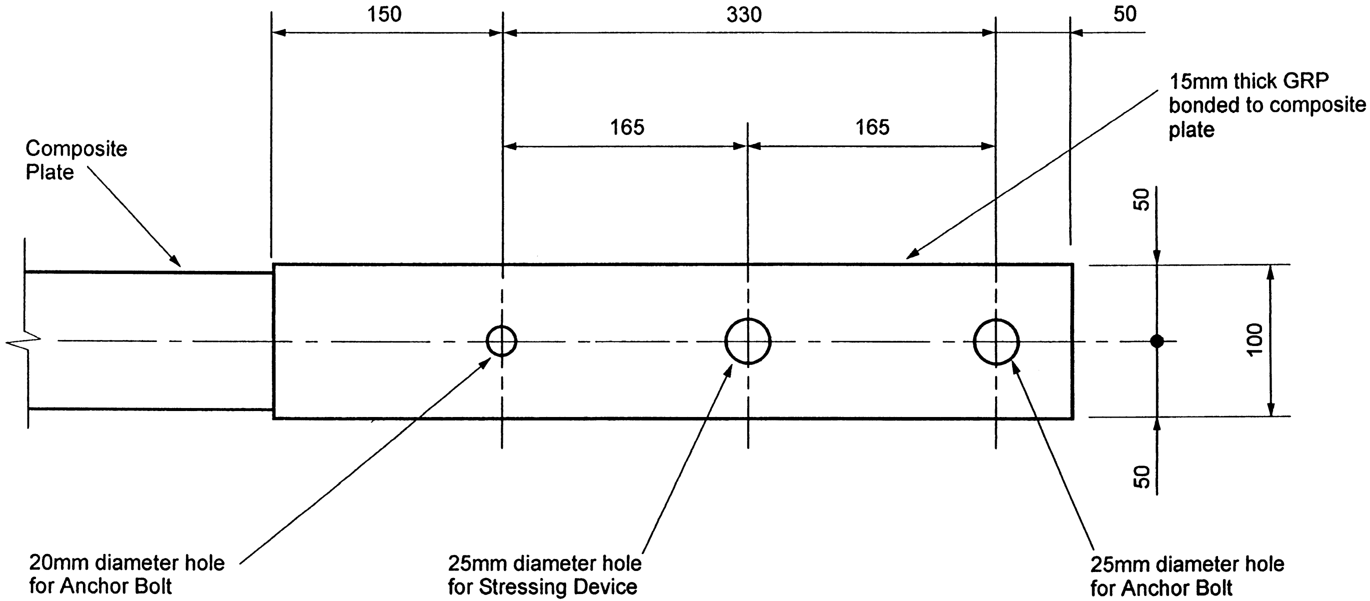


Figure 5.9 Plan of stressed anchor plate.

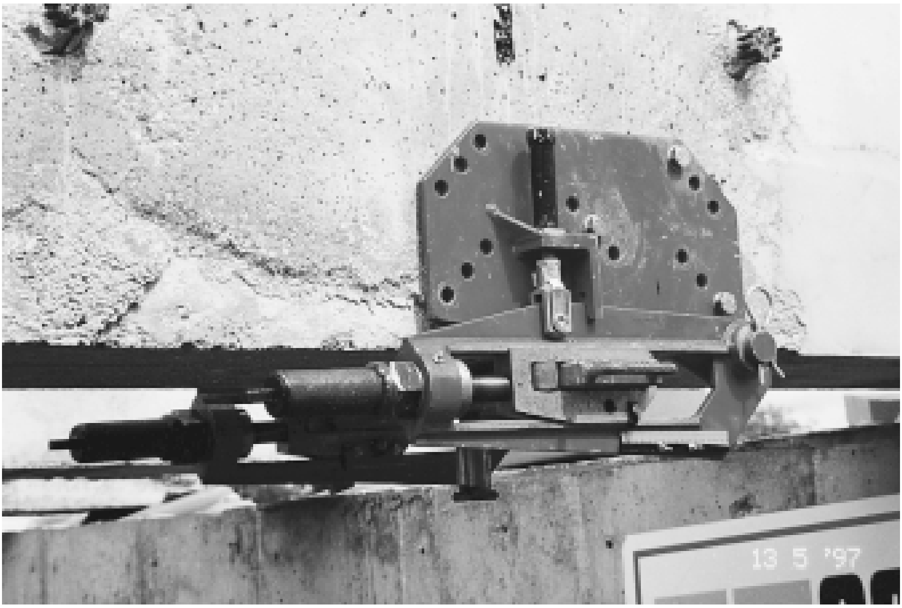


Figure 5.10 Stressing device mounted on beam.

beneath the core hole shows the development of tensile strain in the plate as the internal tendon was cut. As coring took place at other locations, these peak strains were redistributed along the length of the plate. There was no sign of any distress to the beam during this coring operation. The beam was then retested and gave the load–deflection response shown in Fig. 5.13. The stiffness of this plated beam exceeded that predicted by the finite element analysis.

5.6 Observations

The prestressing studies reported previously in the literature, together with the findings of the ROBUST investigative work, lead to a number of significant conclusions regarding the behaviour of beams strengthened with prestressed plates. The more important points may be summarised as follows:

- It has been shown that when bonding composite plates to concrete, the concrete is unable to withstand peeling forces greater than about 5% of the plate strength and, therefore, it is necessary to provide an effective anchorage at the ends of the plate for prestressing forces greater than this value.

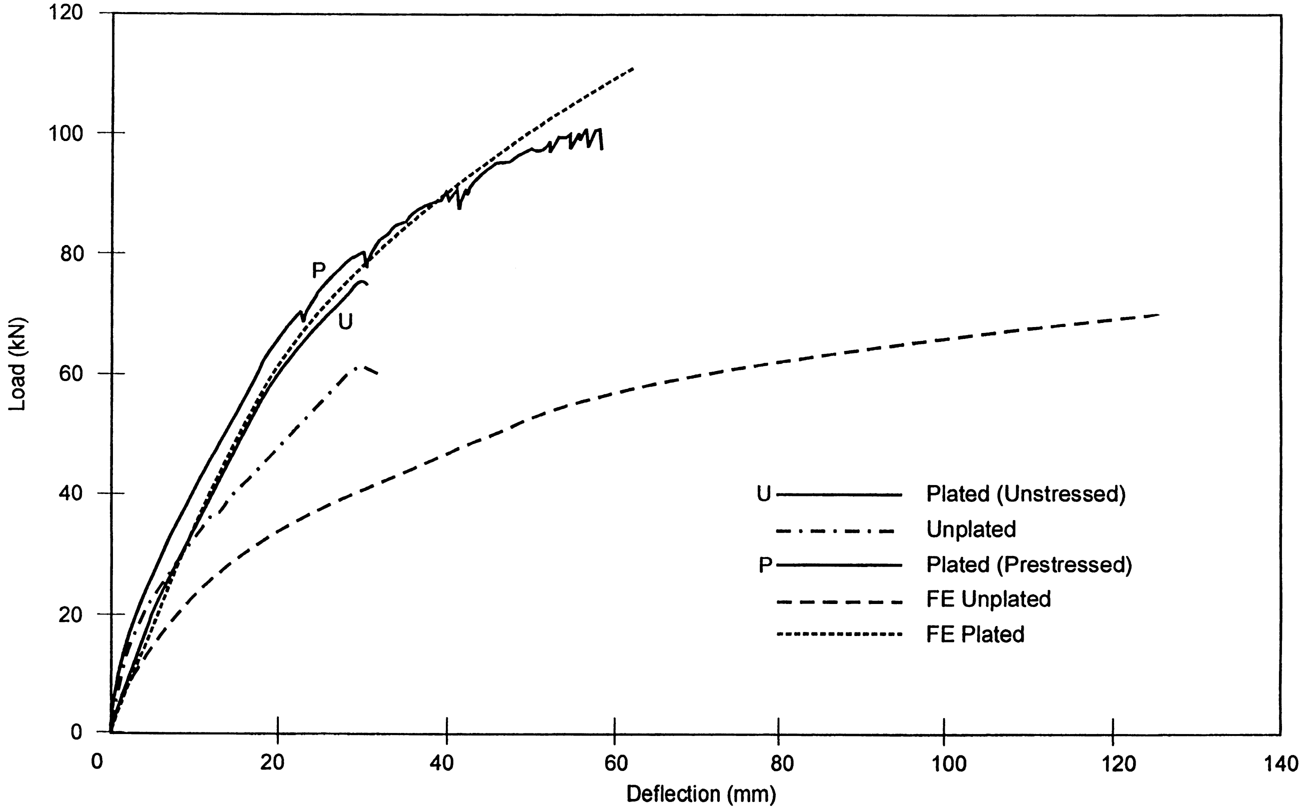


Figure 5.11 Load deflection plots for beam 7.

(a)

(b)

Figure 5.12 Failure mode for beam 7.

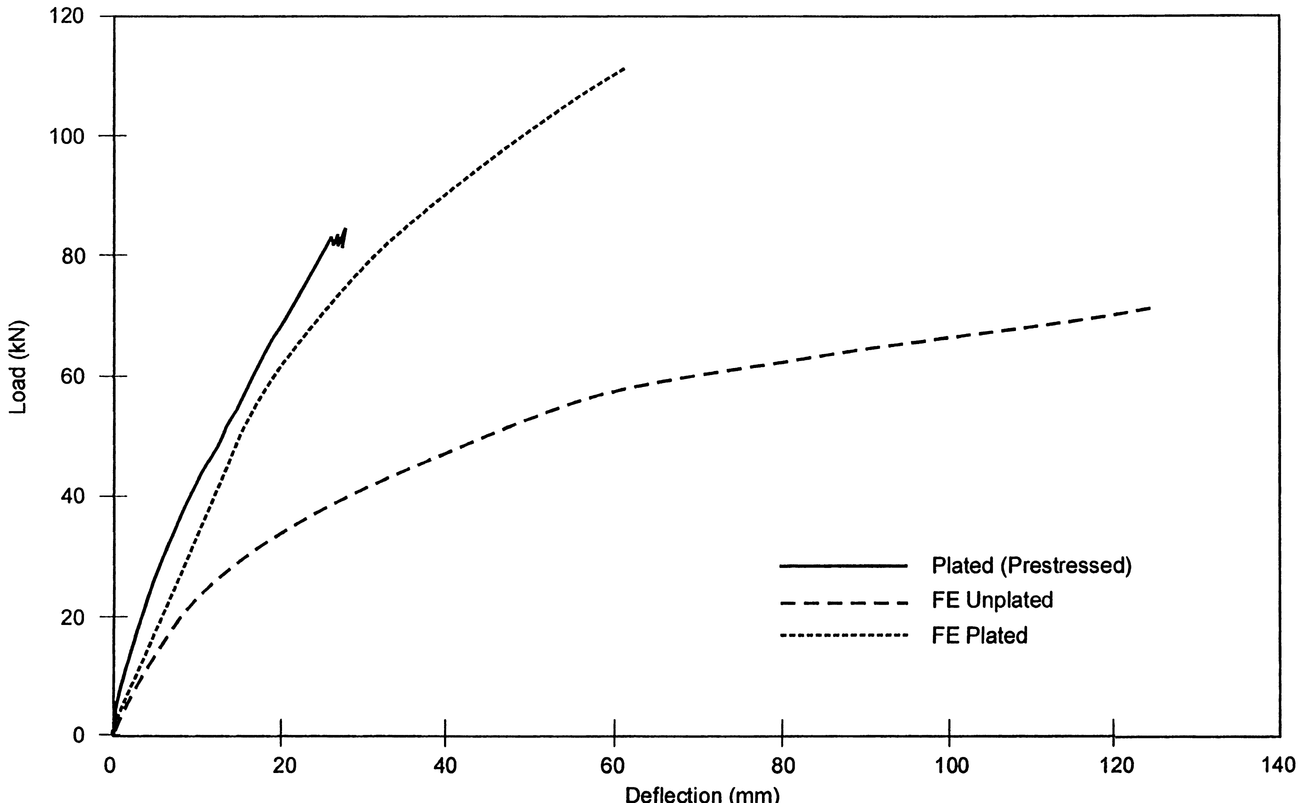


Figure 5.13 Load deflection plots for beam 8.

- It is essential that anchorage bolts are well bonded into the concrete to prevent concrete fracture after prestress transfer and during subsequent loading; the bolts must extend beyond the level of the internal tensile rebars.
- Composite action between the plate and concrete is maintained when the plate prestress is transferred to the concrete.
- Since the lower half of the beam is placed in compression by release of the prestress in the plate, flexural cracking will be much less extensive and develop at a later stage under load compared to an identical non-prestressed beam. This effect of controlling cracks is significant in enhancing the durability of the reinforced concrete with respect to corrosion.
- Prestressing will increase the serviceability load of a given member as long as the stress carried by the internal steel is the governing factor; there is little benefit to be gained when the strength of the concrete governs the serviceability load.
- Prestressing increases significantly the applied load at which the internal steel begins to yield compared to a non-prestressed beam.
- Prestressing the plate prior to bonding also affects the mode of failure. The plate puts the lower half of the beam into compression throughout its length, confining the concrete and reducing the extent of shear cracking which could initiate shear or shear step failure. As a result, the failure will generally occur at the adhesive/CFRP interface or within the bottom layers of the concrete; this results in an increase in failure load for cases governed by a failure mode associated with shear.
- For a non-stressed plated beam where the shear span/beam depth ratio is below 4.0, the failure mode is likely to be due to plate separation initiated at a wide shear crack. However, prestressing the plate prevents or reduces the opening of the shear cracks and a flexural failure is more likely.
- Under a given applied load, the position of the neutral axis is lower when the plate is pretensioned prior to bonding. Consequently, more of the concrete is in compression, resulting in a more efficient use of the material. Furthermore, the heights of the tensile cracks are reduced.
- With increasing applied load, a prestressed plate will delay the point at which the number and extent of shear cracks become significant.
- A prestressed plate with a smaller cross-sectional area can achieve the same benefits of strengthening compared to that of a non-prestressed plate (i.e. improved cracking, yield and ultimate loads), thereby reducing the material cost, although the installation costs will be greater.
- The field investigation demonstrated the feasibility of installing externally bonded prestressed plates on site and has opened the way for further trials on bridge decks and other structures. Beams with pre-

stressed plates withstood higher ultimate loads and were stiffer than their counterparts with unstressed plates. The site tests also demonstrated that the provision of prestressed plates can act as a 'safety net' against the failure of existing tendons in prestressed concrete elements; non-tensioned plates will provide the same effects, assuming that they are adequately anchored at their ends. This failure may occur as a result of corrosion, the reason why the bridge from which the full size test beams were taken was demolished.

5.7 Concluding remarks

When strengthening a member, the level of prestress that can be applied will be limited by the tensile strength of the plate; tensile failure of the plate should not precede either yielding of the internal steel or compressive failure of the concrete to ensure adequate ductility. However, it should be recognised that, as the level of pretension is increased, so the stiffness of the strengthened member is also increased, thereby reducing the tensile and compressive strains. In this case, the member to be strengthened must have adequate shear capacity for the enhanced ultimate load. This requirement is complicated by the fact that increasing plate pretension provides greater crack containment, which increases the diagonal tension strength of the section and may prevent the 'peeling off' failure mode associated with shear 'steps'; this latter may govern failure at lower prestress levels. From the test programme, it appears that the level of FRP pretension may also have to be limited by the strength of the plate end anchorages, by the horizontal shear strength of the adhesive/FRP plate interface and by the bottom layers of concrete.

5.8 References

Garden H N (1997) *The Strengthening of Reinforced Concrete Members Using Externally Bonded Composite Materials*, Doctoral Thesis, University of Surrey.

Garden H N and Hollaway L C (1997) 'An experimental study of the strengthening of reinforced concrete beams using prestressed carbon composite plates', *Proc. 7th Int. Conf. Structural Faults and Repair*, Edinburgh, 1997, Vol 2, pp 191–199.

Garden H N, Hollaway L C and Thorne A M (1998) 'The strengthening and deformation behaviour of reinforced concrete beams upgraded using prestressed composite plates', *J Mater Struct* **31** (May) 247–258.

Leeming M B, Garden H N and Darby J J (1996) 'The rehabilitation of deteriorated post-tensioned beams by bonding with advanced composite plates', *Proc. FIP Symposium on Post-Tensioned Structures*, London, 1996.

Meier U, Deuring M, Meier H and Schwegler G (1992) 'Strengthening of structures with CFRP laminates: research and applications in Switzerland', in *Advanced Composite Materials in Bridges and Structures, 1st International Conference*, eds K W Neale and P Labossière, Sherbrooke, Canada, pp 243–251.

Triantafillou T C and Plevris N (1991) 'Post-strengthening of RC beams with epoxy bonded fiber composite materials', *Proc. Specialty Conference on Advanced Composites Materials in Civil Engineering Structures*, Nevada, 1991, pp 245–256.
Trinh J L (1990) 'Structural strengthening by external prestressing', *Proc. NATO Advanced Research Workshop on Bridge Evaluation, Repair and Rehabilitation,* Baltimore, USA, April 30–May 2, 1990, pp 513–523.

6
Environmental durability

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6.1 Introduction

It is envisaged that many applications of fibre reinforced polymer (FRP) strengthening would be outdoors and hence the durability of the concrete/FRP system under aggressive environments must be considered. The environmental resistance of any bonded assembly depends on the durability of the individual component materials, as well as on the bond between them. In the use of FRP materials for external strengthening of concrete, the individual components are the concrete, the fibre reinforced polymer composite and the adhesive, which is usually an epoxy. In general, properly designed, compacted and cured concrete can be expected to show good long term durability and should remain maintenance free for many years under normal service conditions. The durability of concrete, either in a prestressed or reinforced form is possibly one of the most well studied subjects in civil engineering because of its importance to everyday life and, consequently, it will not be discussed in this book. It should nevertheless be stated that the cover concrete is likely to represent the weakest component in the strengthened zone, so that the integrity of the strengthening system is somewhat dependent upon the properties of the concrete in shear and in tension. In structural applications, the integrity of the adhesive bond and the external FRP strengthening medium under adverse environmental conditions are the issues of prime importance and these will be discussed in the following sections.

Progress in the field of plate bonding relies largely on demonstrating the long term durability of the strengthening system under varying environmental conditions. As a bridge strengthening technique, the minimum required life is 30 years. The environmental durability problems encountered when steel plates are utilised as the strengthening medium are more intractable than those which may arise when composite materials are used. This is because of the difficulties of ensuring an adequate bond between the adhesive and the steel, together with the possibility of electrochemical corrosion of the bonded surface.

One of the most important requirements of an adhesive joint is the ability to retain a significant proportion of its load-bearing capacity for long periods under the wide variety of environmental conditions which are likely to be encountered during its service life. The long term integrity of bonded joints implies both chemical and mechanical durability under varying temperature, moisture and other environmental factors which, for external purposes, may include spray from de-icing salts or from the sea. Adhesive bonded joints with equivalent bond strength values in short term static tests may differ markedly with respect to durability.

The measured residual joint strength after environmental exposure is a function of change in the cohesive properties of the resin, the properties of the adherends and in the adhesion between the adhesive and adherend. Therefore, joint durability demands a three-fold consideration of the structural integrity of the cured adhesive, the adherends and the environmental stability of the interface. Joint design and material data should allow selection of an adhesive type and FRP material which will themselves be sufficiently durable to withstand the service environment. The more complex problem and the far more difficult to design against, is that of the environment attacking the interfacial regions of a joint.

6.2 Environmental and service conditions

The durability of joints and particularly structural adhesive joints is generally more important than their initial strength. Bonded joints used in a civil engineering environment may be subjected to a variety of service conditions. The normal service conditions to be considered are:

- temperature
- moisture (humidity, liquid water, salt spray)
- chemical attack (oil, fuel, chemical spills).

An abnormal hazard condition which may also need to be considered is that of fire.

These service conditions should be considered in conjunction with the loading conditions which, for bridge strengthening, relate primarily to peak short term static loading. Sustained loading (leading to creep), fatigue and impact may also need to be considered.

It has been established that water, in liquid or vapour form, represents one of the most harmful environments for bonded joints (Kinloch, 1983). The problem is that water is found universally and the polar groups which confer adhesive properties make the adhesive inherently hydrophilic. High energy substrate surfaces (Section 3.4) are also generally hydrophilic.

Concrete itself is susceptible to the effects of moisture in a fairly predictable way (Section 6.6). Of greater significance at this stage is that the properties of the matrix resin in FRP materials, together with the properties of adhesives, are susceptible to the effects of heat and moisture. The result of moisture absorption, which is reversible, is to lower the glass transition temperature (T_g) of these materials, leading to a change in their mechanical properties. The effect of elevated temperature is to reduce the strength and modulus of polymers; the T_g of the adhesive is likely to be rather less than that of the matrix resin, so that the adhesive is the governing factor.

Adhesive bonded joints are generally affected by exposure to moisture and elevated temperature. In a well made joint where a sound bond has been achieved, the main effect will be on the adhesive layer. A small amount of moisture-induced plasticisation of the adhesive in highly stressed regions may actually be beneficial in reducing stress concentrations. However, a small reduction in joint strength should normally be anticipated, in relation to the effects of environmental conditions on the adhesive itself.

The resistance of connections to chemical attack depends upon the nature of the liquid and its effect on both the composite components and the adhesive, if present. Alkalis can cause severe matrix resin softening with a consequent effect on any form of connection. Isophthalic polyesters provide better resistance than orthophthalic polyesters in terms of alkalis and organic solvents and are to be preferred for the majority of glass fibre reinforced plastic (GFRP) components. FRP components made with vinyl ester resins are better still, but a little more expensive. Epoxy materials, both as the matrix resin and as adhesives, are regarded as very inert in acids and alkalis.

The resistance of joints to the effects of fire implies consideration of the entire FRP structure, and a useful commentary is contained in the EUROCOMP Design Code (Clarke, 1996). FRP materials and adhesives are very poor conductors of heat, which is an advantage over metals, but they can also possess a large coefficient of thermal expansion in directions that do not have a significant amount of continuous fibre reinforcement. The effect of dimensional changes of the components and joints directly affected by fire or heat should therefore be considered. Adhesives are weakened by the influence of elevated temperature and may char or burn if exposed directly to fire.

6.3 Factors affecting joint durability

It is inadvisable to discuss joint durability without first reviewing the general behaviour and characteristics of bonded joints. A bonded joint represents a layered system comprising different materials and interfaces, all of which respond in different ways to an externally applied load or change in environmental conditions. It follows that a complete understanding of the behaviour of a bonded joint under load is not a simple matter (Mays and Hutchinson, 1992).

A problem with bonded joints is that much of the load is transmitted through edge zones, and it is these which potentially come under environmental attack first. In fact, the load is progressively borne by the inner regions of the joint although, nevertheless, it is often the most highly stressed regions which are under the greatest amount of environmental attack. However, if the bonded area is sufficient to enable stress redistribution within the adhesive layer, any changes in adhesive or cohesive properties will not compromise the integrity of joints of a suitable geometrical configuration. The large areas involved in plate bonding imply that few problems may be anticipated, provided that a high degree of care has been exercised in the design, specification and fabrication of such joints.

The main factors which influence joint performance and durability are:

- adherend type and nature
- porosity/permeability of adherends
- pretreatment
- surface condition following pretreatment
- primer type (if applicable)
- moisture content of adherends at the time of bonding (concrete and FRP)
- adhesive type/cure cycle
- postcuring of joints (if applicable)
- joint configuration and exact geometrical details
- exposure conditions
- duration of exposure
- imposed stress.

It will be appreciated that changes in any of the above factors can give rise to variations in joint behaviour.

One of the most important factors in joint durability is the environmental stability of the adhesive–adherend interface. This is dictated by the type of adhesive, the nature of the adherends and their surfaces. Changes in the adhesive and the adherend can be allowed for; changes in adhesion are less easy to estimate. Thus, optimisation of surface conditions and pretreatments often represents the key to maximising joint durability. The surfaces of both concrete and FRP materials are relatively stable and, if properly prepared (see Section 3.4), durable bonds with epoxy adhesives are formed relatively easily. This is in contrast to the situation with steel plate bonding, for which an adequate standard of surface preparation for mild steel surfaces is quite hard to achieve in practice. Further, the surface of steel is fairly unstable, especially in the presence of water, such that bonds to the oxide layer are susceptible to degradation. The substitution of

FRP materials for steel for strengthening applications is motivated in a large part by the assurance of superior bond integrity.

The adhesive system selected is clearly very important (Section 3.3). Generally a two-part cold curing paste–epoxy material is used without a primer. There are, however, occasions when a primer may be required for concrete surfaces to ensure satisfactory adhesion. Conceptually, adhesives represent natural candidates for joining FRP materials because they are often similar in composition and nature to the composite matrix resin. The fundamental concepts involved in adhesive selection are that it should:

- adhere well to the surfaces involved
- exhibit low permeability to water
- possess appropriate physical and mechanical properties.

The quality of the FRP material itself should be high for several reasons, as discussed in Section 3.2. These include the need for reproducible and predictable properties (for design and prediction purposes), flatness (to ensure uniform bonding and bondline thicknesses) and excellent consolidation (to reduce permeability and mechanical weakness). A poorly made composite will give rise to durability problems sooner or later.

Joint design, as discussed in Chapters 4, 5, and 8, has an important bearing on joint durability. Among the important concepts are to allow for a large bond area and to avoid unacceptably large stress concentrations in the joint. The synergistic effects of high temperature, excess moisture and applied stress normal to the bondline are undoubtedly detrimental. Joint design should therefore seek to minimise the buildup of stress concentrations which give rise to indirect peel and cleavage loads at adherend/adhesive interfaces.

Finally the bonding operation, including protection of the working environment, is important (Section 3.5). Trained operatives working under skilled supervision should ensure that surface preparation, adhesive application, temporary clamping arrangements and adhesive curing details are handled adequately. Poor control of the bonding operation generally manifests itself subsequently in joint performance and durability problems.

6.4 Environmental durability of adhesive bonded joints

6.4.1 General observations

Experience with structural adhesive bonding has shown that the mechanical properties of bonded joints often deteriorate under warm and wet conditions (Bodnar, 1977; Kinloch, 1983). This is particularly so if the joints comprise either high energy substrates such as metals, glasses and ceramics or else permeable substrates such as concrete and timber (Venables, 1984; Mays and Hutchinson, 1992). Further, failure at the adhesive/substrate interface, rather than failure within the adhesive layer itself, is commonly found only after environmental exposure.

With joints involving polymeric adherends, there are separate considerations relating to whether the adherends are thermoplastic or thermosetting in nature. Bonds to thermoset matrix composite surfaces, such as GFRP and carbon fibre reinforced plastic (CFRP), are stable in the presence of moisture; the effect of moisture is primarily to cause some plasticisation of the cured matrix resin. This may or may not affect the mechanical properties of the material and therefore of a bonded joint, depending on the fibre lay-up and mode of loading (e.g. Section 6.6.2).

6.4.2 Diffusion and absorption of water

Water possesses special properties which can be related to its molecular structure and which govern the way its molecules interact with each other and with other substances. The polarity and ability of a water molecule to form hydrogen bonds makes it a universal solvent, allowing it to dissolve, soften or swell organic substances whose molecules contain sufficient polar groups, such as epoxide. Thus, polar adhesives are naturally hydrophilic whereas non-polar plastics, such as polyvinyl chloride (PVC) and polythene, are not. The solubility of water in epoxies is of the order of a few mass percent, and the coefficient of diffusion of water at 20 °C is around $10^{-13}\,m^2\,s^{-1}$ (Mays and Hutchinson, 1992). The permeability of a material is given by the product of diffusion and solubility (Comyn, 1983).

For moisture to affect an adhesive joint it must first enter the joint either by ‘wicking’ along the interface between the adhesive and adherend, by diffusion into the adhesive and adherends, or through cracks and crazes in the materials involved. Wicking may be significant if appropriate surface treatments have not been carried out so that initial adhesion is minimal. However diffusion will be a dominant mode of entry in concrete and polymer composite adherends, together with the presence of cracks in the concrete.

Diffusion, whilst controlled primarily by concentration differences, is influenced significantly by temperature; the higher the temperature, the faster the rate of diffusion. Heat-cured polymers generally possess fairly rigid molecules which reduce their level of molecular motion and hinder water diffusion. The implication is that a pultruded composite matrix material, with a $T_g > 100\,°C$, should be less permeable than a cold-cured epoxy material with a T_g in the range 50–60 °C.

6.4.3 Processes involved in joint degradation

In considering the behaviour of a bonded joint, it is useful to separate the effects of moisture, temperature and stress on the:

- adhesive material
- adherend material
- adhesion between the adhesive and adherend materials.

For the case of using FRP materials to strengthen existing concrete structures it is necessary to consider the effects of these agents separately on the adhesive, the concrete, the polymer composite, the adhesive/concrete interface and the adhesive/polymer composite interface. In general, the main processes involved in the hydrolytic deterioration of bonded joints are (Comyn 1983, 1985):

- absorption of water by the adhesive
- absorption of water by the adherends
- absorption of water at the bonded interface(s) through displacement of adhesive
- corrosion or deterioration of the substrate surfaces(s).

For the case of FRP materials bonded to concrete it is only necessary to consider in detail the first two points; absorption of water at the bonded interfaces, and corrosion or deterioration of the substrate surfaces can be disregarded. If FRP is bonded to other substrates such as timber, cast iron or steel then the considerations are rather more complex (e.g. Mays and Hutchinson, 1992).

The general effects of moisture on polymeric materials are outlined in Section 6.2. These may be summarised by stating that moisture will cause some plasticisation of both the adhesive and the polymer composite matrix material, and that such effects are reversible. The magnitude of such effects is dependent upon the particular formulations of polymer involved and the conditions under which they have been cured.

In GFRP it is known that moisture can attack the surface of glass fibres leading to corrosion, and that adhesion between the fibres and matrix resin may be reduced (see Section 6.6.2). Clearly this will lead to a reduction in the mechanical properties of the material and therefore also of the bonded joints that are made with it. However generalisations are dangerous because of the influence of the fibre sizing, the orientation of fibres with respect to applied loads and the quality and fabrication parameters associated with its production. In aramid fibre composites (AFRP), moisture may be absorbed by the fibres themselves, leading to a loss of fibre properties and therefore composite material properties; in turn this may lead to changes in the behaviour of bonded joints made with AFRP. No such difficulties have been reported for CFRP to date.

6.5 Procedures for assessing environmental effects on materials and on bonded joints

6.5.1 General remarks

A number of standard test procedures exist for assessing the effects of environmental conditions on materials such as concrete, polymer composite materials and on adhesives (in bulk form). Whilst a similar number of standard test procedures exist for assessing the behaviour of adhesive bonded joints, very few of these can provide useful information on environmental effects. Hardly any procedures actually provide quantitative data for the reasons outlined in Section 6.5.3.

The selection of laboratory exposure conditions presents a significant dilemma. Accelerated testing is commonly achieved by increasing the temperature, proximity to moisture and the imposition of load. However, only moderate increases in temperature above likely operating maxima should be used in order to prevent degradation mechanisms taking place which would not occur in practice. For example, the mechanical properties of the adhesive material will be reduced above its T_g, water uptake will increase markedly above T_g and hydrolysis of some adhesive materials may occur. Freeze–thaw cycling is favoured for construction applications in particular, where thermal shock or freezing of clustered water molecules may give rise to joint failures either directly or indirectly. BS EN 29142 (1993) describes several single and multivariable atmospheric ageing regimes.

When considering the use of accelerated testing involving elevated temperatures, it is advisable to postcure joints constructed with cold cure adhesives. This is to prevent the exposure environment itself from postcuring the adhesive and altering its mechanical properties. Clearly it is necessary to maintain a known base line of joint performance and this can only be achieved by being able to eliminate the effects of temperature itself on behaviour. The joints described in Section 6.7.3 were postcured at 50 °C prior to exposure.

It is most useful to collect information on joints exposed to natural weathering conditions because some workers have found that natural exposure is worse than laboratory ageing. This may be related to the effects of cyclic environmental effects. Exposure in hot/wet climates gives rise to faster joint degradation than exposure in more temperate climates. Sea coast exposure is also generally more demanding than exposure to industrial environments.

6.5.2 Experimental considerations – adhesive and composite material behaviour

Bulk material characterisation represents a useful adjunct to durability trials on joints, enabling environmental effects on the adhesive and composite materials themselves to be separated from those on adhesion and on overall joint behaviour. It has already been stated that the influence of water (and heat) on the adhesive and on a composite matrix resin is generally reversible; water uptake is accommodated largely by swelling and its effect is to reduce T_g. The modulus and strength of adhesives and matrix resins are lowered by plasticisation although fracture toughness and ductility generally increase (Figs. 3.6 and 3.7).

Tests may be carried out using adhesive materials in bulk form which have been cast to shape. Typical geometrical configurations include tensile dumb-bells, blocks for compression, rods or strips for torsion pendulum tests and films for water sorption experiments. Quantitative information on shear stress–strain behaviour can also be obtained from elaborate joints which ensure 'pure' shear deformation in the bondline. Fracture mechanics specimens employing cantilever beam arrangements can be used to obtain values of tensile opening (mode 1) fracture energy, G_1, in order to map toughness as a function of environment (see Section 3.3.6).

Short term tests on fibre reinforced polymer composite materials are well documented in various Standards, and many of these can be extended into environmental exposure trials. For example, flat rectangular test pieces can be subjected to tensile and bending stresses in order to determine strength and modulus following periods of accelerated ageing. Small pieces can also be used for water sorption experiments.

6.5.3 Experimental considerations – bonded joints

In the majority of test joint configurations, the adhesive bondline stresses are far from uniform. Failure loads are therefore related to stress concentrations of the ends of a joint, so that if the bonded area is sufficient to enable stress redistribution within the adhesive layer due to changes in the adhesive material properties, apparent increases in joint 'strength' and 'toughness' may occur initially.

A number of competing mechanisms are taking place and the effects of these are more noticeable in smallish joints unless initial adhesion is very poor. Test joints using small bonded areas are preferred for durability testing in order to minimise experimental timescales. Thus the small joints which are generally used do provide fairly rapid information, but the quite dramatic changes in joint behaviour which sometimes occur should not necessarily cause undue alarm. Small joints can provide useful comparative information on, say, the behaviour of different bonding systems or the effectiveness of different surface treatments. However the size effects inherent in testing mean that no direct correlation with the behaviour of large real-scale joints can be made.

Appropriate comparative tests of adhesion and bond durability should subject the interface to tensile, peel or cleavage forces. Thus pull-off tests, peel tests, fracture energy tests and, ironically, lap-shear tests are routinely employed. The choice depends largely upon the nature of the adhesives and substrate materials involved.

Special consideration needs to be given to the modes of loading imposed on joints involving both concrete and polymer composite materials. Concrete represents a brittle substrate material of low tensile strength, such that direct loading of joints made with it tends to result in premature failure of the concrete. Thus joint configurations which put the concrete into compression are often recommended to avoid substrate failure. Such configurations include slant shear (Fig. 6.1) and compressive shear tests (Fig. 6.2), resulting in a measure of the resistance of a bondline to a combination of shear and compression. Three- and four-point bending tests are also employed (Fig. 6.2), resulting in a measure of average shear or shear and tension resistance. Notwithstanding the above, the partially cored pull-off test (Fig. 6.3a) is used routinely for testing the bond integrity of concrete repair materials. It has also been used in modified form to assess bonds between epoxy adhesives and concrete surfaces (Fig. 6.3b). Results using this procedure are outlined in Section 6.7.2.

Polymer composites are also relatively brittle materials. The distribution and orientation of the fibre reinforcement has an enormous influence on the

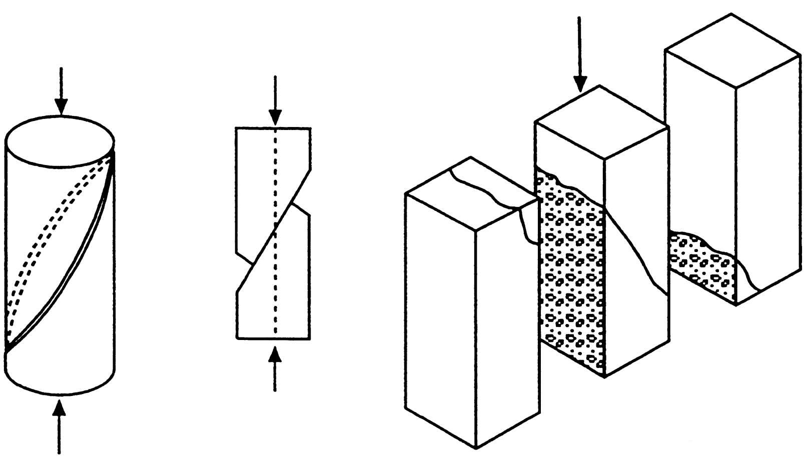

Figure 6.1 Different types of slant shear test configurations for joints involving concrete substrates.

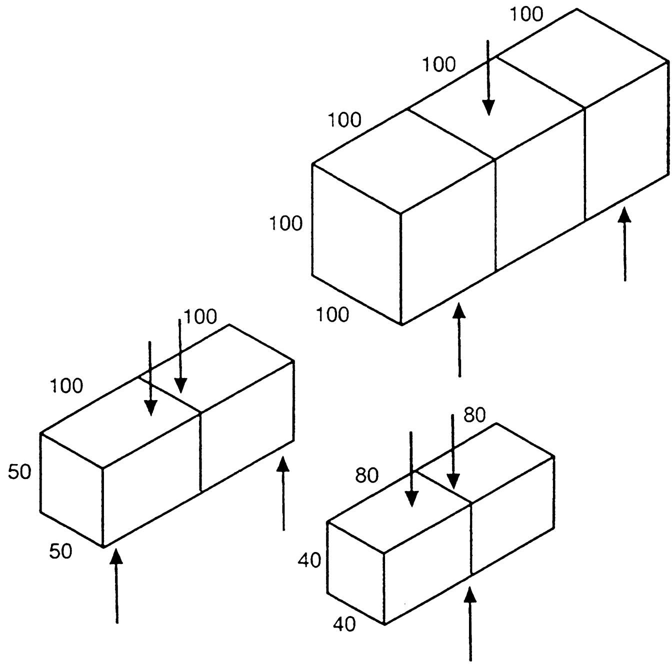

Figure 6.2 Different types of bending and compressive shear tests for joints involving concrete substrates.

load-carrying capacity of joints. In general, the through-thickness strength of composite materials is low because of the layered composition and, sometimes, resin-rich layers. Bonded joints are therefore prone to interlaminar failure. Laboratory experience has shown that single lap shear joints and wedge cleavage joints are satisfactory for use where a high proportion of fibres are parallel to the principal direction of loading. Single lap shear data is given in Section 6.7.3.

6.6 Effect of environment on the component materials used in the ROBUST system

To verify that the ROBUST material systems could maintain stability, separately and in combination under hot/wet environments, the consortium experimentally studied exposed samples of Sikadur 31PBA adhesive and the carbon fibre/vinylester polymer composite strengthening plates. Reinforced concrete (RC) beams 0.8 m long strengthened with these materials were also manufactured and their specifications and results are discussed in Section 6.7.4.

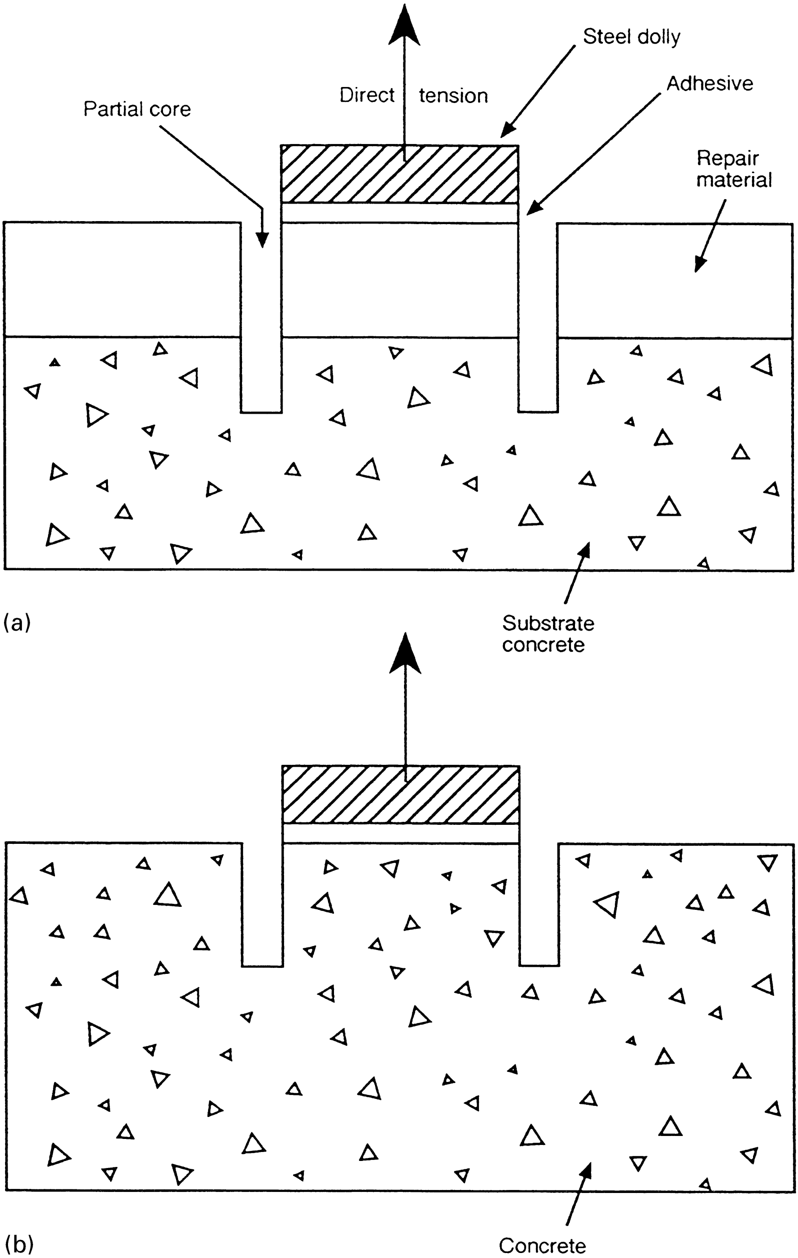

Figure 6.3 Partially cored pull-off tests.

The test and exposure conditions adopted for the Sikadur 31 PBA adhesive, composite materials and plated beams were as follows:

- Thermal cycling of Sikadur 31PBA adhesive, GFRP and CFRP composite plate coupons and unloaded CFRP plated RC beams, between temperatures of −20 °C and +50 °C, represented warming and cooling effects. One cycle took 24 h with the rising temperature taking 5 h and cooling temperature taking 17 h; at the extreme temperatures there was a dwell time of one hour. During exposure, the natural moisture in the air condensed on to the material coupons of adhesive, CFRP and GFRP. After each of the exposure periods of 0, 50 and 180 thermal cycles the specimens were loaded to failure to assess the influence of the freeze–thaw environment.
- Sikadur 31PBA adhesive, GFRP and CFRP composite plate coupons were exposed to a warm, moist atmosphere of 30 °C and 100% relative humidity. They were tested after 0, 50 and 180 days; in each case the material was tested 24 h after its removal from the humid atmosphere.

The results from these tests are described in Sections 6.6.1 and 6.6.2, and summarised in Tables 6.1 and 6.2.

6.6.1 Sikadur 31PBA adhesive

The general properties and characteristics of this two-part cold curing filled epoxy adhesive are discussed in Section 3.3, together with the effects of moisture on its properties. In general, the type of structural adhesive used can affect both the rate and degree of environmental attack on bonded joints (Minford, 1983). A bonded joint may be appreciably weakened if the adhesive is chemically attacked to any significant extent by the service environment. The detailed chemistry of the adhesive also appears to influence the stability of the interfacial regions due to the formation of more stable intrinsic interfacial forces. Furthermore, the physical impact which the service environment has on the adhesive is dependent on the composition of the adhesive.

The more highly filled the adhesive, the lower should be its long term water absorption (Tu and Kruger, 1996). The absorption of moisture tends to accelerate time-dependent processes by lowering the T_g, thereby reducing the performance of the adhesive at high temperatures. As such, moisture combined with heat has a particularly unfavourable influence on the adhesive. However, whether plasticisation of the adhesive layer by the ingress of the water actually affects the strength of the bonded joint is hard to predict since, for many joint configurations, a decrease in the adhesive modulus may decrease the stress concentrations in the joint and lead to an increase in joint strength. Similarly, the toughness of joints subjected to fracture often increases somewhat because of greater plastic deformation and enhanced crack-tip blunting properties within a plasticised matrix (Ripling *et al.*, 1971; Kinloch and Shaw, 1981; Kinloch, 1982). Cohesive strength may, however, eventually be reduced sufficiently to offset the increased toughness (Hutchinson, 1986).

The mode of failure of bonded joints when initially prepared is usually by cohesive fracture in the adhesive layer, or possibly in the substrate materials if these are particularly weak. However, a classic symptom of environmental attack is that after such exposure, the joints exhibit some degree of interfacial failure between the adhesive and the adherend (Brewis *et al.*, 1982). The extent of such failure increases with time of exposure to the hostile environment. Whether the failure path is truly at the interface, or whether it is within a boundary layer of the adherend or adhesive remains a matter of debate. Several authors (e.g. Brockmann, 1983; Kinloch, 1987) have emphasised that the structure of the cured adhesive adjacent to the adherend surface differs from that of the bulk, because of the influence of surface morphology and chemistry on the initial wetting and absorption of adhesive. The inference is that this weak boundary layer of adhesive may be less densely cross-linked and/or have a lower concentration of filler particles than that of the bulk material, and may therefore be more susceptible to hydrolytic destruction. The rate of interfacial transport of water could also be somewhat higher than that through the bulk material.

To enable some material characterisations of the adhesive used in the ROBUST system to be obtained, bulk coupons were exposed to the hot and wet environmental conditions described earlier.

Table 6.1 shows the adhesive characteristics after the three periods of the two types of exposure; the modulus is given as the initial tangent modulus

Table 6.1 Adhesive material characteristics after environmental exposure (Garden and Hollaway, 1997)

Exposure type and duration	Modulus of elasticity (GPa)	Tensile strength (MPa)	Ultimate strain (microstrain)
Thermal cycling			
Unexposed (control)	9.0 (± 21.1%)	26.3 (± 27.0%)	4300 (± 46.5%)
50 thermal cycles	11.3 (± 0.5%)	29.6 (± 10.8%)	3467 (± 18.3%)
100 thermal cycles	11.5 (± 1.1%)	31.4 (± 9.7%)	4302 (± 17.6%)
180 thermal cycles	11.8 (± 19.7%)	38.2 (±13.3%)	4333 (± 19.2%)
Humidity exposure			
Unexposed (control)	9.0 (± 21.1%)	26.3 (± 27.0%)	4300 (± 46.5%)
50 days	7.8 (± 1.2%)	13.4 (± 5.3%)	3663 (± 22.5%)
100 days	6.4 (± 10.2%)	16.2 (± 7.2%)	5284 (± 21.5%)
180 days	5.7 (± 19.7%)	15.7 (± 2.5%)	7980 (± 20.8%)

value at zero stress. In the thermal cycling environment the stiffness of the material increased in value, indicating a continuing polymerisation process. However, the negative influence of the humidity atmosphere on the adhesive indicates an increase in ductility associated with the plasticisation of the material. The ultimate strain increase indicates moisture uptake with plasticisation occurring and the experiment does reflect the serious effect of a high humidity environment. However it should be noted that plate bonding systems, whether for strengthening of bridge or building structures, will not generally be exposed to such extreme environments.

6.6.2 FRP adherends

Joints involving FRP adherends are far less susceptible to environmental attack by water than are joints made with adherends with higher energy surfaces such as metals (see also Section 3.4). However, in most environments, polymer composites show a degree of change with time. In Section 6.4.3 it was stated that the most important factors in inducing such change are moisture and natural weathering, and to these might be added elevated temperatures. In addition, the effects of sunlight, particularly the UV component, can have some influence on degradation. Separately, and in combination, these factors may all contribute to a deterioration in properties.

The effects of the service environment on the mechanical, physical and chemical properties of polymer composites depend to a large extent on the efficiency with which the composite is prepared, in particular the quality of exposed surface. However, the contribution of the polymer resin system used is the most important factor in relation to durability. The stability of the polymer depends on the chemistry and conformation of the molecules, and is strongly dependent on the cross-linked structure in thermosetting polymers. Since the resins used to manufacture composite materials and adhesives subsequently used to bond them are both polymeric, consideration of the effects of the environment are equally applicable to both cases.

In civil engineering structures it should be mentioned that the composite will not normally be heated to a temperature near to the limit of its mechanical performance. This temperature is dependent upon the manufacturing technique, but would typically be in excess of 140° for polyesters, vinylesters and epoxies. However, temperature exposure has three effects on polymer composites. In conditions of constantly fluctuating temperatures, differences in the thermal expansion coefficients of the resin and reinforcing fibres could contribute to progressive debonding and weakening of the materials. However, for well prepared composites (this invariably implies mechanical production) this is not generally a problem.

Unreinforced polymers have very high coefficients of thermal expansion, but the values are considerably reduced by the addition of fibres and fillers. The effect of temperature on the matrix properties is usually reversible unless the T_g is approached. The resistance of materials to strain under load is highly dependent on temperature because of the viscoelastic properties of the matrix. It must be stated that the above conditions are the worst scenario for composites and, although plate bonding which is undertaken inside buildings is not generally exposed to continually fluctuating or large variations of temperature, these effects might be relevant to bridge strengthening where externally bonded plates are exposed to diurnal and seasonal fluctuations.

As discussed for adhesives in Sections 6.2 and 6.6.1, moisture is usually a more significant factor than heat in causing deterioration of composites, since it is likely to have both chemical and physical effects on all of the components individually and on their interaction. The effects of moisture depend upon the specific polymer and fibre materials utilised in the manufacture and design of the composite, as well as the duration and global environment to which it is exposed. Moisture will be absorbed by the composite if the resin is sufficiently hydrophilic (Loos *et al.*, 1981). In addition, the ingress of water through capillary channels, voids and at exposed cut surfaces will affect the rate and extent of degradation since moisture molecules destroy some of the chemical bonds in the resin. However, this problem is mainly relevant to GFRP composites. The ROBUST system uses CFRP in which the matrix is vinylester; this system has excellent resistance to corrosive environments and to moisture uptake (see Section 3.2).

Moisture reduces the T_g of polymeric resins and has a plasticising effect which, in turn, affects the properties of FRP composites. However, changes in matrix moduli have little effect on longitudinal tensile strength and modulus of the composite since the matrix plays only a minor role in these properties. In addition, the decrease in elastic modulus of the resin due to any uptake of water is reversible in that the modulus returns to its original value when the moisture diffuses out. As the coefficient of thermal expansion of unidirectional carbon fibre/polymer composite of about 60–65% by weight of fibre is almost zero (carbon fibre has a negative coefficient of thermal expansion), the ultimate tensile strength of this type of composite is relatively insensitive to temperature in the range $-73\,^{\circ}$C to $+107\,^{\circ}$C, regardless of the moisture content of the material. The temperature range over which plate bonding would be utilised is well within these limits. Temperatures in the range $-73\,^{\circ}$C to $+177\,^{\circ}$C were observed by Shen and Springer (1981) to have a negligible effect on the modulus of elasticity, regardless of the moisture content of the material (their moisture content varied from dry to fully saturated). It might also be supposed that moisture will have little effect on the tensile properties if the fibres themselves are unaffected, as should be the case with carbon.

As stated in Section 6.2, composites composed of glass fibres in polyester matrix can be vulnerable to environmental attack but their resistance does increase when going from orthophthalics to isophthalics to vinylester polymers (Norwood, 1994). Furthermore, a reduction in strength of the fibres under atmospheric conditions has been observed within short periods of time after they have been subjected to load. This static fatigue or stress corrosion results from a chemical reaction between water vapour and the surface of the glass that permits a pre-existing flaw to grow to critical dimensions and bring about spontaneous crack propagation. The rate of this reaction is dependent on the magnitude and local stress conditions, as well as the temperature, pressure and composition of the surrounding atmosphere. ROBUST confirmed that CFRP composites were far superior to GFRP under these atmospheric conditions and consequently this system concentrated upon CFRP composite.

Table 6.2 shows the results of the CFRP and GFRP tensile tests following exposure of the specimens to the thermal cycling and humidity regimes

Table 6.2 Composite material characteristics after environmental exposure (Garden and Hollaway, 1997)

Material type and exposure time	Modulus of elasticity (GPa)	Tensile strength (MPa)	Ultimate strain (microstrain)
CFRP thermal cycling			
Unexposed (control)	110.88 (± 4.2%)	1414 (± 7.6%)	12340 (± 6.0%)
50 thermal cycles	113.40 (± 5.1%)	1386 (± 6.2%)	11827 (± 5.2%)
100 thermal cycles	116.28 (± 3.4%)	1627 (± 7.9%)	14060 (± 7.1%)
180 thermal cycles	121.44 (± 2.2%)	1734 (± 8.1%)	14348 (± 6.0%)
GFRP thermal cycling			
Unexposed (control)	36.09 (± 2.9%)	955 (± 6.5%)	26185 (± 7.3%)
50 thermal cycles	36.65 (± 3.5%)	1056 (± 5.2%)	28512 (± 5.8%)
100 thermal cycles	37.62 (± 2.5%)	973 (± 5.8%)	28104 (± 8.8%)
180 thermal cycles	37.17 (± 4.8%)	1004 (± 4.8%)	19350 (± 7.3%)
CFRP humidity exposure			
Unexposed (control)	110.8 (± 4.2%)	1414 (± 7.6%)	12340 (± 6.0%)
50 days	109.95 (± 3.8%)	1587 (± 8.5%)	13967 (± 5.2%)
100 days	111.77 (± 4.5%)	1476 (± 3.7%)	13471 (± 3.7%)
180 days	114.57 (± 3.5%)	1609 (± 8.8%)	14326 (± 2.3%)
GFRP humidity exposure			
Unexposed (control)	36.09 (± 2.9%)	955 (± 6.5%)	26185 (± 7.3%)
50 days	35.82 (± 3.4%)	862 (± 4.8%)	23813 (± 6.0%)
100 days	36.31 (± 2.6%)	857 (± 5.9%)	26116 (± 6.5%)
180 days	35.71 (± 1.8%)	835 (± 5.4%)	27664 (± 7.8%)

described earlier. Mean data points are quoted, the values in parentheses representing the maximum deviation as a percentage. The moduli of the CFRP specimens subjected to thermal cycling exhibited a consistent increasing trend with increasing exposure time, whereas those values for the GFRP specimens showed a decreasing trend. The lack of consistent trends for all other values suggests that the exposure conditions had no conclusive influence on those material properties.

All tests were conducted at a time of 24 h after removal from the temperature cabinet to enable them to reach the surface dry state; the specimens were stored under ambient laboratory conditions during this time.

6.6.3 Key observations

The observations that the ROBUST consortium made from these environmental tests were that:

- The adhesive mechanical properties were increased by the temperature cycling, the modulus of elasticity increasing rapidly initially. The postcuring effect of temperatures greater than the cure temperature was believed to be responsible for the improvement in properties.
- The CFRP composite experienced an increase in modulus and strength due to the thermal cycling between −20 °C and +50 °C. This was possibly caused by postcuring of the epoxy matrix, indicating the superior performance of CFRP compared with GFRP which experienced no significant improvement.
- Under elevated temperature and humidity there is softening of the adhesive, whilst the CFRP composite became stiffer and stronger but to a lesser extent than under thermal cycling exposure. These effects were not considered to be significant.

6.7 Influence of surface treatment and effects of environment on joints and interfaces

6.7.1 Introduction

It was stated in Section 6.3 that optimisation of substrate surface conditions and pretreatments often represents the key to maximising joint durability. Whilst the surfaces of both concrete and polymer composite materials are relatively stable, they do require adequate preparation for structural bonding (Section 3.4). Central to optimisation of surface conditions to provide joints of high strength and durability is the adoption of appropriate mechanical test techniques and methods.

There is a clear emphasis at the beginning of this section on the use of test procedures which seek to test the integrity of bonded interfaces. Later subsections deal with an assessment of the overall performance of small scale beams.

6.7.2 Joints involving adhesive–concrete interfaces

The tensile pull-off test is probably the most frequently used on-site test method for assessing the quality of concrete and also for examining the adhesion of repair materials and coatings. Tests may be conducted to determine both initial and long term 'strength' following environmental exposure.

The partially cored pull-off test (PrEN 1542, 1996) was employed in the ROBUST project. This was used to assess the effects of concrete surface preparation and concrete surface moisture content on the bond performance of Sikadur 31PBA, a two-part cold cure paste epoxy resin. Essentially, steel dollies were bonded to the concrete pull-off locations as defined by 50 mm diameter partial cores to a depth of 15 mm (Fig. 6.3b). The bondline thickness was of the order of 2 mm and a minimum time period of 7 days elapsed before any pull-off tests were carried out. The characteristic concrete cube strength was 42 MPa and the typical age of the concrete at the time of testing was around 20 weeks.

The test parameters used were:

- Three concrete surface moisture contents, generated by:
  - 14 weeks laboratory ageing (20 °C/50% rh)
  - 8 weeks laboratory ageing, 3 weeks water immersion at 20 °C, followed by 3 weeks laboratory ageing
  - 11 weeks laboratory ageing, followed by 3 weeks water immersion at 20 °C.
- Two types of surface treatment:
  - low pressure alumina grit blasting (just exposing small aggregate particles)
  - high pressure chilled iron grit blasting (exposing medium sized aggregate particles).
- Two types of ageing following exposure:
  - 50 24 h freeze–thaw cycles (−18 °C to +18 °C)
  - laboratory ageing (20 °C/50% rh)

The results of the concrete pull-off tests are given in Table 6.3. The average pull-off strengths ranged from 1.65–2.91 MPa with the lower figures corresponding to concrete subjected only to laboratory conditions prior to adhesive bonding. Curiously, the highest figures corresponded to joints made with the dampest concrete and then subjected to freeze–thaw cycling. In all cases the locus of failure was cohesive within the concrete. No distinction could be drawn between the relative merits of 'low' and 'high' pressure

Table 6.3 Results of partially cored pull-off tests (Rahimi, 1996)

Concrete conditioning before bonding	Concrete surface treatment		Conditioning after bonding		Test results		Failure mode
	low pressure alumina grit blasting	high pressure chilled iron grit blasting	50 24 h freeze–thaw cycles (−18 °C to 18 °C)	ambient laboratory conditions (20 °C, 50% rh)	average failure load (kN) with SD in brackets	average pull-off strength (MPa)	
14 weeks under ambient laboratory conditions (20 °C, 50% rh)	X		X		5.08 (1.13)	2.59	within concrete
	X			X	3.24 (1.2)	1.65	″
		X	X		5.20 (0.75)	2.65	″
		X		X	3.80 (0.34)	1.94	″
8 weeks under ambient laboratory conditions plus 3 weeks water immersion at 20 °C followed by 3 weeks under ambient laboratory conditions	X		X		5.26 (0.7)	2.68	″
	X			X	4.38 (1.41)	2.23	″
		X	X		5.62 (1.36)	2.86	″
		X		X	4.54 (0.45)	2.31	″
11 weeks under ambient laboratory conditions followed by 3 weeks water immersion at 20 °C	X		X		5.36 (0.88)	2.73	″
	X			X	5.72 (0.69)	2.91	″
		X	X		5.14 (0.93)	2.62	″
		X		X	4.78 (1.46)	2.43	″

Failure loads represent the average of four to five test results.

grit blasting. Furthermore the dampness of the substrate did not affect the bond between this particular adhesive and the concrete, at least as determined by this test procedure.

6.7.3 Joints involving adhesive–FRP interfaces

The lap shear joint is that used almost universally in testing adhesives or surface preparation techniques. It owes its popularity to its convenience of manufacture and test, as well as to the fact that the adhesive is subjected to cleavage as well as to shear. It thus simulates the actual use of an adhesive in a variety of applications, including that in plate bonding.

The single lap shear joint is suited to assessing qualitatively the adhesion and bond integrity between materials, since joints made with relatively stiff adhesives and thin adherends fail by a cleavage mechanism. Data so generated are qualitative only, but the locus of joint failure can provide information on the durability of the bond. Joints may be subjected to environmental exposure both unstressed and stressed in suitable fixtures.

Single lap shear joints fabricated generally in accordance with ASTM D3163 (1973) were used in the ROBUST project. In fact the adherend coupons measured only 60 mm × 20 mm, the joint overlap was set at 10 mm, and the bondline thickness of Sikadur 31PBA adhesive was 0.5 mm. The joints were postcured at 50 °C for 6 h prior to environmental exposure. The test parameters used were:

- peel-ply surface treatments for the adherends
- two types of polymer composite adherends (fabricated from prepreg materials): GFRP (1.8 mm thick) and CFRP (1.2 mm thick)
- unstressed exposure for up to 1.5 years at 40 °C/95% rh
- three levels of exposure at 40 °C/95% rh under stress at 10% to 30% of the initial control joint strength.

A comparison of the results of unstressed exposure is shown in Fig. 6.4 (Rahimi, 1996). The somewhat erratic response in the first few weeks simply indicates some plasticisation of the bondline perimeter, resulting in a relief of bondline stress concentration and changes in joint behaviour. Thereafter the dominant effect of plasticisation of both the bondline and the adherends takes over, resulting in an overall reduction of perhaps 15% in the joint strength. The reduction in strength stops somewhere between 20 and 50 weeks. In all cases the locus of failure remained cohesive within the adhesive layer, except for the initial joints tested. This is very encouraging and indicates a stable adherend/adhesive system.

The results of the stressed joints are shown in Fig. 6.5 (Rahimi, 1996). It is clear that as the applied stress increased, the time to failure decreased, as expected. Again the locus of failure for all joints remained

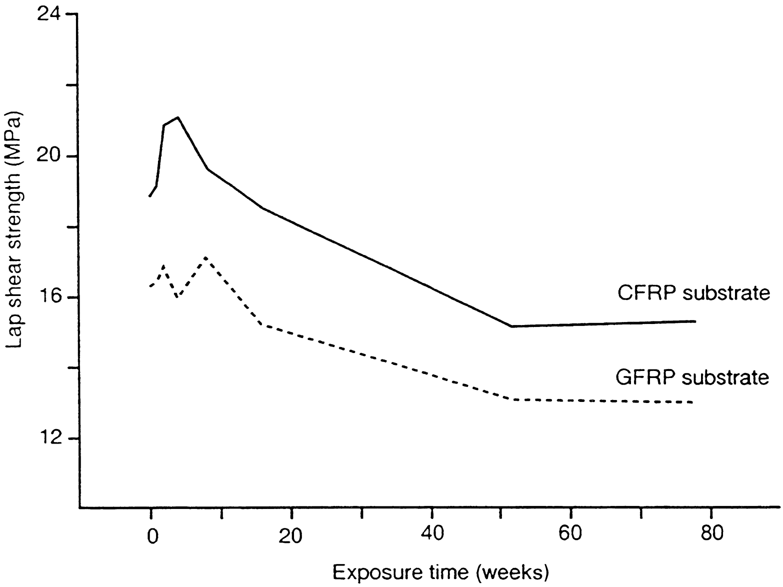

Figure 6.4 Effect of exposure at 40°C/95% rh on the performance of unstressed single lap shear joints made with CFRP and GFRP adherends.

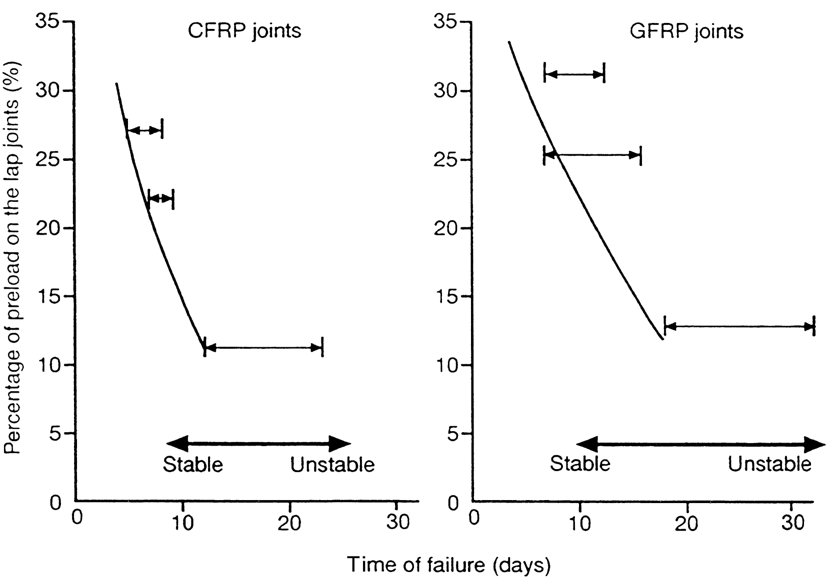

Figure 6.5 Effect of exposure at 40°C/95% rh on the performance of single lap shear joints made with CFRP and GFRP adherends, stressed at different levels.

within the adhesive layer. The data suggest that joints made with such small bonded areas are unable to carry sustained loads for long periods, but this should not cause alarm for the joint size-effect reasons stated in Section 6.5.3.

For the case of the particular adherend/adhesive systems studied here, the results indicate that there was some water-induced plasticisation of the small scale laboratory joints. This was caused predominantly by some softening of the adhesive layer, an outcome which small joints are far more sensitive to than large ones, but which nevertheless can be designed for. The most important point, which gives great confidence, is that the bond between the adhesive and FRP materials was stable under hot/wet conditions.

6.7.4 Effects of environment on strengthened small scale beams

To investigate the environmental effects on the ROBUST plated beam system, RC beams 0.8 m long and 100 mm $\times$ 100 mm cross-section were used. The reinforcement of the concrete consisted of three, 6 mm diameter steel rebars in the tensile region and two, 6 mm diameter rebars in the compression region; the beams were over-reinforced in shear. The CFRP composite plates had a cross-section of 65 mm $\times$ 0.7 mm. The concrete was 54 MPa grade with an elastic modulus of 35 GPa. The beams were exposed to the same environmental history as described for the individual components in Section 6.6.2; the thermally cycled specimens were unloaded during the exposure but the humidity-exposed specimens were loaded in four point bending as specified below.

The testing regime for the post-environmental exposure (thermal cycling) utilised a four point loading system in which the constant moment length was 100 mm and the span was 740 mm. This arrangement was also used during the humidity exposure and the total load applied to the beam during this time was 17 kN. This represented the value of load that would cause yield of the steel reinforcement and this was well above the serviceability load of that beam.

Table 6.4 shows the results for the plated RC beams after a period of 180 thermal cycles between $-20\,°C$ and $+50\,°C$. The values obtained for the beams in terms of comparative strengthening and stiffening effects are shown relative to identical beams tested directly to failure.

The beams exposed to the hot humid conditions showed no appreciable creep (measured by removable deformation dial gauge) or fall-off of load (measured by load cells permanently in position). There was no detectable degradation of either of the beams.

Table 6.4 Strengthening comparison of 0.8 m long durability beams subjected to thermal cycling (Garden and Hollaway, 1997)

Beam	Yield load (kN)	Ultimate load (kN)	Postyield stiffness ($kN\,mm^{-1}$)
Unexposed (control)	24.0	48.9	7.3
Beam 1	26.0	46.9	5.4
Beam 2	25.0	44.0	6.3

6.8 Other factors affecting service performance

6.8.1 Freeze–thaw action

Consideration has been given to repairing bridges in cold countries, particularly where structural strength needs to be restored following damage by corrosion to reinforcing steel. Green and Soudki (1997) devised some experiments on small columns and beams to investigate the effects of freeze–thaw action. One 24 h freeze–thaw cycle consisted of 16 h at $-18\,°C$, followed by 8 h in water at $+18\,°C$. Cylinders 150×300 mm wrapped with CFRP sheets were subjected to 200 cycles; beams $1200 \times 150 \times 100$ mm were plated with both CFRP and GFRP, and then subjected to 50 cycles. They concluded that the FRP sheets were very effective at restoring the strength of the columns damaged by freeze–thaw action and that the strengthened beams behaved in a similar way to their counterparts maintained at laboratory ambient temperatures. The FRP materials and the bond between the concrete and the FRP sheets were unaffected by the freeze–thaw cycling.

6.8.2 Fire

The vinylester polymer (Palatal A430-01), which was the polymer used in forming the pultruded composite plate in the ROBUST project, is a non-fire-retardant resin. During the project no attempt was made to increase the fire resistance of the pultruded composite as the aim of the research work was specifically to investigate strengthening of the reinforced concrete beams by composite plate bonding.

However, polymer materials composed of carbon, hydrogen and nitrogen atoms (i.e. they are organic materials), are all inflammable to varying degrees and could suffer some deterioration if exposed to fire unless some form of protection against it is provided (Hollaway, 1993). It is possible to incorporate additives into the resin formulations or to alter their structure, thereby modifying the burning behaviour and producing a composite with enhanced fire resistance. Furthermore, it is possible to provide a fire protective coating after the plate is bonded into position onto the external surface of the structural unit.

The fire hazards of polyester resins (vinylester is an enhanced polyester) may be reduced by incorporating halogens; halogens are one of the fluorine, chlorine, bromine and iodine family of chemicals. Fire hazards can also be reduced by combining synergists such as antimony oxide into the resin formulations; these are commonly known as hot acid resins. The commonest way, and indeed the cheapest, of obtaining these is by the addition of chlorinated paraffin and antimony oxide or by the utilisation of halogenated phosphates such as trichloroethylphosphate. Such systems when combined with fibres would conform to the British Standard Test for Fire Retardance, Class 2, of BS 476 Part 7.

To enable polyester composites to be used in building applications, resins have been developed which can attain a Class 1 rating, thus complying with the Building Regulations. Thus numerous other means of introducing halogens, either by additives such as pentabromotoluene and tris(dibromopropyl)phosphate or by reactants such as tetrabromophthalic anhydride or dibromoneopentyl glycol have been developed. The effects of alternative synergists and inert fillers have resulted in the availability of clear and opaque resins which can be fabricated into composites to give a Class 1 fire rating.

An alternative method of fire protection is by employing intumescent coatings. These can be based on polymer resins with three additives:

- a source of phosphoric acid
- a polyhydroxy compound which reacts with phosphoric acid to form a char
- a blowing agent.

In a fire situation these coatings yield a carbonaceous expanded char which protects the underlying composite. Structures treated by this method can develop fire ratings of Class 1 and Class 0.

Deuring (1994), undertook fire tests on RC beams, plated with carbon fibre/epoxy matrix polymer composites, bonded with Sikadur 30 S-02 and Sikadur 31 SBA S-08 epoxy adhesive. Some of the plates were protected with 60 mm thick PROMATECT-L.

When decisions are required regarding fire protection, it is advisable for the engineer to discuss the most appropriate system to be used with the resin manufacturers. If fire protection is required for plate bonding it is unlikely that the additives, which would be incorporated into the resin, would affect the strength of the composite significantly, but the in-service properties of colouring and UV might be affected. However, as the polymer composite plate is located on the soffit of a concrete beam, and therefore protected from the direct rays of the sun, any degradation from this source will be minimal. Additionally, any change in the colouring of the composite will be masked by the black carbon fibre; consequently, from the weathering point of view, degradation will be insignificant. It is worth mentioning, however, that if UV degradation is a problem, UV stabilisers can be added to the liquid resin in addition to any additives for fire resistance.

6.9 Summary

The main considerations relating to the environmental durability of externally bonded reinforcement are the quality of the materials used and the integrity of the adhesive bonds. The ROBUST system of strengthening represents an environmentally stable system with good durability as demonstrated in laboratory experiments. This behaviour results from the use of good quality unidirectional CFRP/vinylester matrix material manufactured by the pultrusion process, an epoxy adhesive which exhibits good all round physical and mechanical characteristics (together with low permeability to water), excellent adhesion to concrete and excellent adhesion to CFRP conferred by use of the peel-ply surface preparation technique coupled with the wetting characteristics of the adhesive. It has been shown that the materials, and bonds to them, are very stable within the normal range of operating conditions anticipated.

6.10 References

ASTM D3163 (1973) *Standard Test Method for Determining the Strength of Adhesively Bonded Rigid Plastic Lap-shear Joints in Shear by Tension Loading*, American Society for Testing and Materials, Philadelphia.

Bodnar M J (ed.) (1977) *Durability of Adhesive Bonded Structures*, J Applied Polymer Science, Applied Polymer Symposia 32, New York, J Wiley & Sons.

Brewis D M, Comyn J and Shalash R J A (1982) 'The effect of moisture and temperatures on the properties of an epoxide-polyamide adhesive in relation to its performances in single lap joints', *Int. J. Adhesion Adhesives* **2**(4) 215–222.

Brockmann W (1983) in *Aspects of Polymeric Coatings*, ed. K.L. Mittal, Plenum Press, New York, pp 265–280.

BS EN 29142 (1993) *Adhesives – Guide to the Selection of Standard Laboratory Ageing Conditions for Testing Bonded Joints*, BSI, London.

Clarke J L (ed.) (1996) 'Structural design of polymer composites', *EUROCOMP Design Code and Handbook*, London, E & F N Spon.

Comyn J (ed.) (1985) *Polymer Permeability*, Elsevier Applied Science, London.

Comyn J (1983) 'Kinetics and mechanisms of environmental attack', in *Durability of Structural Adhesives*, ed. A J Kinloch, London, Applied Science, Chap. 3.

Deuring M (1994) *Brandversuche an nachtraglich verstärken Trägern aus Beton, Report No. 148795*, EMPA (in German).

Garden H and Hollaway L C (1997) *Environmental Durability of Composites*, Internal report of the University of Surrey.

Green M F and Soudki K A (1997) 'FRP strengthened concrete structures in cold regions', *Proc. Int. Conf. on Recent Advances in Bridge Engineering – Advanced Rehabilitation*, eds. U Meier and R Betti, EMPA Dubendorf, Switzerland, pp 219–226.

Hollaway L C (1993) *Polymer Composites for Civil and Structural Engineering*, Blackie Academic and Professional, Glasgow.

Hutchinson A R (1986) *Durability of Structural Adhesive Joints*, PhD Thesis, University of Dundee.

Kinloch A J (1982) *J. Mater Sci* **17** 617–651.

Kinloch A J (1983) *Durability of Structural Adhesives*, Applied Science, London.

Kinloch A J (1987) *Adhesion and Adhesives: Science and Technology*, Chapman and Hall, London.

Kinloch A J and Shaw S J (1981) in *Developments in Adhesives – 2*, ed. A J Kinloch, Applied Science, London, pp 83–124.

Loos A C, Springer G S, Saunders B A and Tungl R W (1981) 'Moisture absorption of polyester E-glass composites' in *Environmental Effects on Composite Materials*, ed. G S Springer, Technomic, Westport, USA, pp 1–62.

Mays G C and Hutchinson A R (1992) *Adhesives in Civil Engineering*, Cambridge University Press.

Minford J D (1983) in *Durability of Structural Adhesives*, ed. A J Kinloch, Elsevier Applied Science, London, pp 135–214.

Norwood S R (1994) 'Fibre reinforced polymers', in *Handbook of Polymer Composites for Engineers*, ed. L C Hollaway, Woodhead Publishing, Cambridge.

PREN 1542 (1996) *Products and Systems for the Protection and Repair of Concrete Structures – Test Methods – Pull-off test*, BSI, London.

Rahimi H (1996) *Strengthening of Concrete Structures with Externally Bonded Fibre Reinforced Plastics*, PhD Thesis, Oxford Brookes University.

Ripling E J, Mostovoy S and Bersch C F (1971) 'Stress-corrosion cracking of adhesive joints', *J. Adhesion* **3**(2) 145.

Shen C H and Springer G S (1981) Effects of temperature and moisture on the tensile strength of composite materials', in *Environmental Effects on Composite Materials*, ed. G S Springer, Technomic, Westport, USA, pp 79–92.

Tu L and Kruger D (1996) 'Engineering properties of epoxy resins used as concrete adhesives', *ACI Mater J* **93**(1) 26–35.

Venables J D (1984) 'Review: durability of metal-polymer bonds', *J Mater Sci* **19** 2431–2453.

7
Time-dependent behaviour and fatigue

R BARNES AND H N GARDEN

7.1 Introduction

This chapter is divided into two parts, the first concentrating upon creep deformation characteristics of the component parts of a reinforced concrete (RC) plated beam and the beam system itself. The second part of the chapter discusses the fatigue characteristics of the component parts and the plated beam. The two parts will present results of tests conducted during the ROBUST project to determine these two properties for the ROBUST plating technique.

Part A Time-dependent behaviour

7.2 Introduction

The resin systems used to manufacture both fibrous composites and the adhesives subsequently used to bond them are polymeric materials and, as such, the same considerations in terms of time dependency apply to both. In any situation where an adhesive connection is required to transmit stresses due to sustained load, the possibility of creep in the adhesive must be considered. Long term bond integrity implies, amongst other things, the avoidance of excessive creep and the possibility of creep-induced failures throughout the service life.

In this particular application, since the adhesive is the component which transfers load to the external plate, creep within the bondline may affect the structural performance of the strengthened member. The action of creep on an adhesive is to cause a degradation of the effective modulus of the material, or a loss of rigidity. This may reduce the stress transferring ability of the adhesive and thus the efficiency of the strengthening system. Similarly, creep of the external fibre reinforced polymer (FRP) plate itself would result in time-dependent increases in member deflection under the action of a constant load. The fact that three of the materials involved in the application are viscoelastic and that the behaviour and interaction of these materials may affect the structural performance of the overall system, make it necessary to understand the time dependency of this system.

For applications involving bridge rehabilitation or upgrading, the external plate is most likely to be bonded to the structure without additional propping, so that none of the member's self weight is supported by the plate, the system only providing strengthening for superimposed or live loads applied subsequently. In such circumstances, long periods of sustained loads are unlikely. However, if the technique of pretensioning the external FRP plate prior to bonding is implemented, the plate will remain heavily loaded at all times and the adhesive will be under sustained stress. Furthermore, for building strengthening applications, the level of live load or superimposed dead load may be reasonably constant for long periods of time, placing the plate, and thus the adhesive, under conditions of sustained stress.

7.3 Time-dependent characteristics of concrete

The relationship between stress and strain for concrete is a function of time. Under sustained stress, concrete undergoes a gradual increase in strain, referred to as creep, most conveniently taken as an increase in strain above the initial elastic value. Creep generally has little effect on the strength of a structure, but will cause an increase in deflections under service loads and a redistribution of stress. Considerable research effort has been put into the subject (for example, Evans and Kong, 1996; Neville, 1970; ACI, 1982) and a complete discussion will not be given here.

7.4 Time-dependent characteristics of steel

When metal is subjected to a stress, slow plastic deformation, usually referred to as 'creep', can occur under constant load, even when the stress level lies below the elastic limit. Creep associated with reinforced and prestressed concrete can have a considerable effect upon the durability of the material. The satisfactory performance of the rebars depends upon their ability to maintain a high elastic tensile stress throughout the lifetime of the structure. Further information on the creep of steel in conjunction with concrete can be found in Neville *et al.* (1983).

7.5 Time-dependent characteristics of adhesives

The mechanical properties of polymers have characteristics of both elastic solids and viscous fluids, and hence they are classified as viscoelastic materials. On application of load, the material deforms instantaneously like an elastic solid. However, if the load is maintained, the material continues to deform to some extent in a manner similar to a viscous fluid. Therefore, for full characterisation of such a material, a knowledge of its time-dependent response is necessary. The transition from relatively elastic behaviour to relatively viscous behaviour often occurs within a timescale in the range of concern for practical application.

The viscoelastic behaviour of polymers is well documented; a comprehensive text is given by Ferry (1980). Viscoelastic effects can result in creep deformation or stress relaxation. Figures 7.1 and 7.2 illustrate a simplified creep strain–time relationship for a polymer under a uniaxial stress state and for a shear creep strain–time relationship for an adhesive lap joint, respectively. Creep may lead to delayed fracture, termed creep rupture, under long term loading. In this case, the applied stress should be lower than that required to cause fracture under monotonic loading conditions. The time-dependent strain of the material divided by the constant applied stress is termed the creep compliance. Since adhesives are polymeric

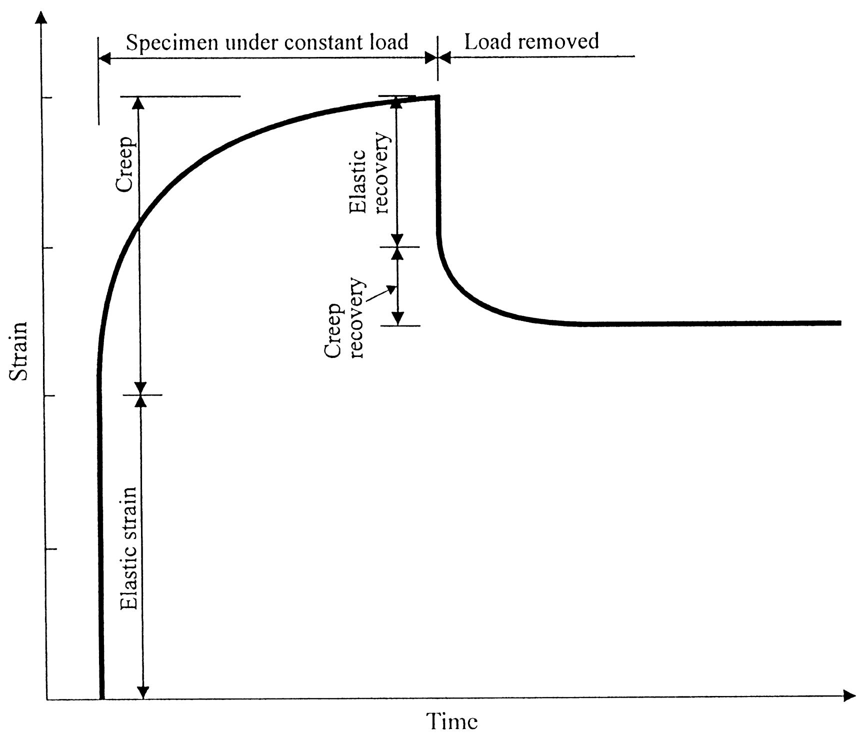

Figure 7.1 Simplified creep strain–time relationship for a polymer under uniaxial stress state.

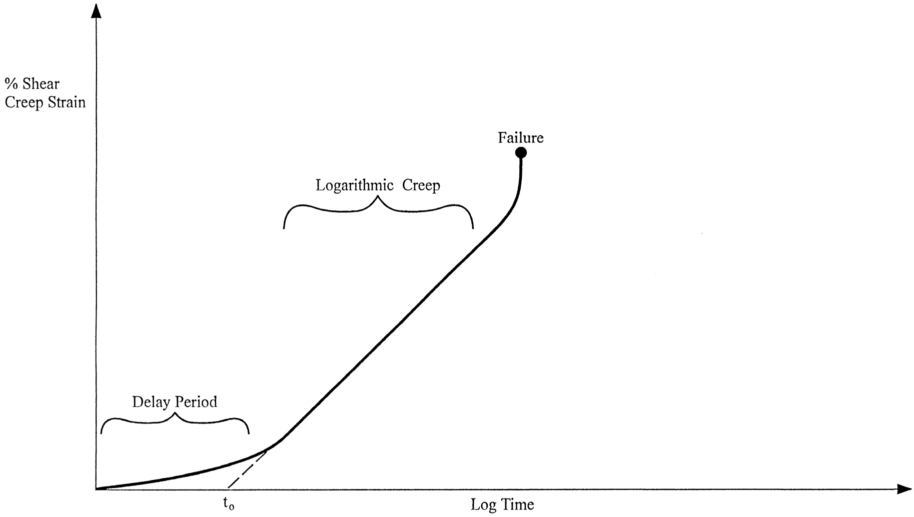

Figure 7.2 Schematic representation of a creep versus time graph for a structural adhesive joint.

materials, they exhibit viscoelasticity and as such are subject to time-dependent behaviour.

The creep of adhesives in general is affected by a number of parameters, including the chemical nature of the resin system, the volume and types of filler used, the dimensions of the bond area and the absorption of plasticisers such as moisture or oil. However, the main factors are the level of the applied load, time and temperature. In addition, the magnitude of the creep response for thermosetting resin systems is highly dependent on its cure history, particularly if under working load the resin is continuously loaded and is also at an elevated temperature. Creep rate varies with stress level, and in general the higher the stress the greater the creep rate. At low applied stress levels, many polymers behave in a viscoelastic manner, in which the strain response of the material at any time of loading is a linear function of the constant applied stress. This implies that the principle of superposition can be used for changes in applied stress (Hollaway, 1993). However, at higher stress levels, the viscoelastic behaviour of polymers can become highly non-linear. Perhaps of greatest importance to structural adhesive joints in civil engineering is behaviour at relatively low stress levels, since bonded joints tend to be designed to withstand low stresses for a long period of time rather than the higher stresses which may cause rapid creep to failure. For example, in steel plate bonding applications, it is recommended that any sustained stress in the adhesive bonded joint be kept below 25% of the short term joint strength to minimise creep effects (Mays, 1993).

When an adhesive is placed under load, the stress is not distributed evenly throughout the adhesive but concentrates in certain areas within the bondline. With lap shear joints for instance, the maximum stresses occur at the ends of the overlap. It is these maximum stresses which are thought to determine the overall creep behaviour of the joint. Since typical joints involve high stresses near the free ends, even relatively low load levels may result in significantly non-linear behaviour. As such, the practical approach both to delaying the onset of creep and to reducing its rate is by providing large bonded areas.

The ambient temperature also affects the occurrence and rate of creep deformation. As the temperature passes the glass transition temperature, T_g, of an adhesive, there is a marked change in the creep properties exhibited by an adhesive bonded joint (Adams and Wake, 1984). For the majority of adhesives, an increase above the normal operating temperatures for which the material is designed causes an increase in the creep rate; then once the T_g is reached, a further increase occurs. The creep behaviour of an adhesive at elevated temperatures can be affected by a number of factors including the proportion of fillers, the shape of the specimens, the stress levels encountered and the extent of the temperature rise. To limit the effects of creep under sustained load, an adhesive possessing a T_g well above the service temperature is required. In general, an increase in relative humidity plasticises the adhesive in the joint, reducing its stiffness and strength. It has also been found that water or high humidity has the effect of lowering the T_g of the material. If the range through which the T_g drops coincides with the temperature of the joint then this will greatly affect the adhesive strength and cause an increase in creep. However, if the range through which the T_g drops is far from the operating temperature of the joint then the effect of moisture on the modulus of elasticity of the adhesive will be of primary importance, since this would also affect the creep properties of the adhesive. In general, the more highly cross-linked the hardened adhesive structure and the higher the curing temperature and hence T_g, the better the creep resistance (Mays and Hutchinson, 1992).

7.5.1 Creep testing of structural adhesives

There are basically two different types of creep test available for the characterisation of structural adhesives; these involve the utilisation of bulk material tested in tension and the adhesively bonded joints using a single or double lap joint. The latter test more nearly simulates the material as it is used in practice.

The creep of Sikadur 31, a two-part epoxy resin adhesive, similar to that used in the ROBUST project, was studied at varying stress and temperature levels alongside a toughened single part hot cured epoxy paste (Permabond

ESP 110) which was examined under identical long term loading (Lark and Mays, 1984; Mays, 1990). A double lap steel joint was employed, the adherends being manufactured from 25.4 mm × 6.4 mm mild steel with an overlap length of 80 mm. The two-part adhesive was cured at room temperature whilst the hot cured adhesive was cured at 100 °C. Control tests were performed at room temperature at an age of 7 days, the failure load being recorded and the mean shear strength determined by dividing the failure load by the bonded area.

Creep testing rigs based on the dead load and lever principle, with a capacity of up to 45 kN, were employed. Room temperature (20 °C) and relative humidity (50% rh) were maintained by an air conditioning system. Where specimens were tested at elevated temperatures of up to 65 °C, the temperature was maintained using a heating tape coiled around the adhesive joint.

Room temperature creep curves for the above two-part epoxy resin adhesive, expressed in terms of log strain against log time (h), are presented in Fig. 7.3. The mean shear stress at which each test was conducted is shown adjacent to each curve. It can be seen that the initial slope of the logarithmic creep curve increases with applied stress. It should be mentioned that the stress values used in these tests are much greater than would occur in practice.

The relationship between the applied mean shear stress and time to failure is illustrated in Fig. 7.4 for the two-part cold cure adhesive. Examining Fig. 7.4 reveals a small reduction in performance between tests at 20 °C and 40 °C but a more significant reduction at 55 °C. Figure 7.5 shows the shear stress versus time to failure for the single-part hot cure epoxy resin adhesive and it will be seen that there is a small reduction in performance at 55 °C and 65 °C compared with 20 °C. The glass transition temperatures for the two adhesives were measured at 44 °C and 105 °C for the cold cure and hot cure adhesives, respectively. The poor creep performance of the cold cure adhesive at 55 °C (a value above its T_g) demonstrates the effect of temperatures above the glass transition value.

From a civil engineering point of view, it is important to establish the maximum values of stress which can be sustained so as to ensure survival for the design life of the structure. In Table 7.1 (Barnes and Mays, 1988) this maximum stress value (expressed as a fraction of the short term strength) is displayed for service times of 1 year, 30 years and 120 years. The values were obtained by extrapolating the logarithmic stress versus time to failure mean lines, a method which must be undertaken with some caution. Based on the statistical analysis applied to the results a lower bound was also obtained.

This work indicates that the sustained stress in an adhesive joint should not exceed 25% of the short term strength for the normal design life of the

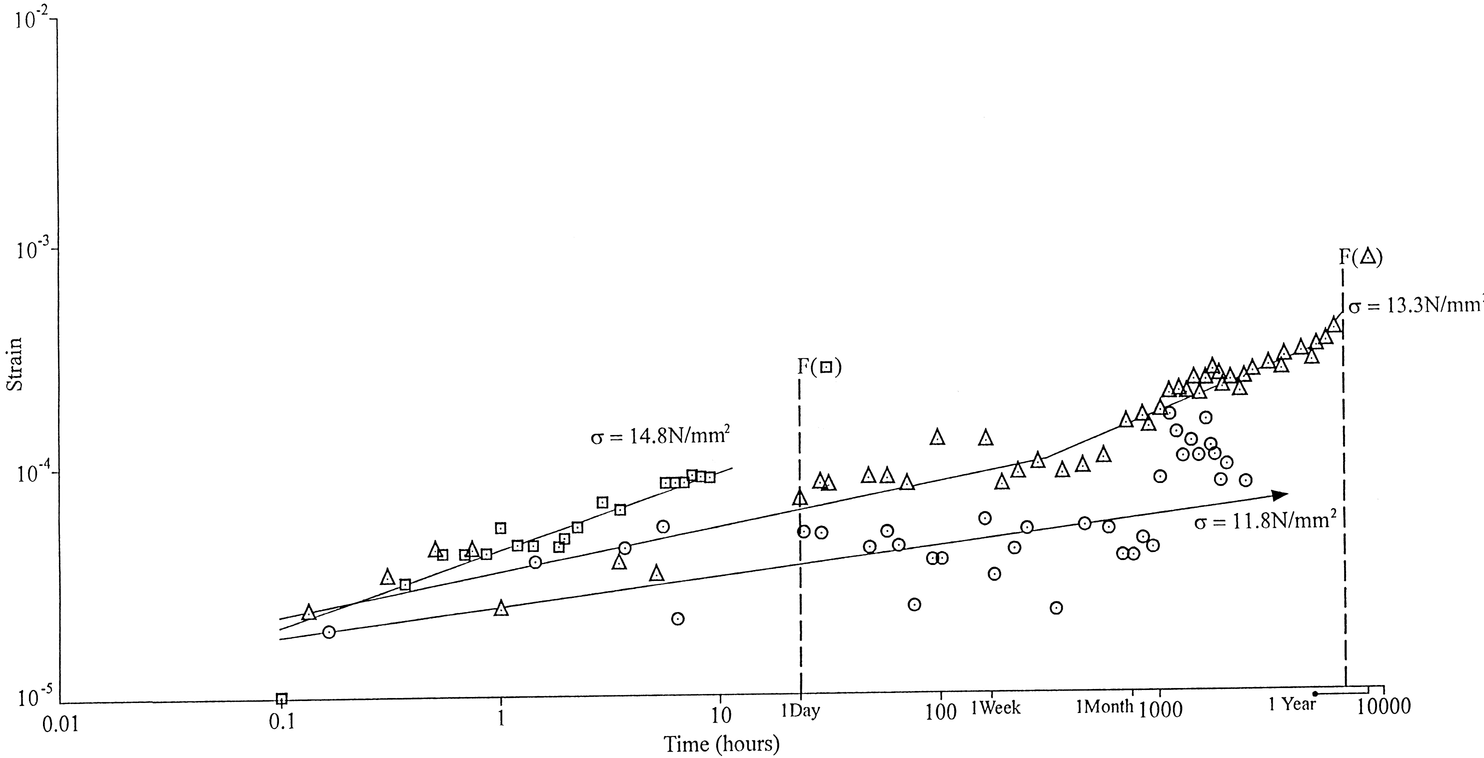

Figure 7.3 Creep curve for two-part epoxy resin adhesive at 20°C (Mays, 1990).

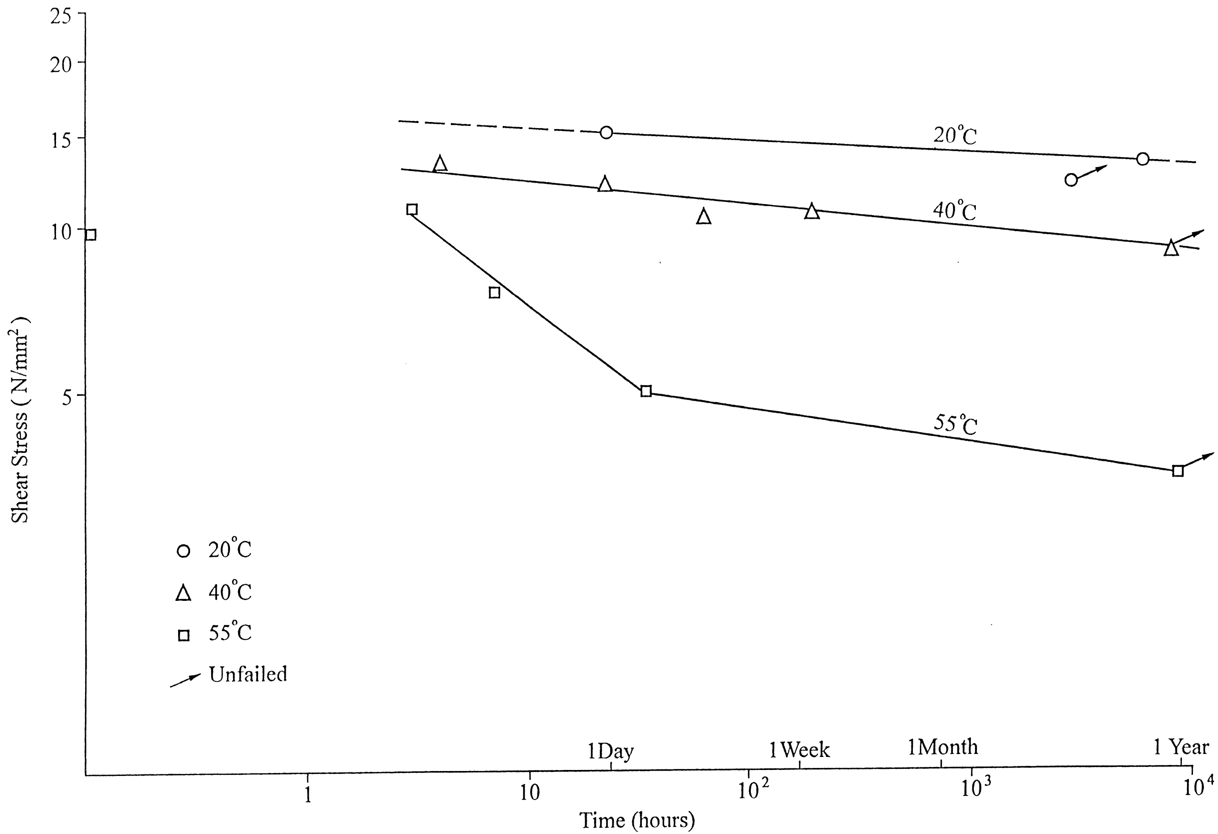

Figure 7.4 Shear stress versus time to failure for two-part cold cure epoxy resin adhesive (Mays, 1990).

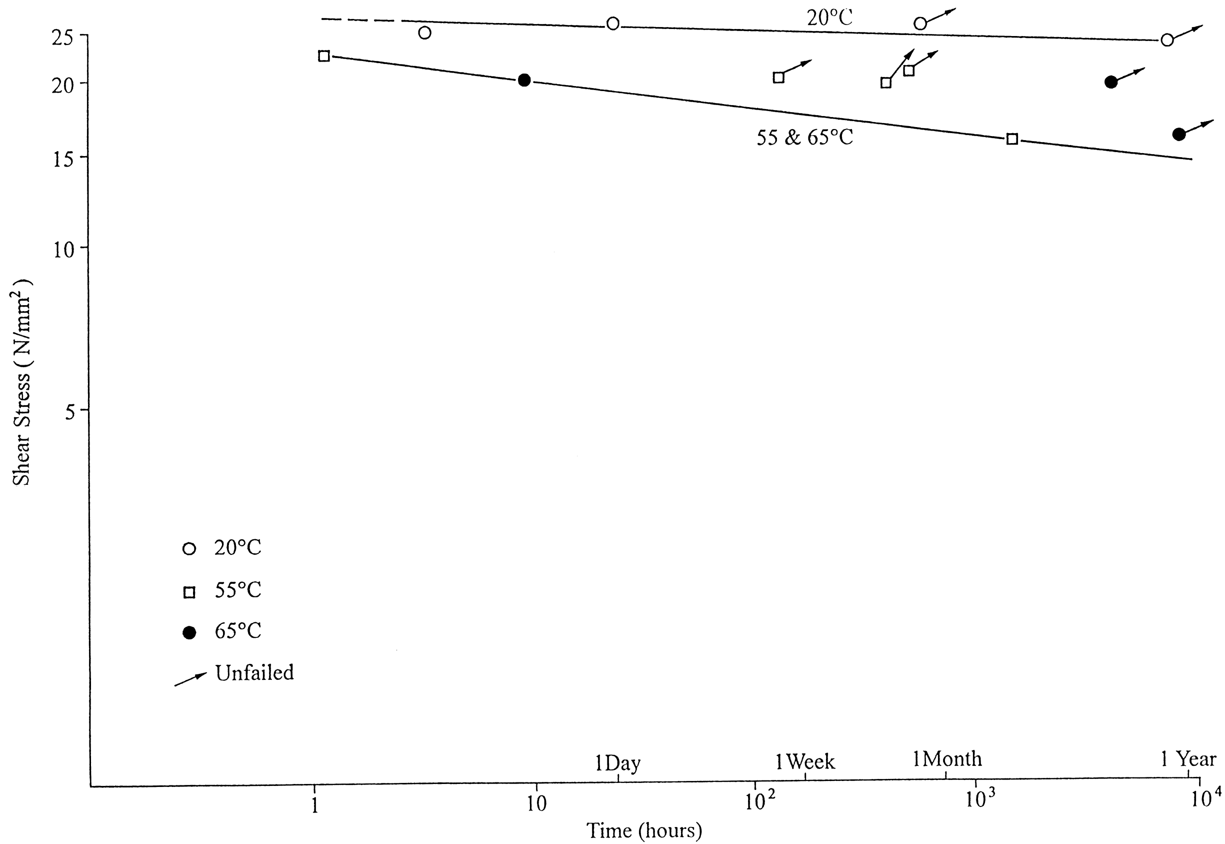

Figure 7.5 Shear stress versus time to failure for single-part hot cure epoxy resin adhesive (Mays, 1990).

Table 7.1 Relationship between normalised stress and time to failure

Adhesive	Temperature (°C)	Normalised stress at time to failure of		
		1 year	30 years	120 years
Cold cure	20	0.69	0.63	0.6
Cold cure	40	0.46	0.38	0.36
Cold cure	55	0.18	0.11	0.09
Hot cure	20	0.67	0.63	0.61
Hot cure	55 & 65	0.41	0.34	0.32

structure. Also, due to the deleterious effects of temperature, adhesive bonded joints should not be exposed to service temperatures in excess of the glass transition temperature for that adhesive.

7.6 Time-dependent characteristics of plated beams using steel plates

Most studies on the time-dependent behaviour of reinforced concrete beams strengthened with externally bonded steel plates have combined the effects of weathering and sustained loading. However, the durability of adhesive bonds is discussed in Chapter 6, consequently this section will focus on the effects of sustained load.

Since 1972, research on plate bonding has been conducted at the Swiss Federal Laboratories for Materials Testing and Research (EMPA) at Dubendorf. The early work (Ladner and Weder, 1981) included studies of the long term performance of steel plated beams under sustained load.

Sixty-six reinforced concrete beams (150 mm × 250 mm × 2400 mm) were strengthened with external steel plates (3 mm × 100 mm × 1950 mm) and prior to strengthening the beams were loaded to produce flexural cracking. Forty-one beams were loaded with a constant load of value 28% of the failure load, some of the beams also being subjected to weathering in outdoor test areas. The remaining beams were used as controls, six of which were statically tested before the onset of the creep programme. There was little difference in failure load or failure deflection between these beams and beams which had been subjected to sustained loading for one year prior to static testing. Although the majority of creep will have taken place during this period, the length of time for this creep investigation was short.

The increase in central deflection with time was also monitored with an average creep of 1.3 mm over 800 days, the initial elastic deformation upon application of the load averaging 2.3 mm. Creep measurements over 10 years on another plated reinforced concrete beam produced the same trend in the results expected of a conventional reinforced concrete beam.

Concerns have been expressed by the Institution of Civil, Municipal and Structural Engineers Standing Committee on Structural Safety (Standing Committee on Structural Safety, 1983) about the long term performance of resins. Consequently, the Transport and Road Research Laboratory, UK (TRRL) undertook a programme of testing to determine the long term properties of steel plated reinforced concrete beams (Calder, 1990). Creep tests on plated unreinforced concrete beams showed no statistically significant differences in failure loads between loaded specimens and unloaded controls following 10 years' exposure. This would indicate that if creep of the adhesive does take place the failure of the beam system is not significantly reduced.

Reinforced concrete beams were also used in the TRRL work, half of the beams being precracked prior to plating. All the beams were then held under sustained load sufficient to produce and hold open flexural cracks. Control beams were kept in a laboratory environment (20 °C, 65% rh) whilst the remaining specimens were exposed at three outdoor exposure sites, chosen to represent conditions encountered throughout the UK. Two different resins were used, resin I (Ciba Geigy XD800) and resin II (Colebrand CXL83), both of which were structural epoxy adhesives. Beams plated with resin I showed no difference in failure load following eight years' sustained load when compared to control beams tested soon after plating. Resin II, however, was more susceptible to environmental attack; those beams which survived eight years' exposure failed prematurely due to plate debonding. The tests showed that moisture penetration leading to steel corrosion was a far more serious threat to long term performance than creep under sustained load.

Another UK study of the long term behaviour of plated RC beams was conducted in an area of high industrial pollution near Sheffield (Swamy *et al.*, 1995). Eight plated beams were maintained under constant load and the failure loads after 11–12 years' exposure were found to be in excess of the 28 day controls. The increased concrete maturity with time obviously had an influence on these results. There was no significant difference in failure loads in 11–12 year old beams between those under sustained load and those left unloaded.

In Spain, creep tests with temperatures ranging from −11–+42 °C were conducted using three different adhesive types and varying adhesive layer thickness (Canovas, 1990). Satisfactory behaviour for three years was reported for plated beams with minimum adhesive thicknesses (up to 1 mm). As the adhesive layer thickness increased, however, so did creep, as evidenced by increased central deflections. With one of the adhesives this led to failure of the beam after 420 days. This adhesive was deemed to be unsuitable for future use in plate bonding.

It is generally accepted that the long term performance under load of steel plated reinforced concrete beams is satisfactory provided that a suitable adhesive and adhesive layer thickness are utilised.

7.7 Time-dependent characteristics of FRP component materials and FRP composites

The matrix material of FRP composites is polymeric, and hence these materials are subject to viscoelastic behaviour. The creep response of the composite material is a combination of elastic fibres which do not creep and a flexible viscoelastic polymer matrix which does. The creep of composite materials is highly dependent upon:

- the type of polymer and its stress history
- the direction of alignment, type and volume fraction of fibre reinforcement
- the nature of the applied loading
- the temperature and moisture conditions to which the element is exposed.

A number of investigators have reported on the time-dependent nature of polymer composites in general, for example Wu and Ruhmaan (1975), Crossman and Flaggs (1978) and Schaffer and Adams (1981). Creep of these materials can be substantial, particularly when subjected to high loads, elevated temperatures or moisture absorption. Scott *et al.* (1995) present a review of literature concerned with the creep behaviour of FRP composites, with particular reference to the development of accelerated test methods for predicting long term performance of FRP for structural highway applications.

7.7.1 Mechanisms of FRP creep

There are two distinct mechanisms by which FRP materials can creep, the occurrence of which depends on the state of the microstructure of the material. In predicting the creep response of a composite material, it is generally assumed that the material microstructure remains intact, with the composite structure of the fibre and matrix deforming as one unit, the resin phase transmitting stress between the fibres. Consequently, the creep of the composite will be due solely to the viscoelastic flow of the polymer matrix. This assumption may be valid at elevated temperatures near to the T_g of the polymer, but in the general working temperature range for construction materials the primary mechanism of creep is more complex. It is likely that the majority of the observed creep, stress rupture and stress relaxation at ambient temperatures is due to the time-dependent growth of fibre–matrix debonds and resin cracks within the material microstructure (Johnson, 1979).

It is generally accepted that microdamage in the form of cracks is initiated when a component of force acts perpendicular to the fibre axis. Cracks may also be initiated due to differences in the thermal expansion coefficients of the constituent materials (Sillwood, 1982). The high density of geometric discontinuities in fibrous composites, such as fibre ends and voids, and the brittle nature of the resin systems used, also aids the initiation of microdamage. This damage growth mechanism can result in non-linear, stress-dependent viscoelastic behaviour over a wide range of applied stress, even when the material is lightly loaded (Howard and Hollaway, 1987).

The occurrence of creep under sustained static loading is therefore a combination of bulk material strain and microflaw initiation, both of these mechanisms being time dependent due to the viscoelastic nature of the polymer matrix. However, it is difficult to distinguish from the macroscopic behaviour of a composite material under load, the relative combination of the bulk viscoelastic deformation of the polymer and that of the microflaw growth. A number of investigators, for example Aboudi (1990), have reported sudden jumps in the creep strain which may correspond to the onset and growth of damage as opposed to the smoother process of the bulk creep strain of the resin alone.

7.7.2 Creep characteristics of FRP composites

The synthesis of data reported on the creep characteristics of polymer composite materials is difficult due to the diversity of the resin and fibre types, fibre orientations and combinations, testing environments and fabrication processes. The extent to which the stiffness and strength characteristics of a composite material are affected by the time-dependent mechanical properties of the matrix can generally be related to the proportion of load being carried by the polymer. Significant time dependence is demonstrated when the applied load is shared between the fibre and the matrix, or is carried mainly by the polymer matrix. The effect is most pronounced in low fibre volume fraction materials and in tests normal to the predominant fibre orientation. Provided that the contribution of the matrix phase to the overall stiffness of the material is small, the effect of the time-dependent changes in the properties of the matrix material on the overall composite is also likely to be small. Since time dependence is reduced as the proportion of the applied load carried by the reinforcing fibres increases, it follows that virtually time-independent behaviour can be achieved through the use of a high volume fraction of high stiffness fibres placed unidirectionally in the predominantly loaded direction. Research on the creep of unidirectionally reinforced laminates is reported by, amongst others, Dillard and Brinson (1983), Hiel *et al*. (1983), Tuttle and Brinson (1986), Ponsot *et al*. (1989) and Aboudi (1990).

In the context of external plate bonding, the plate performs a simple task, acting as a uniaxially stressed tensile component as the composite member deflects. The direction in which the strengthening material will be loaded is therefore known. If a fibre reinforced composite is to be utilised, it can be manufactured with all of its strength in this direction as a unidirectional laminate of continuous fibres. The greater the volume of fibres which can be included in the composite, the greater will be its stiffness and strength, and hence the greater its efficiency. It follows from the discussion above that such a material should display minimal time dependence, especially if high stiffness fibres are incorporated.

7.8 Time-dependent characteristics of plated beams using polymer composite plates

Bonded FRP systems can produce a dual viscoelastic problem, the combination of a viscoelastic adherend and a viscoelastic adhesive. In addition, a third viscoelastic component is introduced when one of the adherends employed is concrete subjected to shear in the cover concrete and compressive stresses above the neutral axis. Although many investigations have been carried out to evaluate the time dependency of concrete, adhesives and FRP materials individually, only one recent study by Plevris and Triantafillou (1994) could be found in the literature in which these materials were employed in combination. In this study, an analytical procedure to predict the time-dependent behaviour of RC beams strengthened with FRP laminates was presented. These procedures, implemented by computer, were used in a parametric study to assess the effects of the type and area fraction of FRP material on the long term response of internal stresses and strains and overall curvature of the member. An age-adjusted effective modulus of elasticity was used to represent the behaviour of the concrete in compression, while Findley's model (Findley, 1960) was utilised for the composite material.

The addition of CFRP plates reduced the rate of compressive creep in the concrete of the plated beams compared with the unplated ones; this decrease in creep rate was greater for plates with larger cross-sectional area. Furthermore, overall curvature and tensile stresses in the internal steel were reduced by the addition of external FRP plates; the response of the steel was shown to be time independent. The response of beams strengthened with either glass fibre reinforced plastic (GFRP) or carbon fibre reinforced plastic (CFRP) were shown to be similar.

7.9 Creep tests conducted during the ROBUST project

7.9.1 Creep testing of CFRP

In the ROBUST programme of work, a unidirectional CFRP prepreg composite manufactured by 'Cytec' ('CYCOM 919 HF – 42%-HS-135-460 (P/N 02098)) containing 58% weight fraction of Toray 'T300' fibres and 42% epoxy resin (the properties of which have been given in Chapter 3) was held under sustained tensile loads of various magnitudes at 22 °C, 40 °C and 60 °C and a relative humidity of 50%; the material was the same as that used in a number of the beam tests from which the static behaviour of plated beams was determined (Garden *et al.*, 1996; Garden *et al.*, 1997; Garden and Hollaway, 1997) as discussed in Chapter 4. The higher temperatures were chosen to accelerate the potential creep process arising from the viscoelastic behaviour of the epoxy resin and to simulate possible service temperatures. The variation of longitudinal strain with time for the CFRP is shown in Fig. 7.6 for each temperature under the highest applied tension which was nominally 60% of the composite tensile strength. The slight initial fall in strain at 40 °C and 60 °C is attributed to the slippage of the CFRP from beneath the aluminium end tabs of the coupon specimens, an effect observed due to the softening of the adhesive at these elevated temperatures, continuing until a new equilibrium load was established. While this slippage was undesirable, the settlement of the strain at the new level reflects the absence of any creep of the CFRP under the new stresses; the applied load remained constant during the period of settled strain. The absence of any apparent creep is consistent with the previous literature which reports excellent CFRP creep resistance with high fibre volume ratios, even when the material is loaded to a high proportion of its tensile strength.

7.9.2 Creep testing of plated beams using CFRP plates

In the ROBUST programme of sustained load tests, 1.0 m long reinforced concrete beams of 100 mm square cross-section were externally strengthened with the CFRP plates whose creep behaviour is represented in Fig. 7.6, so it was known that any overall creep of the plated beams under load would not have been contributed to by the composite material itself. A typical deflection response of the 1.0 m beams, loaded such that the top extreme surface of the concrete and the internal tensile rebars were under the same stress magnitude in the unplated and plated cases, is shown in Fig. 7.7. These identical stress magnitudes were applied to the two beam systems to ensure that the presence of the plate and the adhesive would be the only

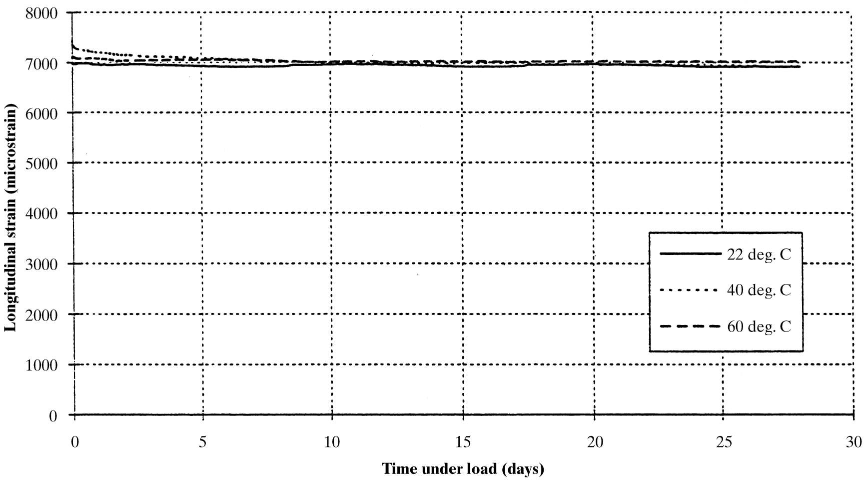

Figure 7.6 Variation of tensile strain under sustained load for a typical CFRP composite (Garden, 1997).

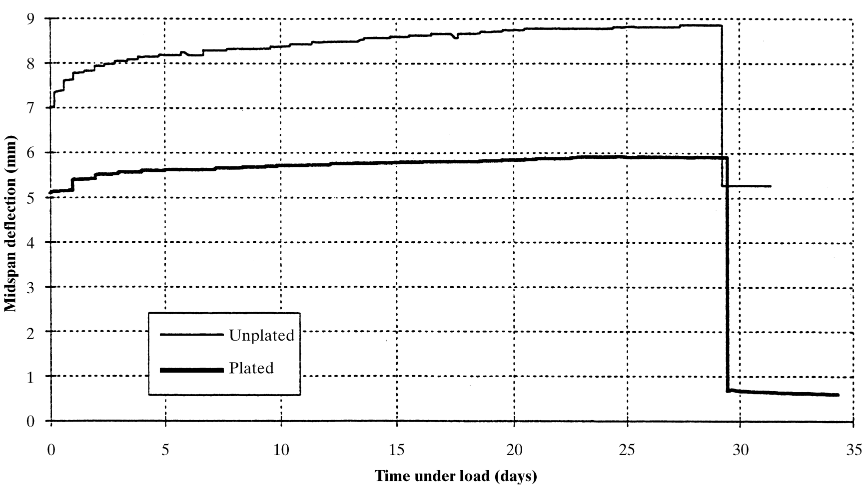

Figure 7.7 Deflection responses of 1.0 m CFRP plated beams under sustained load (Garden, 1997).

cause of any difference in the structural behaviour under sustained load. This state of stress was achieved by loading the plated beam to its rebar yield load, and the unplated beam to that load beyond yield which produced the same maximum compressive stress in the concrete. The rebars were plastic beyond their yield strain so it was known that the rebar stresses were equal in both cases; sustained tensile tests of the rebars confirmed that the

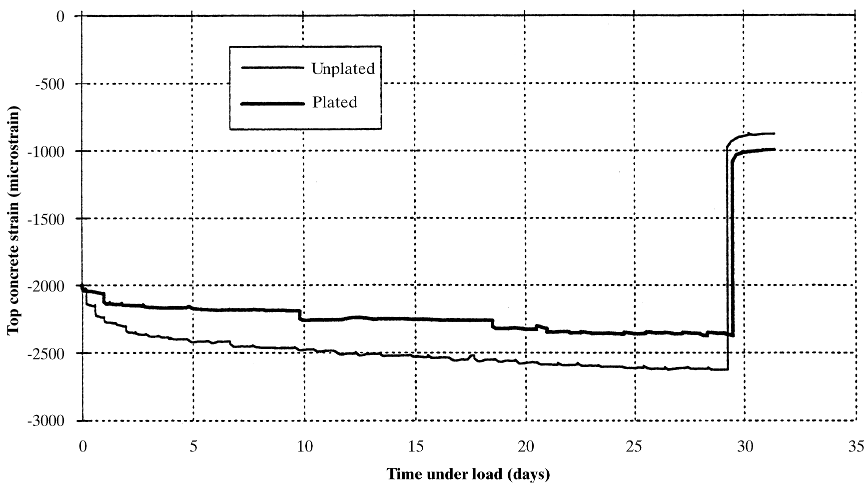

Figure 7.8 Concrete compressive strain responses of 1.0 m CFRP plated beams under sustained load (Garden, 1997).

internal reinforcement did not contribute to the creep of the beams, the steel strain having remained constant.

The vertical steps in Fig. 7.7 mark the increases in elastic deflection corresponding to the restoration of the load which was necessary twice daily due to creep of the beams. As this load restoration applied an elastic strain to the beam, the total creep deflection at any time '*t*' was calculated as the total increase in deflection under static load from time zero to time '*t*', minus the sum of these vertical elastic steps. The creep deflection value of the unplated beam was 0.33 mm and was relatively large, representing 4.7% of the original 7.00 mm elastic deflection, whilst the creep deflection of the plated beam was 0.14 mm and was only 2.7% of the originally smaller deflection of 5.10 mm. The creep of the unplated beam was 3.7% of the total deflection at the end of the creep period, higher than the 2.4% in the plated case, indicating a reduction in the time-dependent deformation due to the bonded plate.

Figure 7.8 shows the relationship between the midspan concrete compressive strains at the top surfaces of the 1.0 m beams and the time under continuous loading. The unplated concrete beam crept at a higher rate and had a magnitude of creep greater than the plated concrete beam, reflecting the deflection behaviour. The ratio of unplated to plated concrete beam creep strain was 2.18, similar to the deflection ratio of 2.36, suggesting the time-dependent deflection of the plated beam was predominantly flexural and not influenced by any significant vertical shear creep. At the start of the continuous loading period, significant shear cracks were apparent in the shear span.

Further ROBUST creep tests were undertaken at the serviceability load of these beams. The graphs of deflection against time and creep strain of the concrete against time were of similar shape to those above. The creep deflection was approximately one-third of the total deflection for both the unplated and plated beams. Furthermore, the compressive concrete creep was approximately one-third of the total compressive strain under the serviceability load. From strain gauge data taken at various positions along the bonded composite plate, whilst the beam was under a sustained load, an increase in strain was measured which was also one-third of the total tensile strain of the composite plate. As the stress/strain relationship for CFRP composite material is linear to failure and the material does not creep, the increase in strain of the composite plate was attributed to the deflection of the beams due to creep of the concrete. The influence of the adhesive layer on the global creep response was uncertain but, since the concrete and the composite plate both experienced the same proportional long term strain increments with respect to the total strain, it may be said that the adhesive experienced these same proportional strain increments (namely, it experienced no creep at all during sustained load). Consequently, the viscoelastic property of the adhesive appears to present no particular problems when the plated bonded beam is under a sustained load at ambient temperature. The fact that the unplated and plated beams experienced similar proportional creep suggests the sustained load behaviour of beams is governed by the compressive creep of the concrete and, therefore, not by any time-dependent effects of the adhesive.

Part B Fatigue behaviour

7.10 Introduction

Dynamic fatigue is the failure of a material or assembled component due to the application of a large number of stress/strain changes, the magnitude of which is lower than those at the static failure load. Fatigue occurs due to irreversible processes that take place when a cyclic load is applied to a material, as will be reviewed in the following sections.

The process of fatigue involves progressive and irreversible deterioration and may lead to excessive deformations and crack widths, debonding of internal reinforcement in concrete members and rupture of the reinforcement and/or the concrete. For post-tensioned prestressed beams, fatigue failure may occur at the anchorages. However, for a post-tensioned prestressed concrete beam for which precompression is such that the section remains uncracked under service load cycling, fatigue does not generally cause significant problems. An important property of materials subjected to dynamic fatigue loading is the ratio of the stress level at which irreversible damage occurs to the stress for complete fracture of the material. The stress at which fatigue failure occurs after a given number of loading cycles is referred to as the 'fatigue strength'.

7.11 Fatigue of unplated beams

A typical reinforced concrete bridge deck may experience up to 7×10^8 stress cycles during the course of its 120 year lifespan, thus it is important to be able to assess the fatigue performance of such structures. Concrete, steel and reinforced concrete members are all susceptible to fatigue loading. During the fatigue of concrete, irreversible deformation in the form of cracking occurs; this is extensive and causes higher strains at failure than would occur under a static load. Cyclic loading at low levels (below the fatigue limit) improves the subsequent fatigue performance (when loaded above the fatigue limit) of concrete and also the static strength (an increase of 5–15%). The initial low stress cycling is thought to cause a densification of the concrete.

As concrete ages its fatigue strength increases at a similar rate to its static strength. Consequently, for a given number of cycles, failure due to fatigue occurs at the same percentage of ultimate strength. Failure at the bond between the cement paste and the aggregate is thought to be the predominant failure mode in fatigue. Fatigue specimens tend to exhibit less broken aggregate particles than comparable static test specimens.

However, it is the behaviour of the reinforcing steel that tends to dominate the behaviour of conventionally reinforced concrete structures. The Transport and Road Research Laboratory (TRRL) undertook a programme of research (Moss, 1982) into the fatigue behaviour of the rebars, both in air and within reinforced concrete beams.

Axial tests of rebars can conveniently be undertaken utilising small coupon specimens in tension; the test frequency used can be high and thus data can be rapidly obtained. However, the test conditions are not representative of those rebars in reinforced concrete, consequently, tests on reinforced concrete beams are necessary. With these beams the testing frequency needs to be kept relatively low (in the TRRL case 3 Hz) to avoid hysteresis effects. Such tests are time consuming and expensive.

TRRL used concrete beams, 3400 mm long, 120 mm wide and 220 mm deep, spanning 3000 mm and loaded in four point bending at the third points. The beams utilised C55 concrete and were reinforced in the tensile zone with a single T16 bar.

In the analysis of the results the following relationship was derived:

$$N\sigma_r^m = K$$

where σ_r = stress range, N = cycles to failure, m = inverse slope of $\log \sigma_r - \log N$ curve = 8.7, $K = K_0$ gives the mean line of the relationship = 0.11×10^{29} or $K = K_2$ gives the mean minus 2 standard deviation line = 0.59×10^{27}.

Fatigue failure of the reinforcement was found to occur at the position of a crack in the concrete and the crack initiation site in the steel was associated with the rib pattern on the surface of the bar.

The fatigue of a reinforcing bar in direct tension was found to be different from that of a bar in the tension zone of a reinforced concrete beam. A bar embedded in a concrete beam and loaded in flexure is more highly strained in the section furthest from the neutral axis of the beam and, in addition, as the concrete cracks in the tension zone of the beam, high strain values and hence high stresses in the bar spanning the crack will be developed. However, as the maximum stress occurs at only one location there is less chance of this coinciding with a defect in the bar. This occurrence gives an enhanced fatigue life to a rebar embedded in a concrete beam compared with a bar cycled in air in direct tension.

As with other engineering properties the geometric scale is an important consideration in testing. Experimental data generated from laboratory tests on small steel specimens differ from those derived from 'life size' specimens or the simulated testing of full scale structures. The larger the specimen, the lower the fatigue limit. Thus small laboratory specimens can give artificially high values of safe working stress levels.

Earlier work at RMCS (Emberson and Mays, 1996) examined the fatigue behaviour of repaired concrete beams. Twelve beams 2540 mm long by 230 mm deep by 150 mm wide were repaired either on the tension or compression face. Each beam was subjected to a cyclic load, between 3 and 30 kN at a frequency of 1 Hz, in four point bending. Endurance of the repaired beams ranged from 70–280% of the control beams (average of 590 000 cycles to failure). In each case the failure was caused by tensile fatigue fracture of the main reinforcement. This corresponds to the overall conclusion that the fatigue behaviour of reinforced concrete beams is governed by the steel reinforcement.

7.11.1 Environmental effect on fatigue behaviour

The fatigue behaviour of reinforced concrete under environmental live loading from wind and waves was an important aspect in a research programme called 'Concrete in the Oceans (CiO)' undertaken in 1989 for the Department of Energy (Leeming, 1989). A comprehensive review of existing fatigue data was undertaken and a computer database of published results was compiled. Of specific interest to the programme were crack blocking and corrosion. Crack blocking occurs in concrete beams tested cyclically in seawater where deposits build up in the cracks giving rise to a reduction in the deflection range and hence an increased fatigue life. Corrosion can cause both initiation of the fatigue crack and blunting of the fatigue crack by intense local corrosion. The effect of reinforcing bar size was also investigated; however, there was no clear trend from tests on 16 mm, 32 mm and 40 mm bars in concrete beams. Following a programme of fatigue tests on reinforced concrete beams a design curve was derived.

7.12 Fatigue of adhesives

Much of the research into the fatigue performance of adhesive bonds has been carried out with aerospace applications in mind. Pioneering work was carried out in Germany in the 1960s by Matting and Draugelates (1968) when a vast amount of data was collected using single lap joints of sheet metal alloys bonded with phenolic and epoxy adhesives. The results were presented in the same way as used for metal fatigue, that is in the form of *S–N* curves of stress against number of cycles to failure using logarithmic scales. With metals it is generally accepted that endurance is dependent on the stress range applied during cyclic loading, although with epoxy adhesives there is some evidence to suggest that crack growth rate may be dependent on the maximum stress intensity factor in the cycle.

This early work also provided evidence that the *S–N* curve is not very sensitive to frequency. However, as adhesives become more 'rubbery', for example as they pass through the transition temperature, T_g, the time to failure becomes more important thus implying different failure mechanisms; the mechanism before the T_g is reached is associated with crack growth, whilst the failure mechanism, for situations with values greater than the T_g, is a rate process. For both phenolic and epoxy adhesives, the endurance limit, defined as the maximum stress at which the survival time is indefinite, was found to be at about 15% of the static strength.

Other work (Marceau *et al.*, 1978) using aluminium/epoxy lap joints at a range of frequencies showed that lower frequencies are more severe in terms of the mean damage sustained per cycle. The difference in fracture modes observed at different frequencies supports the theory that two different failure mechanisms are operating, that is a fatigue separation at high frequencies compared with a creep–rupture separation at low frequencies. Elevated temperatures were also observed to shorten fatigue life but the effect of humid air was less significant.

In the UK, Wake *et al.* (1979) attempted to establish the reality of an endurance limit based on the argument that successive cycles of stress above the limit caused the growth of cracks from existing flaws. Their work suggests an endurance limit close to a peak loading corresponding to 35%

of static ultimate strength. A small study was also conducted to assess the relative importance of the crack initiation and propagation stages on subsequent strength. It was concluded that, if an endurance limit exists, load cycling below it may only serve to initiate cracks (not detected by subsequent static tests) rather than to propagate them. Failure may then be initiated should cycling above the endurance limit subsequently take place.

Much of this early research and its subsequent development relates to types of adhesive and substrates unlikely to be encountered in civil engineering applications. For this reason independent work was commissioned by the TRRL, now TRL (Mays, 1990). The test programme used steel-to-steel double lap joints manufactured with one of two cold cure epoxies typical of those used in structural engineering. These were subject to cyclic loading at a frequency of 25 Hz at load ranges to produce failure at endurances of up to 100 million cycles. Fatigue tests were undertaken on specimens cured and weathered under a range of climatic conditions and tested at temperatures ranging from $-25\,°C$ to $+55\,°C$.

The results are presented in Fig. 7.9 in terms of stress range rather than peak stress, to be consistent with accepted engineering practice for metals. The correlation obtained using this approach was quite acceptable, although the quoted values of stress range will be somewhat sensitive to geometrical configuration of the joint employed. Although the results lend credence to the existence of an endurance limit, its value has not been precisely determined.

Compared with *S–N* curves for metal fatigue, those for adhesive bonded joints are relatively flat, that is, there is a large improvement in endurance for a relatively small reduction in stress range. From an engineering point of view, this is advantageous since, in the absence of an endurance limit, the *S–N* curve may be extrapolated to determine the stress range necessary to establish the maximum conceivable number of repetitions of a constantly applied load.

For the cold cure adhesives utilised in this study, the lower bound to fatigue performance for service temperatures between $-25\,°C$ and $+45\,°C$ is represented by the relationship:

$$NS_r = K$$

where N = cycles to failure, S_r = stress range, $K = 2 \times 10^{22}$ for mean minus 2 standard deviation line.

Extrapolating to 7×10^8 cycles, representing the maximum conceivable number of significant load variations during the 120 year design life of a bridge, yields a limiting stress range of $4.0\,N\,mm^{-2}$. This figure approximates to the tensile shear strength of the concrete in applications involving reinforced concrete members with externally bonded reinforcement. However, typical adhesive shear design stresses at the serviceability state are unlikely

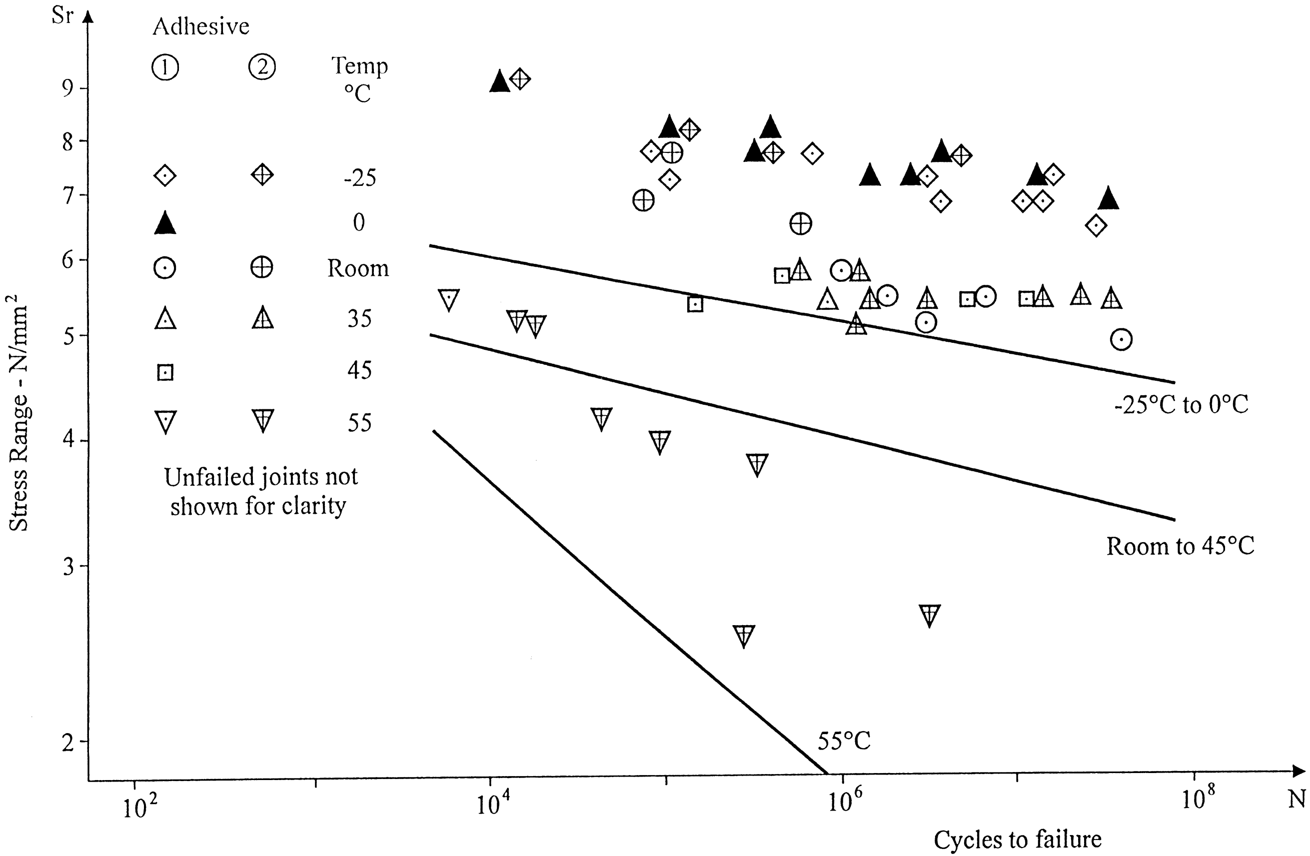

Figure 7.9 Lower bound to fatigue performance of cold cure epoxies (Mays, 1990).

to exceed 1.0–1.5 N mm^{-2} so that fatigue is unlikely to be a critical design consideration at these temperatures.

The brittle failure modes of joints made with these cold cure adhesives for service temperatures up to 45 °C confirm the prediction that a mechanism dependent on crack growth is predominant. However, at 55 °C, a temperature above the T_g, the more ductile failure mode suggests a significant change in material response which is reflected in a dramatic reduction in fatigue performance. Thus, the use of cold cure adhesives such as these is not recommended in situations where the service temperature exceeds 45 °C or T_g + 10 °C, whichever is the lower.

7.13 Fatigue of FRP materials

One of the significant advantages of composite materials over metals is their superior fatigue performance. Composites have traditionally found applications in the aerospace industry, in which their uses have reflected their superior fatigue resistance. Steel usually fails by the sudden propagation of a single crack at the end of the fatigue life, but polymeric composites experience progressive fatigue degradation due to failure of the fibres and the matrix and of the matrix/fibre interfaces (Ellyin and Kujawski, 1992).

Unlike isotropic materials, the cyclic degradation of anisotropic materials, in the form of microcrack growth, delamination and subsequent mixed mode crack propagation, may be well established before complete fracture occurs. A considerable degree of experimental scatter is found in the literature, due partly to uncertainty about the moment in time at which a composite can be considered to have failed (Dharan, 1975). The progressive degradation of composites renders their change in modulus a more important design criterion than for a homogeneous material, but the fatigue performance of composites is likely to be more than adequate due to the relatively slight static constraints. The scatter in static strengths of polymeric composite materials leads to low design stresses being adopted in practice; these low working stresses may result in the elimination of fatigue as a design criterion.

The slopes of the *S–N* curves (i.e. the measure of relative fatigue performance) of composites are determined principally by the strain in the matrix, since the fatigue limit of the matrix is lower than that of the fibres (Curtis, 1989). The fatigue behaviour of composites containing various resins is not very different despite great chemical differences, although the fatigue properties of epoxies are slightly superior due to their greater strength, higher resin/fibre interface strengths, lower curing shrinkage which implies lower residual stresses before loading, and high strain to failure which prevents fibre exposure and delays stress corrosion in GFRPs (Dew-Hughes and Way, 1973).

The four basic fatigue failure mechanisms of polymeric composites are matrix cracking, delamination, fibre fracture and interface debonding (Hollaway, 1993). The type and degree of these mechanisms vary depending on material property, laminate stacking sequence and the type of fatigue. Matrix cracking in the off-axis plies of multidirectional laminates is usually the first damage mechanism because the lack of fibres oriented in the direction of the applied load causes more load to be distributed to the matrix. The slope of the tensile *S–N* curve increases with reducing static strength as the proportion of off-axis fibres increases; this arises due to the increasing likelihood of matrix damage with a reducing proportion of fibres oriented in the direction of the applied load. The final fracture of multidirectional composites occurs when the 0° fibres fail. The fibres of unidirectional composites carry the majority of the applied load and these composites have excellent fatigue resistance as a result (Curtis, 1989). Delamination is attributed to the presence of interlaminar stresses in the vicinity of a free edge under in-plane loading (Hollaway, 1993) giving rise to damage propagation from the free edge inwards. Renton and Vinson (1975) found that the laminate stacking sequence will affect the fatigue failure mode of bonded composite adherends; unidirectional composites caused cohesive failure in the adhesive, while an alternating ±45° sequence, with a 45° layer against the adhesive, resulted in composite delamination.

The critical stage in the fatigue failure of composites is the initial resin/fibre debonding. Dew-Hughes and Way (1973) cited results which showed that fibre/matrix debonding in GFRPs may start at 30% of the static tensile strength and at decreasing stresses under cyclic loading, while failures due to the cyclic loading of CFRPs may be delayed until 70% of the static strength. The existence of debonding at a low proportion of the static strength implies damage in the first cycle under dynamic fatigue conditions, as confirmed experimentally by Kim and Ebert (1978) for GFRP composites.

Statistically distributed flaws in the fibres of a unidirectional CFRP specimen may cause fibres to break before resin damage appears; a crack will extend into the resin matrix with further increasing load and its path will depend on the strength of the fibre/matrix bond of the fibres towards which the crack travels. A strong fibre/matrix bond causes cracks to extend into the matrix, while low interface strengths result in interfacial debond and fibre pull-out (Hollaway, 1993). Also, the elastic modulus of carbon fibres may be as much as six times greater than the modulus of glass fibres, so the fracture strains of carbon fibres are relatively low, certainly below the strains at which common resin matrix materials begin to craze (Beaumont and Harris, 1971). The fracture strains of glass fibres, however, are greater than the strains to first matrix damage.

The rate of matrix and interfacial damage propagation is affected by the bulk strain in the resin matrix. The matrix strain is, in turn, dependent on the modulus of elasticity and the volume fraction of the fibres. High modulus carbon fibres result in relatively low strains and, therefore, lower rates of damage propagation than in composites containing lower modulus glass fibres (Curtis, 1989). In regions of a composite with closely spaced fibres (i.e. if the fibre distribution throughout the cross-section is not uniform), resin cracks may propagate by the coalescence of individual fibre/matrix debonds (Hull, 1992). When the fibres are widely spaced, the growth of a crack between fibres depends on the fatigue crack growth resistance of the matrix.

Little improvement in fatigue behaviour is generated by the use of reinforcing fibres, of a particular type, with a higher static strength in a given resin matrix, since the fatigue failure is usually governed by damage in the matrix and at the fibre/matrix interfaces so the improved fibre strain will not be used to advantage (Owen, 1974).

The effect of environmental exposure on the fatigue behaviour of composites depends on the sensitivity of the laminate to the properties of the matrix with respect to environmental conditions (Curtis, 1989). This is because it is usually the matrix or fibre/matrix interface that is affected by moisture absorption, as noted in Chapter 6. A high proportion of the total moisture uptake reaches the fibre/matrix interfaces, as seen in the small weight gains of unreinforced resins exposed to steam for 200 h, compared with the weight gains of CFRPs under the same exposure (Beaumont and Harris, 1971). GFRPs are affected by an increase in moisture content due in the main to the stress corrosion of the fibres, so the influence is smaller in composites under lower stress (Dew-Hughes and Way, 1973).

The high fibre/matrix interface bond strengths of CFRPs produce little overall fatigue sensitivity to moisture at room temperature (Owen, 1974). As for static strengths, the fatigue strengths of composites are degraded most by the combined influence of elevated temperature and relative humidity. In addition to providing relatively high static and fatigue strengths, a unidirectional fibre orientation results in the best environmental durability under cyclic loading, since the fibre/matrix interfaces are not loaded in tension normal to the fibres (Beaumont and Harris, 1971; Demers, 1997a; Demers, 1997b).

7.14 Fatigue of plated beams using steel plates

The early EMPA work (Ladner and Weder, 1981) included fatigue tests on plated reinforced concrete beams. Initial tests on small rectangular cross-section beams ($150 \times 250 \times 2400$ mm) were followed by tests on a 'T' beam ($500 \times 900 \times 6700$ mm).

For the rectangular beams, a small static load was applied in order to initiate cracking prior to plating. A frequency of 4.3 Hz was selected, with a maximum of 10^7 load cycles, the load levels being varied for each of the eight beams tested. Steel stress ranges in the plate at midspan varied from $120 \pm 20\,\mathrm{N\,mm^{-2}}$ to $150 \pm 130\,\mathrm{N\,mm^{-2}}$. Three beams survived 10^7 cycles, in two beams fracture of the steel plate occurred and the remaining three beams failed due to fatigue of the bond. In these cases the steel plate debonded from the adhesive at crack locations in the concrete and this debond crack then propagated towards the end of the plate.

The 'T' beam was strengthened with both a flexural plate on the soffit and shear plates on the side of the beam. The beams were subjected to four phases of 2×10^6 cycles, each subsequent phase consisting of an increased load range. One of the internal rebars failed near the end of the second fatigue phase. Testing continued with the span reduced from 6000 mm to 4900 mm. Following cyclic loading, the beam was tested statically and this showed that the beam had survived the fatigue phases without any damage (excepting the rebar fracture mentioned above).

In 1985, under-reinforced concrete beams (100 mm $\times$ 100 mm $\times$ 1000 mm) were strengthened using three different adhesives to bond on steel plates and subsequently tested in fatigue (Mays, 1985). Testing was carried out for up to 2×10^7 cycles at a frequency of 6 Hz. There was no discernible difference in performance for the three different adhesives.

Fatigue failures occurred by progressive horizontal cracking within the concrete at the level of the internal reinforcement and eventual plate debonding at the concrete/adhesive interface. The failure mode was dominated by the stress concentration occurring in the adhesive close to the plate ends. Within the adjacent concrete, compatible stresses led to cracking and hence a shift in the position of the stress concentration. This progressive concrete cracking and plate debonding eventually led to failure. Beams that were precracked prior to plating failed due to fracture of the soffit plate adjacent to one of the flexural cracks.

When compared with an unplated control specimen, an improvement in fatigue performance was evident. At a load range of 18.9 kN the control beam failed at 257 000 cycles compared with 6.6×10^6 cycles for a plated beam at the same load range.

Overall, no relevant fatigue damage occurred with the tensile stress (from the measured plate strain) in the soffit plate below $150\,\mathrm{N\,mm^{-2}}$. It was concluded that as the peak stress in the external plate reinforcement rarely exceeds $100\,\mathrm{N\,mm^{-2}}$ under typical design loading then fatigue is unlikely to cause concern.

Further fatigue testing of strengthened reinforced concrete beams was undertaken at the EMPA Laboratories in Switzerland (Holtgreve, 1986).

Reinforced concrete ‘T’ beams, 600 mm deep and spanning 7000 mm, were initially loaded to induce cracking. Steel plates were then bonded to the soffits of the beams whilst they were under dead load. Following curing of the adhesive, the beams were cycled at a rate of 4 Hz from 117–254 kN. After 2×10^6 cycles the cyclic loading was stopped. There was no evidence of any bond deterioration between the steel and the concrete and the beams were then subjected to a static ultimate load test. Typical failure loads of 436 kN were achieved, with failure occurring in the outer tensile zone of the concrete, no failure of the adhesive being evident. Maximum strains of 0.2% were recorded in the steel plate.

Welded plates are not recommended for use in steel plate bonding and their susceptibility to fatigue loading has been demonstrated (Canovas, 1990). Rectangular section reinforced concrete beams 200 mm deep, 250 mm wide and spanning 3000 mm were strengthened with adhesively bonded steel plates some of which were joined by welding. The beams were tested at 8.3 Hz for 2×10^6 cycles between 50% and 75% of the ultimate bending moment and a further 2×10^6 cycles between 60% and 80%. The results showed that some adhesives are more susceptible to fatigue than others and that welded steel plates are very susceptible to fatigue.

The mode of failure arising from dynamic loading has been carefully studied (Hankers, 1990). Dynamic tests were carried out on plated rectangular reinforced concrete beams, with a cross-section of 260 mm $\times$ 150 mm, spanning 1800 mm. Measurements of load, deflection, plate strains, rebar strains and visual inspection were performed as the beams were tested at a frequency of 4 Hz.

At the high load levels, failure tended to be by debonding. Starting from bending cracks near the central load application point, the crack (between the steel plate and the concrete) proceeded towards the ends of the unanchored plate and this led to sudden failure. At the lower load levels the unfailed beams were tested statically, producing higher load capacities than the statically tested control beams. Possible vibration-activated adhesive polymerisation was suggested.

Rectangular cross-section beams (300 mm $\times$ 200 mm $\times$ 3000 mm) were again used in the most recently reported study on the dynamic behaviour of reinforced concrete beams strengthened with externally bonded steel plates (Taljsten, 1994). Two beams were tested dynamically in four point bending, with a load of 75 ± 55 kN and at a frequency of 3 Hz. The results of this and the previous research suggest that plated beams can behave better than unplated beams when subjected to cyclic loading, possibly because the bonded plates delay the initial cracking and reduce crack widths.

7.15 Fatigue of short span plated beams using FRP plates

As with the long term sustained load testing of plated beams, the fatigue behaviour of plated concrete members has received much less attention than the testing of more convenient small scale lap shear specimens.

Fatigue tests of beams strengthened by bonded CFRP plates were undertaken at EMPA (Kaiser, 1989; Deuring, 1993) and described in English in various publications (Meier and Kaiser, 1990; Meier and Kaiser, 1991; Meier *et al.*, 1993a; Meier *et al.*, 1993b). For the T-beams at a loading frequency of 4 Hz, the first fatigue failure occurred in one of the two internal bars at 4.8×10^5 cycles, the second bar broke at 5.6×10^5 cycles, the first bar broke at another location after 6.1×10^5 cycles and the second bar broke again at 7.2×10^5 cycles, compared with the first external damage of the CFRP plate at 7.5×10^5 cycles and complete failure of the plate at 8.05×10^5 cycles, indicating that the plate was able to sustain significant further fatigue loading after failure of the internal reinforcement.

Fatigue tests by Shijie and Ruixian (1993) showed that the fatigue lives of GFRP plated beams were three times longer than the lives of comparable unstrengthened beams. The fatigue strengths were increased by between 15% and 30% and the midspan deflections reduced by 40%; the maximum crack widths and crack propagation were also reduced. The postcyclic static strength and stiffness diminished with increasing numbers of cycles but by a smaller magnitude than in unstrengthened beams.

Chajes *et al.* (1995a) used Sikadur 32 adhesive to bond aramid, carbon and glass fibre composite plates to small scale reinforced concrete beams in which the plate had the same tensile strength as the single internal rebar, achieved by varying the number of plate layers of the three types. The beams, which were still under test at the time of writing, were to be subjected to 70% of their static ultimate capacity for 2.5×10^6 cycles and then their postcyclic static capacities were to be compared with the results of the beams tested by Chajes *et al.* (1995b).

In the ROBUST programme, five 1.0 m long beams were plated with a prepreg CFRP composite material, before being load tested cyclically at a frequency of 1 Hz, between applied loads which generated tensile stresses in the internal mild steel round rebars ranging between 2% and 90% (at the lower and upper loads, respectively) of the yield stress. The corresponding values for the unplated beams were 3% and 59%. All the beams were loaded to an upper level of approximately half of their ultimate capacity, representing 132% of the serviceability load in the plated cases. The tensile stress range in the rebars was important since reinforced concrete beams have typically been found to fail due to the fatigue fracture of the reinforcing steel under cyclic loading (Canovas, 1990; Bannister, 1969; Lovegrove, 1979; Tilly, 1979; Tilly and Moss, 1982). This was the case in the unplated 1.0 m beam tested in the ROBUST project in which two out of three rebars failed after about 1.5×10^6 cycles. Figure 7.10 shows the deflection amplitude against the number of cycles of load for one of the plated beams. Although the stress in the rebars was greater in plated beam compared with the unplated one, no rebar damage was recorded up to 11×10^6 cycles at which point the test was stopped. This result reflected the ability of the plate to limit the concrete crack widths.

In the fatigue tests undertaken on the 1 m long beams in the ROBUST project, it was shown conclusively that, under a shear span/beam depth ratio of between 3.4 and 4.0, it is necessary to incorporate a bolted plate end anchorage system. It was explained in Chapter 4 that under a static load it is necessary to install an anchorage system at the free end of the plates to gain the maximum strength and stiffness of the plated beam system. Furthermore, it was also concluded that the bolts in the anchor system should be located nearer to the free end of the anchorage block. This will increase the bond length over which the plate tension is transferred to the bolts and the plate splitting problem, which occurs when the bolts are positioned in the centre of the plate, will be eliminated. In all anchorage systems, a suitable adhesive for bonding the anchorage block to the composite must be used; in the ROBUST project the 3M adhesive was employed.

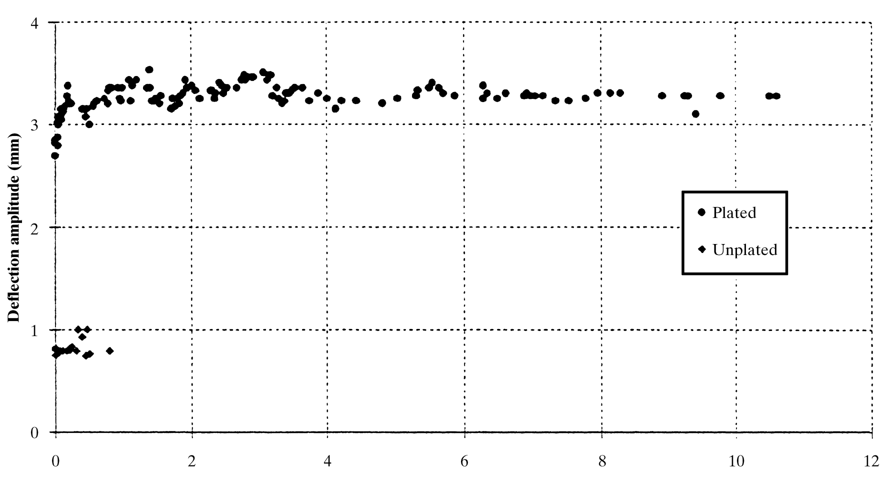

Figure 7.10 Deflection amplitude variations of unplated and plated beams (Garden, 1997).

The fatigue performance of the composite material is usually excellent so this will not be a design issue. Even under an upper cyclic plate stress representing 50% of the plate strength, using a prestressed composite in the ROBUST work, the CFRP plate material exhibited no indication of damage.

7.16 Fatigue of long span plated 2.3 m beams using FRP plates

The ROBUST project continued by undertaking research into the fatigue, at the RMCS, of long span plated beams using FRP plates. The aim of the experimental programme was to provide initial design guidance on the fatigue performance of reinforced concrete beams strengthened with CFRP plates. Six reinforced concrete beams, 2300 mm long, 130 mm wide and 230 mm deep were used for this study, similar sized beams being utilised in the static testing regime discussed in Chapter 4. Reinforcement consisted of three number T12 bars in the tension zone at an effective depth of 205 mm, two number T8 top bars and R6 links at 150 mm centres to ensure a flexural failure mode. All the steel was tied rather than welded in order to avoid fatigue failures of any welds. The beams were cast using grade C50 concrete. Four of the six beams were strengthened by bonding the CFRP plate developed in the ROBUST project.

For testing, the beams were supported on a purpose built rig which permitted the beams to behave in a simply supported manner but which also limited the amount of longitudinal and transverse movement which could develop during the test. Load was applied via a servohydraulic actuator to a spreader beam and hence to the beam itself through rollers in order to achieve four point bending. The beams spanned 2100 mm and were loaded at two points, each 130 mm from the centre span. Figure 7.11 shows the testing arrangement.

Prior to the fatigue loading, a static load test was undertaken up to the maximum load to be used in the subsequent fatigue loading cycle. Each beam was subjected to a cyclic load in four point bending at a frequency of 1 Hz. This frequency was selected on the basis of previous research (Emberson and Mays, 1996).

For the static test, the midspan deflection and the strains across the section at midspan were measured, using a linear variable differential transformer (LVDT) and a demec gauge, respectively, at each load increment. During the fatigue test, the load, midspan deflection and number of cycles were monitored. At 200 min intervals, 25 data points were recorded, at a rate of 20 data points per minute. For beam number six, the strain in the concrete and the CFRP plate was also recorded.

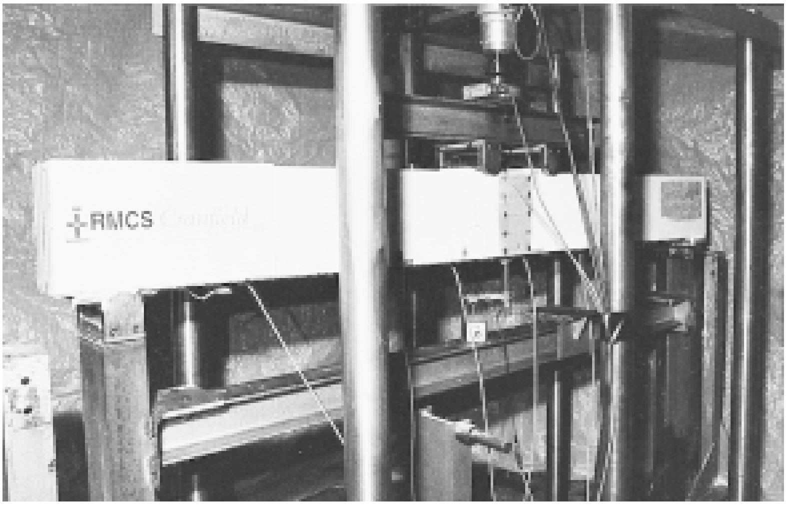

Figure 7.11 Long span beam fatigue testing apparatus.

7.16.1 Results of the 2.3 m long beams tested under ROBUST

Six beams, two unplated control beams and four plated beams, were tested. The load conditions are detailed in Table 7.2. The main objective of the programme was to obtain data that will enable an understanding to be gained of the fatigue performance of reinforced concrete beams strengthened with CFRP plates. This incorporates the dual aims of:

1 representing the fatigue loading on reinforced concrete structures,
2 comparing the fatigue life and behaviour of an unplated beam and a comparable plated beam.

The beam types to be discussed below will be referred to as beams 1 to 6, where beams 1 and 2 are the unplated beams and beams 3, 4, 5, and 6 are the plated members. When a fatigue load investigation is undertaken there are various loading options which can be applied to the beams being examined; the options chosen in the ROBUST project are explained below.

1 To apply the same loads to both the unplated and plated beams. In many cases in practice, a degraded beam needs to be strengthened to its original capacity. Beams 1 and 4 were cycled under the same loading magnitudes.
2 To apply loads to both types of beam to give the same stress in their respective rebars. In order to make a comparison, the unplated and plated beams should be tested at the same percentage of ultimate stress

Table 7.2 Loading conditions and results summary of fatigue tests at RMCS on 2.3 m beams

Test no.	Type of beam	Predicted ultimate capacity (kN)	Load range (kN)	Load range[1]	No. of cycles	Max. stress[2] (MPa)	Stress range (%)	Failure mode
1	Unplated	75	4–40	48	20 000	54.8	49.3	Steel yield
2	Unplated	75	3–32	39	732 600	43.7	39.3	Steel yield
3	Plated	113	5–49	39	508 500	54.8	49.3	Steel fracture
4	Plated	113	4–40	32	321 195	43.7	39.3	Steel fracture
5	Plated	113	4–40	32	1 889 087	43.7	39.3	Steel fracture
6	Plated	113	2.9–32	25.9	12×10^6	36.3	33.0	Test stopped

[1] % of ultimate capacity.
[2] % of ultimate stress in steel bar at maximum load.

in the steel at maximum load and not at the same stress value. Beams 1, 2 and 3, 5 were tested with comparable values of percentage of ultimate stress in the steel rebars at maximum load.

3 To apply an equal percentage of the ultimate load capacity of each respective beam. If an unplated beam were tested at 50% of its ultimate capacity, the plated beam should be tested also at 50% of its ultimate capacity. Beams 2 and 3 were tested at comparable values of ultimate capacity.

Beam 4 failed after a surprisingly low number of cycles and the load conditions were repeated for beam 5. An examination of the static test results showed that beam 4 was considerably less stiff than the other beams, suggesting a possible weakness present in this beam which may have led to premature failure. In general, however, the static test results showed that the unplated beams (beams 1 and 2) are less stiff than the plated beams (beams 3, 5 and 6), as would be expected.

Typical fatigue duration on reinforced concrete structures is in the region of 10^7 cycles or more. However, within the confines of the programme, it was not possible to conduct cycling on each beam for such a long period of time (10^7 cycles at 1 Hz is about 4 months).

The results are summarised in Table 7.2 and are also displayed graphically on the *S–N* curves displayed in Fig. 7.12 alongside the curves for reinforced concrete beams obtained by TRRL (Moss, 1982) and Concrete in the Oceans (CiO) (Leeming, 1989).

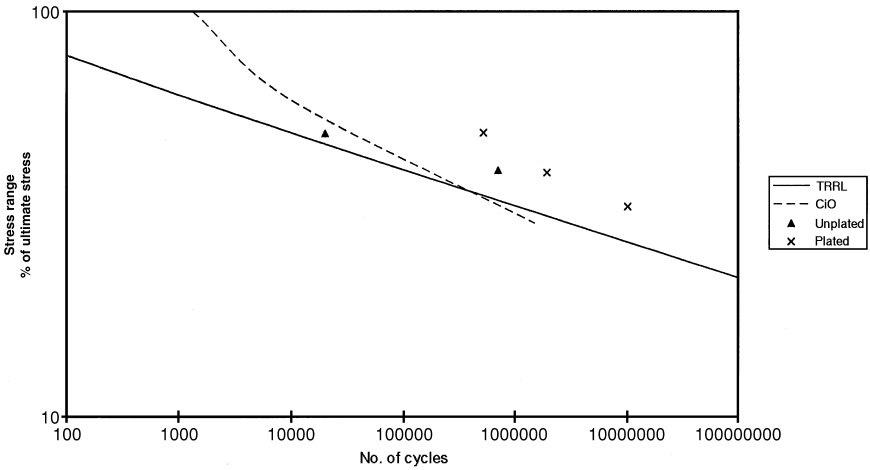

Figure 7.12 S–N curve for fatigue of CFRP plated beams compared with TRRL (Moss, 1982) and CiO (Leeming, 1989) curves.

7.16.2 Discussions of the 2.3 m span beams using FRP plates

The fatigue results will now be assessed by referring to the three load options described above. Considering each of these in turn:

1 Beams 1, 2, 5 and 6 were cycled under the same load. The strengthened beam, in each case, had a considerably enhanced fatigue life.
2 Beams 1, 2, 3 and 5 were tested with comparable values of percentage of ultimate stress in the steel bar at maximum load. Again the plated beams exhibit an enhanced fatigue life.
3 Beams 2 and 3 were tested at the same percentage of ultimate capacity. The plated beam had a shorter fatigue life than the unplated beam.

From the above assessment of the results it would be unwise to expect the same fatigue capacity, in terms of percentage of ultimate capacity, from a plated beam as from an unplated one. A more reasonable value for design guidance would be to expect the same fatigue life for plated and unplated beams with comparable values of percentage of ultimate stress in the steel bar at maximum load.

A study of the failure mechanisms was undertaken, the results of which are described below:

- Beam 1: there was no evidence of failed rebars. It is assumed that failure was by yielding of reinforcement.
- Beam 2: the load dropped to a value of 20 kN after 732 600 cycles. At 744 660 cycles, the beam was unable to sustain an increase in load to

enable the original value of 32 kN to be restored. It appeared that the steel had yielded but no sign of rebar damage was found.

- Beam 3: at 508 500 cycles, a large deflection occurred, presumably due to fracturing of the main internal reinforcement. The cyclic load continued at a low value (0–20 kN) with only a partially bonded CFRP plate. The cyclic load stopped after 584 100 cycles, when the bottom steel fractured.
- Beam 4: at 321 200 cycles, a large deflection occurred, presumably due to fracturing of the main reinforcement. The cyclic load continued at a low value (0–20 kN) with only a partially bonded CFRP plate. When an attempt was made to increase the load to its original value, the CFRP plate failed at the anchorage and the beam hinged about the top steel at 568 200 cycles; the bottom steel fractured at this point.
- Beam 5: at 1 889 087 cycles, a large deflection occurred, presumably due to fracturing of the main internal reinforcement. The cyclic loading continued at low load with the main reinforcing steel fractured and the load being taken in the tension zone of the beam by the composite plate. When an attempt was made to increase the applied load to its original value, the CFRP debonded, the deflection increased, the plate split and finally pulled away from the end anchors. The beam hinged about the top reinforcement and upon inspection it could be seen that the bottom steel had fractured.
- Beam 6: during the initial static loading, the strain was highest at midspan, dropping considerably towards the end of the plates. During the fatigue test, an overall increase in strain at each location indicated stress transfer to the plate as the beam cracked. There was a corresponding increase in compressive strain on the top surface of the concrete beam. There did not appear to be any stress redistribution with time along the length of the plate. The test was concluded at a value of 2.5×10^6 cycles.

7.17 Concluding summary

The component parts of an RC plated beam have all been shown to exhibit varying degrees of creep deformation whilst under a constant load. Creep tests of plated beams strengthened by bonding CFRP composite plates on to their soffits have demonstrated that plated and unplated beams experience similar proportional creep, suggesting that the creep behaviour of such beams is governed primarily by the compressive creep of the concrete. The effect of temperature upon the creep performance of adhesives dictates that the use of plate bonding at elevated temperatures should be viewed with caution.

Composite materials exhibit superior fatigue performance to that of steel and from the research to date it would appear that it is the fatigue of the reinforcing steel within a plated reinforced concrete beam that is the dominant factor in fatigue performance. Thus, it is recommended that for satisfactory fatigue performance the design of CFRP plated beams should limit the performance of the ultimate stress in the steel reinforcement at maximum load to that allowed in an unplated beam.

7.18 References

Aboudi J (1990) 'Micromechanical characterisation of the nonlinear viscoelastic behaviour of resin matrix composites', *Composites Sci Technol* **38** 371–386.

ACI (1982) *Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures,* ACI Committee 209, ACI Detroit.

Adams R D and Wake W C (1984) *Structural Adhesive Joints in Engineering* London, Elsevier Applied Science.

Bannister J L (1969) 'The behaviour of reinforcing bars under fluctuating stress', *Concrete* **3**, 405–409.

Barnes R A and Mays G C (1988) *Creep of Adhesive Joints at Elevated Temperatures,* RMCS Contractor's Report to TRRL, July 1988.

Beaumont P W R and Harris B (1971) 'The effect of environment on fatigue and crack propagation in carbon fibre reinforced epoxy resin', in *Carbon Fibres, Their Composites and Applications,* Proceedings International Conference, organised by the Plastics Institute, London, Feb 2–4, 1971, pp 283–291.

Calder A J J (1990) 'The durability of steel plates bonded to concrete with structural epoxy adhesives', *International Seminar Structural Repairs/Strengthening by the Plate Bonding Technique,* University of Sheffield, September 1990.

Canovas F M (1990) 'Strengthening of structural elements of reinforced concrete by means of glued steel plates – some applications in Spain', *International Seminar Structural Repairs/Strengthening by the Plate Bonding Technique,* University of Sheffield, September 1990.

Chajes M J, Thomson T A and Farschman C A (1995a) 'Durability of concrete beams externally reinforced with composite fabrics', *Construct Building Mater* **9**(3) 141–148.

Chajes M J, Thomson T A and Tarantino B (1995b) 'Reinforcement of concrete structures using externally bonded composite materials', in *Non-Metallic (FRP) Reinforcement for Concrete Structures,* ed. L. Taerwe, London, E & FN Spon.

Cottingham W S, Fluck P G and Washa G W (1961) 'Creep of prestressed concrete beams', *ACI J* **57** 929–936.

Crossman F L and Flaggs D L (1978) Lockheed Missiles and Space Center, Report No D633086.

Curtis P T (1989) 'The fatigue behaviour of fibrous composite materials', *J Strain Anal* **24**(4) 235–244.

Demers C (1997a) 'Tension–tension axial fatigue of carbon fiber reinforced polymeric composites', in *Recent Advances in Bridge Engineering. US – Canada-Europe Workshop on Bridge Engineering,* Zurich, July 1997, pp 170–177.

Demers C (1997b) 'Tension–tension axial fatigue of E-glass fiber reinforced polymeric composites, fatigue strength and fatigue models', *Structural Faults and Repair 97*, Edinburgh July 1997, pp 175–182.

Deuring M (1993) 'Verstärken von Stahlbeton mit gespannten Faserverbund-werskstoffen', *EMPA Bericht Nr. 224*.

Dew-Hughes D and Way J L (1973) 'Fatigue of fibre-reinforced plastics: a review', *Composites, Part A* **4** 167–173.

Dharan C K H (1975) 'Fatigue failure mechanisms in a unidirectionally reinforced composite material', in *Fatigue of Composite Materials*, Proceedings of a symposium presented at December committee week of the American Society for Testing and Materials, Bal Harbour, 1975.

Dillard D A and Brinson H F (1983) 'A numerical procedure for predicting creep and delayed failures in laminated composites', in *Long-Term Behaviour of Composites*, ed. T K O'Brien, ASTM Special Technical Publication No. 813, 1983, Philadelphia, pp 23–37.

Ellyin F and Kujawski D (1992) 'Fatigue testing and life prediction of fibreglass-reinforced composites', in *1st Int. Conf. Advanced Composite Materials in Bridges and Structures*, eds K W Neale and P Labossière, Sherbrooke, Canada, 1992, pp 111–118.

Emberson N K and Mays G C (1996) 'Significance of property mismatch in the patch repair of structural concrete. Part 3: Reinforced concrete members in flexure', *Mag Concrete Res* **48**(174) 45–57.

Evans R H and Kong F K (1996) 'Creep of concrete in R C and P C design', *Civil Eng Public Works Rev* **61**(718) 594–596.

Ferry J D (1980) *Viscoelastic Properties of Polymers*, 3rd edn. John Wiley and Sons, New York.

Findley W N (1960) 'Mechanism and mechanics of creep of plastics', *SPE J* 57–65.

Garden H N and Hollaway L C (1997) 'An experimental study of the strengthening of reinforced concrete beams using prestressed carbon composite plates', *7th Int Conf Structural Faults and Repair*, Edinburgh, 1997.

Garden H N (1997) *Part 2 Response of CFRP plated Reinforced Concrete beams to sustained load, Composite Materials*, Internal Report, Department of Civil Engineering, University of Surrey, July 1997, pp 20–56.

Garden H N, Hollaway L C, Thorne A M and Parke G A R (1996) 'A parameter study of the strengthening of reinforced concrete beams with bonded composites', *3rd Int Conf Bridge Management, Guildford*, E & F N Spon, London, 1996.

Garden H N, Hollaway L C and Thorne A M (1997) 'A preliminary evaluation of carbon fibre reinforced polymer plates for strengthening reinforced concrete members', *Struct Buildings* **122**(2) 127–142.

Hankers C (1990) Strengthening of structural concrete members by bonded steel plates', *International Seminar Structural Repairs/Strengthening by the Plate Bonding Technique*, University of Sheffield, September 1990.

Hiel C C, Brinson H F and Cardon A H (1983) 'The nonlinear viscoelastic response of resin matrix composites', *2nd Int Conf Composite Structures*, Paisley, September 1983, pp 271–281.

Hollaway L C (1993) *Polymer Composites for Civil and Structural Engineering*, Glasgow, Blackie Academic and Professional.

Holtgreve K (1986) 'A contribution in regard to behaviour of reinforced tee-beams with bonded flatsteel components under fatigue loads, *Proceedings International Conference on Adhesion between Polymers and Concrete*, RILEM Technical Committee 52, Aix-en-Provence, September 1986.

Howard C L and Hollaway L C (1987) 'The characterisation of the nonlinear viscoelastic properties of a randomly orientated fibre/matrix composite', *Composites* **18**(4) 317–323.

Hull D (1992) *An Introduction to Composite Materials*, Cambridge, Cambridge University Press.

Johnson A F (1979) *Engineering Design Properties of GRP*, Publication 215, British Plastics Federation, London.

Kaiser H (1989) *Bewehren von Stahlbeton mit kohlenstoffaserverstärkten Epoxidharzen*, Doctoral Thesis, Eidgenössischen Technischen Hochschule Zürich, 1989.

Kim H C and Ebert L J (1978) 'Axial fatigue failure sequence and mechanisms in unidirectional fiberglass composites', *J Compos Mater* **12**(April) 139–152.

Ladner M and Weder C (1981) *Concrete Structures with External Bonded Reinforcement*, EMPA Report 206, Dubendorf.

Lark R J and Mays G C (1984) 'Epoxy adhesive formulation: its influence on civil engineering performance', in *Adhesion 9*, ed. K W Allen, Elsevier Applied Science, London, Chap 7, pp 95–111.

Leeming M B (1989) *Concrete in the Oceans Programme – Coordinating Report on the Whole Programme*, London, HMSO.

Lovegrove J M, Salah El, Din A S and Daoud O K (1979) 'Fatigue crack growth in the tension steel of reinforced concrete', in *Fatigue Eng Mater* **2**(3) 173–183.

Marceau J A, McMillan J C and Scardino W M (1978) 'Cyclic stress testing of adhesive bonds', *Adhesives Age* **21**(4) 37–41.

Matting A and Draugelates H (1968) 'Schwingfestigkeit von Metallklebverbindungen (The vibrational stability of metal adhesive joints)' *Adasion* **11**(1) 5–20, (3) 110–134, (4) 161–176.

Mays G C (1985) *Fatigue Performance and Durability of Structural Adhesive Joints*, PhD Thesis, University of Dundee.

Mays G C (1990) *Fatigue and Creep Performance of Epoxy Resin Adhesive Joints*, Transport and Road Research Laboratory Contractor Report 224.

Mays G C (1993) 'The use of bonded external reinforcement in bridge strengthening: structural requirements of the adhesive', in *Bridge Management 2, Proceedings of the 2nd International Conference on Bridge Management*, eds J E Harding, G A R Parke and M J Ryall, University of Surrey, Guildford, April 18–21 1993, pp 672–680.

Mays G C and Hutchinson A R (1992) *Adhesives in Civil Engineering*, Cambridge, Cambridge University Press.

Meier U and Kaiser H (1990) 'Strengthening of structures with CFRP laminates', *Proc. Int. Seminar Structural Repairs and Strengthening by the Plate Bonding Technique*, University of Sheffield, 1990.

Meier U and Kaiser H (1991), 'Strengthening of structures with CFRP laminates', *Proc. Specialty Conf. Advanced Composites Materials in Civil Engineering Structures*, Nevada.

Meier U, Deuring M, Meier H and Schwegler G (1993a) 'CFRP bonded sheets', in *Fiber-Reinforced-Plastic Reinforcement for Concrete Structures*, Proc. Int. Symposium, eds A Nanni and C W Dolan, Vancouver, 1993, ACI Special Publication No. 138.

Meier U, Deuring M, Meier H and Schwegler G (1993b) 'Strengthening of structures with advanced composites', in *Alternative Materials for the Reinforcement* *and Prestressing of Concrete*, ed. J L Clarke, Glasgow, Blackie Academic and Professional.

Moss D S (1982) *Bending Fatigue of High Yield Reinforcing Bars in Concrete,* Transport and Road Research Laboratory supplementary report 748.

Neville, A M (1970) *Creep of Concrete: Plain Reinforced and Prestressed,* North-Holland, Amsterdam.

Neville A M, Dilger W H and Brooks J J (1983) *Creep of Plain and Structural Concrete*, Construction Press, Longman Group, London.

Owen M J (1974) 'Fatigue of carbon fibre reinforced plastics', in *Composite Materials, 5*, ed. L J Broutman, London, Academic Press.

Plevris N and Triantafillou T C (1994) 'Time-dependent behavior of RC members strengthened with FRP laminates', *J Struct Eng* **120**(3) 1016–1042.

Ponsot B, Valentin D and Bunsell A R (1989) 'The effects of time, temperature and stress on the long-term behaviour of CFRP', *Composites Sci Technol* **35** 75–94.

Renton W J and Vinson J R (1975) 'Fatigue behaviour of bonded joints in composite material structures', *J Aircraft* **12**(5) 442–447.

Schaffer and Adams (1981) *J. Appl. Mech.* **48** 859.

Scott D W, Lai J S and Zureick A H (1995) 'Creep behavior of fiber-reinforced polymeric composites: a review of the technical literature', *J Reinforced Plastics Compos* **14**(6) 588.

Shijie W and Ruixian Z (1993) 'The study of fiber composite plates for strengthening reinforced bridges', *Proceedings 9th Int. Conf. Composite Materials*, Madrid, July 1993, pp 224–231.

Sillwood, J M (1982) *Factors Concerning the Use of GRP in Corrosive Environments,* NPL Report DMA(B)20, Jan. 1982.

Standing Committee on Structural Safety (1983) '5th Report', *The Struct Eng* **61A**(2) 65–69.

Swamy R N, Hobbs B and Roberts M (1995) 'Structural behaviour of externally bonded, steel plated RC beams after long-term exposure', *The Struct Eng* **75**(16) 255–261.

Taljsten B (1994) *Plate Bonding – Strengthening of Existing Concrete Structures with Epoxy Bonded Plates of Steel or Fibre Reinforced Plastics,* Doctoral Thesis, Lulea University of Technology.

Tilly G P (1979) 'Fatigue of steel reinforcement bars in concrete: a review', in *Fatigue of Engineering Materials*, **2**(3) 251–268.

Tilly G P and Moss D S (1982) 'Long endurance fatigue of steel reinforcement', *Proc. IABSE Coll.* Lausanne, pp 229–238.

Tuttle M E and Brinson H F (1986) 'Prediction of the long-term creep compliance of general composite laminates', *Exper Mech* **26**(1) 89–102.

Wake W C, Allen K W and Dean S M (1979) 'The expected service life of structural adhesives', in *Elastomers Criteria for Engineering Design*, ed. C Hepburn and R J W Reynolds, Applied Science.

Wu, E M and Ruhmaan, D C (1975) *ASTM Special Technical Publication* 580, p 263.

8

Analytical and numerical solutions to structural strengthening of beams by plate bonding

P S LUKE

8.1 Introduction

One of the main successes of ROBUST has been the development of practical design rules and guidelines for flexural strengthening of reinforced (RC) and prestressed (PC) concrete beams. In general, designing a structural member using a known code or guideline involves solution of 'simple' design equations. Often to complement these equations, codes provide additional data in the form of graphs, charts and/or tables assisting the engineer to obtain an optimum design. The additional data is normally generated experimentally or mathematically using closed form solutions or a suitable numerical technique. Experience has shown that exhaustive testing is a very expensive and time-consuming process and in recent years more emphasis has been placed on numerical simulation to complement testing. The development of high speed computers and more sophisticated non-linear constitutive material models capable of simulating exactly what happens experimentally has helped to make this transition. Previous chapters have described in detail the wide range of experimental tests that have been carried out in the laboratory and on site. These tests covered a wide range of parameters, but it is possible to use analytical techniques to extend the parameters to enable additional data for the development of design rules for use in the construction industry.

The role of the finite element (FE) method in predicting the measured experimental results carried out in the project is discussed. Its reliability in generating design data, thus forming part of the process of developing practical design rules and guidelines for use of these new materials in the construction industry, is also discussed. Some of the problems associated with prediction of non-linear behaviour of cracked reinforced and prestressed concrete beams using the finite element method are examined. The results obtained from finite element analysis and experimental tests are compared and a series of recommendations are made.

8.2 Classical analysis

Researchers (Meier, 1987; Deuring, 1993) have stated that with the substitution of appropriate properties, classical reinforced concrete beam theory can be applied safely when using advanced composite plates to strengthen beams in flexure. Strengthened full scale T-beams tested at EMPA (Deuring, 1993) also seem to validate the strain compatibility method used in the analysis of cross-sections. These statements have been examined carefully in the light of results obtained from beam tests in ROBUST. An extensive desk study using the classical theory has been carried out on the ROBUST test beams to predict the load/deflection history, the deflection of the beams and the mode of failure of the beams. Classical theory assumes that plane sections remain plane throughout the load history.

The classical theory in general predicted the load/deflection curves reasonably accurately bearing in mind the inherent assumptions of the method. However in most cases, while classical theory predicted that the plates would fail in tension, this phenomenon was not experienced in any of the tests on beams with unstressed plates. In addition, the concrete compressive strain at failure in the test beams varied from 0.19–0.25%, whereas a strain of 0.35% is normally used for design purposes. It can be deduced from the test results that due to the inherent conservatism built into the classical theory, it can be used safely to design beams strengthened in this manner.

8.3 Finite element analysis

Most classical theories for reinforced and prestressed concrete design are based on relatively simple equations. These theories are generally based on linear elastic models. They are usually not capable of dealing with problems where gross material and geometric non-linearities exist. In the absence of design formulae to model accurately the behaviour of reinforced and prestressed concrete beams strengthened with advanced composite materials, the finite element method has been used to model the experimental data. The finite element method is a versatile and powerful numerical technique for analysis of engineering problems. Being a numerical tool, accuracy of the generated results is dependent on a number of factors.

One of the main approximations associated with non-linear behaviour of concrete problems is accurate modelling of concrete cracking. Under application of load, concrete cracks in the tension zone when stresses exceed its tensile strength. The number of cracks and their propagation pattern across the depth of a concrete beam are unique and impossible to predict accurately. In addition, when concrete cracks, the stress path becomes discontinuous and the load transfer changes at the cracked section. Development of accurate material criteria and concrete elements that would model discrete cracking of concrete is the most challenging part of any numerical technique.

The finite element technique has evolved over the past few years and today many sophisticated constitutive models are available which can reasonably predict various features of concrete behaviour including cracking, crushing and creep. Various procedures have been adopted for predicting cracking in concrete and these fall broadly into two categories, namely the discrete crack approach and the smeared crack approach.

The smeared crack approach has been adopted widely for predicting the non-linear behaviour of concrete (William and Warnke, 1975; Rots and Blaauwedraad, 1989). In this approach cracks are modelled as local discontinuities which are distributed or smeared within the finite element model. Using this technique, relative displacements of crack surfaces are represented by crack strains and the constitutive behaviour of cracked concrete can be modelled in terms of a stress–strain relationship. This approach fits the nature of the finite element method, as the displacement continuity field remains intact. However, it does have shortcomings, as it tends to spread crack formation over the entire structure, which makes it difficult to predict localised failures. This has not stopped the method being used widely and accurately for predicting load/deflection, load/strain behaviour of concrete.

The discrete crack approach seems the most obvious way of modelling concrete, because of the nature of concrete cracking which lends itself to a discontinuous nature. With this technique, cracks are introduced into the finite element model using interface elements between the concrete ones. The problem with this approach is that the position and direction of crack growth within the model is predefined, which is not so for concrete. Numerous models are being developed to introduce automatic remeshing techniques, which would help in alleviating some of these problems and allow cracks to propagate and change direction throughout the model.

Most commercially available finite element codes have material models capable of modelling concrete. Codes such as ANSYS, ABAQUS and LUSAS have models which treat concrete when it cracks as a smeared band of cracks. A more recent finite element code, DIANA, has developed constitutive models for concrete whereby the cracks are treated as discrete cracks. Damage models have also been developed in recent years for modelling non-linear effects in materials. The approach has been extended to concrete and offers a powerful alternative to the traditional smeared and discrete crack methods. It is quite clear from the developments in non-linear material models that more versatile models will be available in the major finite element codes in the next few years.

Within the ROBUST project ANSYS, ABAQUS and LUSAS were used to model the beams tested in the laboratory and on site. Oxford Brookes

University and University of Surrey used LUSAS and ABAQUS respectively to model their beams. This chapter mainly discusses the work carried out by Mouchel in developing numerical models to validate and predict the experimental results, both in the laboratory and on site.

Mouchel's main finite element package (ANSYS, 1993, 1994, 1995) was used to carry out analysis of over one hundred 1.0 m, 2.3 m and 4.5 m reinforced concrete beams tested in four point bending under the ROBUST project at both Oxford Brookes University and University of Surrey and ten post-tensioned prestressed concrete beams removed from the A34 Botley flyover in Oxfordshire, UK and tested on site by the Royal Military College of Science. Initially, the analysis was carried out using eight noded quadrilateral isoparametric two-dimensional (2D) plane–stress elements and later using eight noded three-dimensional brick elements to model the concrete.

8.3.1 Two-dimensional analysis

In the 2D analysis, two plasticity material options were utilised to model cracking of concrete, namely Drucker-Prager (DP) failure criteria and bilinear kinematic hardening (BKIN). Material characterisation tests were carried out in the laboratory to determine properties of the steel rebar, carbon fibre reinforced polymer (CFRP) and glass fibre reinforced polymer (GFRP) plates and the concrete used in the analysis, and these were used as a starting point for the 2D analyses. Typical measured values of the modulus of elasticity (E) for the beams tested in the laboratory were 35 GPa, 205 GPa and 140 GPa for the concrete, steel and CFRP, respectively. The measured compressive strength (f_{cu}) for the concrete ranged from 46–60 MPa with its tensile capacity measured at 1.5–3 MPa. Mild steel bars were used for the 1.0 m beams with a yield stress of 350 MPa and an ultimate stress of 460 MPa. High yield steel bars were used for the 2.3 m and the 4.5 m beams with a measured yield stress of 460 MPa and an ultimate stress of 556 MPa. CFRP is a brittle material with a linear–elastic stress–strain relationship to failure. The measured ultimate tensile capacity was 1500 MPa.

Although the behaviour of the plated beams was modelled with acceptable accuracy using DP, satisfactory results were not obtained when attempting to trace the load deflection path of the unplated beams. DP uses the Von-Mises yield criterion with the associative flow rule. The material response here is elastic perfectly plastic. The yield surface does not change with progressive yielding, hence there is no hardening rule. Input parameters required are an angle of internal friction and dilatancy and a cohesion value. Research has been carried out to determine these input parameters for concrete, but they can be variable. To cater for this variability, a major parameter study was carried out to study the sensitivity to the input values. It was observed that the models were very sensitive to these values. In addition, the FE models had to be fine tuned by adjusting material properties for every test, which rendered its results unreliable for predicting behaviour of other beams in the absence of experimental data. The second material option examined was BKIN, which uses the Von-Mises yield criterion with the associated flow rule and kinematic hardening. This is better suited for metals as failure of concrete is brittle unlike most metals, which are ductile. Other material options available in ANSYS, namely the multilinear and anisotropic failure criteria, were also examined with the BKIN giving the closest correlation with the experimental results, and this was adopted in the 2D analysis without the hardening option.

The BKIN option in ANSYS assumes that the material strength in both tension and compression is equal. Concrete has a different strength in tension and compression hence it was necessary to divide the mesh generated for a beam into regions with different material properties as shown in Fig. 8.1. Regions '1' and '2' show the initially assumed tension and compression areas of the beam. This was later modified by varying the tension depth of the beam after examining results of the analysis to find failure either in the concrete or the CFRP plate. This was repeated until failure was obtained. Figure 8.2 shows that the load deflection graph generated by the FE analysis is reasonably close to the experimental results. The FE strain values of the CFRP plate did not agree well with the measured experimental results. In addition, the method was rather time consuming as it required several iterations to modify the tension–compression regions to achieve failure. In these FE analyses the actual measured material properties were used throughout the iterations.

In general, the 2D analyses gave an indication of the behaviour of the strengthened beams, but as the constitutive models were based on plasticity methods more akin to metals, the results obtained could not have been used with confidence to predict the behaviour of the beams.

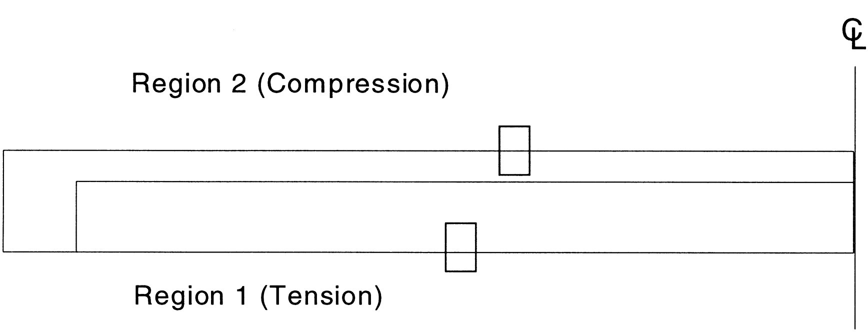

Figure 8.1 Division of 2.3 m beam for two-dimensional analysis.

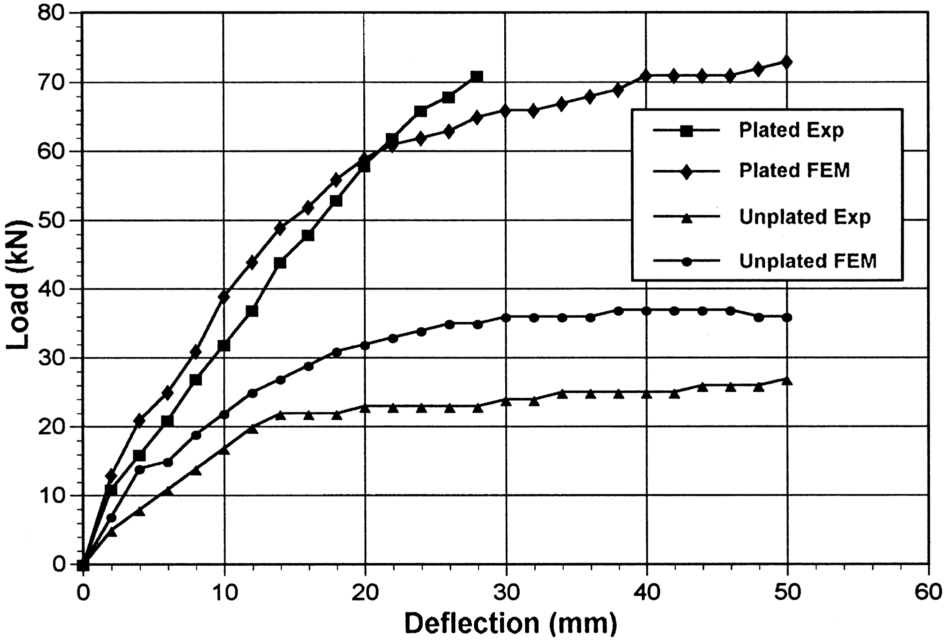

Figure 8.2 Load deflection plot for two-dimensional analysis of a 2.3 m long concrete beam.

8.3.2 Three-dimensional analysis

A three-dimensional brick element (SOLID65) in ANSYS, called '3D Reinforced Concrete Solid', is generally used for modelling solids with or without reinforcing rebars. In concrete applications, the element has the capability to model the concrete while the rebar capability is available for modelling the reinforcement behaviour. The most important aspect of this element is the treatment of non-linear material properties. The concrete is capable of cracking, crushing, plastic deformation and creep in three orthogonal directions at each integration point. When cracking occurs at an integration point, the cracking is modelled through an adjustment of material properties which effectively treats the cracking as a 'smeared band' of cracks, rather than discrete cracks. The presence of a crack is represented through modification of the stress–strain relations by introducing a plane of weakness in a direction normal to the crack face. A shear transfer coefficient is also introduced which represents a shear strength reduction factor for those subsequent loads which induce sliding (shear) across the crack face. There are 16 possible combinations of crack arrangement and appropriate changes in stress–strain relationships incorporated in the element. The open or closed status of integration point cracking is based on a strain value called the crack strain. If this value is less than zero, the associated crack is assumed to be closed. If the value is greater than zero, the associated crack is assumed to be open. When cracking first occurs at an integration point, the crack is assumed to be open for the next iteration.

If crushing occurs at an integration point, material strength is assumed to have degraded and has no contribution to the stiffness of the element at the integration point. A number of input parameters are required to define the concrete material data in addition to the *E* and Poisson's ratio values. These factors include the tensile cracking stress, crushing stress and shear transfer coefficients for open and closed cracks. Parametric studies were carried out on the shear transfer coefficients to test their sensitivity to the analysis. It was generally found that coefficients of 0.8–0.9 seemed to give good results for all the modelled beams.

Three-dimensional tension or link elements were used to model the reinforcement explicitly, as the reinforcement option of the element uses a smeared approach. Eight noded plate elements were used to model the adhesive and CFRP. Bilinear and multilinear material failure criteria were used for the CFRP and reinforcement, respectively. Figure 8.3 shows a typical finite element mesh of a 2.3 m beam tested at Oxford Brookes University (due to symmetry, one quarter of the beam was modelled for analysis). Figure 8.4 shows comparison of results of the FE analysis with the experimental values. It can be seen that the FE results compared favourably with the experimentally measured values for both plated and unplated

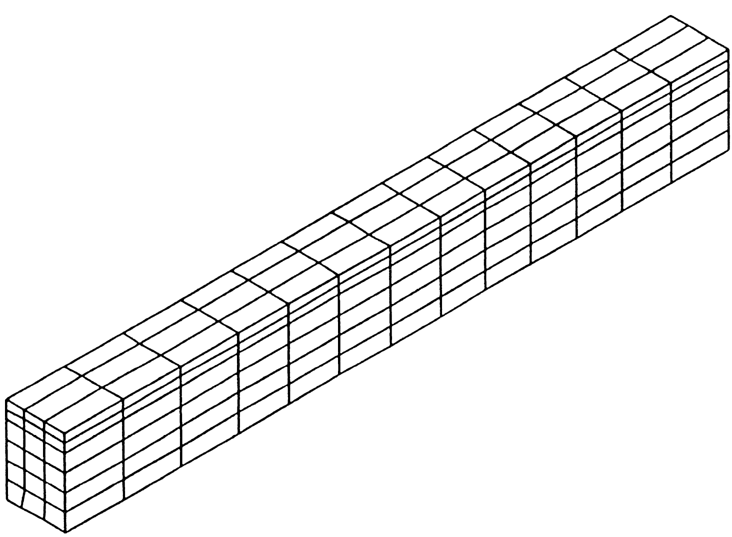

Figure 8.3 Three-dimensional mesh of a 2.3 m concrete beam (quarter model).

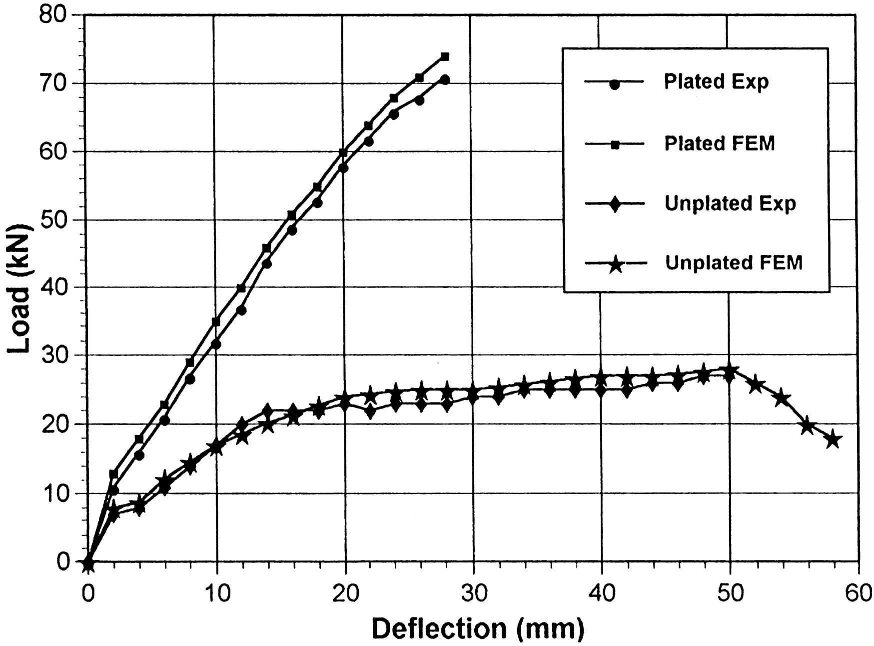

Figure 8.4 Load deflection plot for three-dimensional analysis of a 2.3 m long concrete beam.

concrete beams. The strain values of the CFRP plate calculated by FE also agreed well with the experimental results as shown in Fig. 8.5. Actual measured material properties were used throughout the load history of the beams, giving credence to the results.

This method of modelling the beams using three-dimensional finite element techniques has also been used to provide an accurate prediction of the load–deflection and load–strain history of the experimentally tested beams prior to carrying out the experimental tests. Figure 8.6 shows load deflection curves of 1.0 m beams tested at University of Surrey. The FE results were generated prior to testing the beams.

8.3.3 Effect of mesh density

When undertaking any finite element analysis, and in particular a nonlinear analysis, it is extremely important to choose suitable elements and an associated mesh in order to obtain a satisfactory solution to the problem. As the method is approximate, it is important to have a good understanding of the consequences of the assumptions when choosing the element types used and size of mesh. This allows the effects of the approximation to be

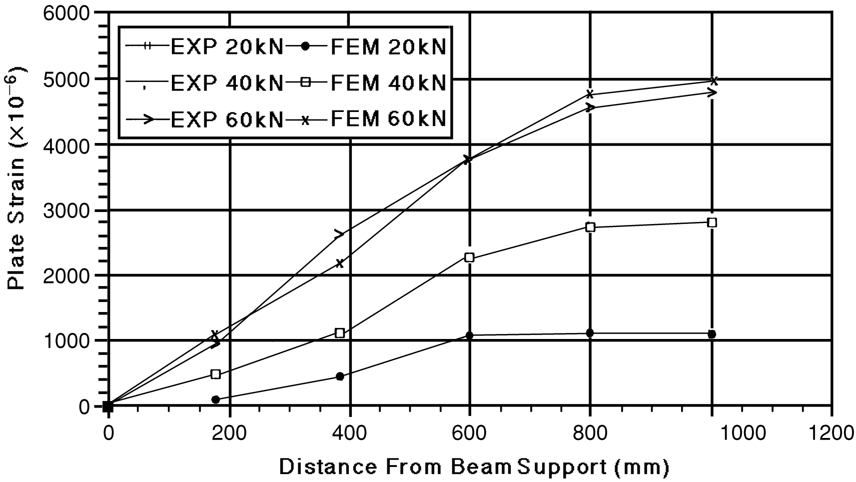

Figure 8.5 Plot of experimental and FE plate strains for 2.3 m long concrete beam.

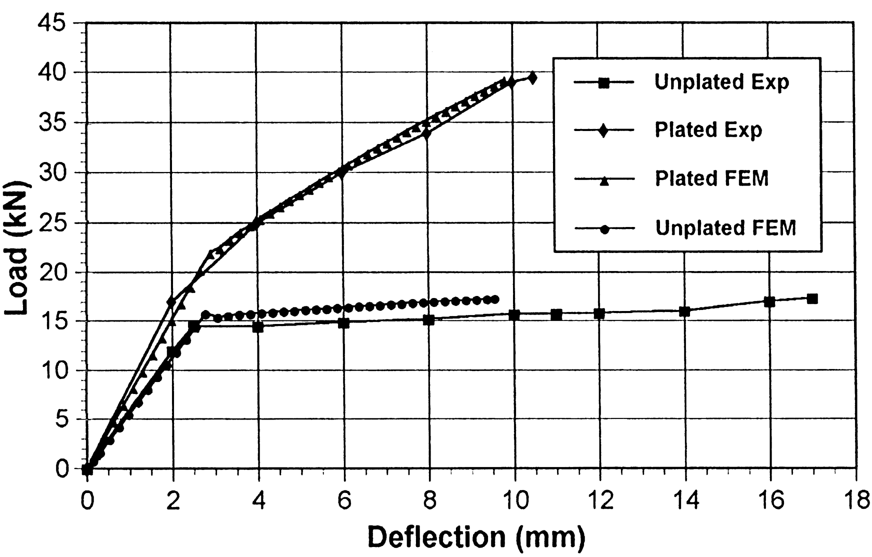

Figure 8.6 Load deflection graph for three-dimensional analysis of 1.0 m long concrete beam.

minimised within the solution. It requires skill and experience to be able to define a mesh that will produce accurate answers economically for all types of structural systems. In effect, the art of using the finite element method lies in choosing the correct mesh density required to solve a problem. If the mesh is too coarse then the inherent element approximations will not allow an accurate solution to be obtained and alternatively, if the mesh is too fine the cost of the analysis can be out of proportion to the results obtained. In order to address this problem, a linear and non-linear parametric study was carried out on the beams with meshes ranging from a fairly coarse to a very fine size. The results indicated that for a coarse mesh in particular, the non-linear analysis solutions were failing to converge and the results obtained were in some cases 50% different from the actual results. However, the solution results obtained from an economic mesh size and compared with one that was ten times more dense, were shown to be within 1% of each other; the solution time for the latter problem was more than three times that for the former. The problem utilising the fine mesh above required 7 h solution time on a Pentium 90 with 32 Mb of memory. It is clearly important, therefore, to choose a suitable mesh density for an analysis, to enable a balance to be obtained between the operation time and the accuracy constraints. Once this exercise had been undertaken for one set of beams, the resulting mesh could be used satisfactorily with minimal modifications made to it on other beam sizes within the programme.

8.4 Effect of adhesive material

A parametric study was carried out to determine the effect of the adhesive layer on the analysis and whether this layer could be ignored in the numerical analysis. The results of the analysis showed that peel and shear stresses increased non-linearly with an increase in adhesive elastic modulus. However, the predicted peak peel and shear stresses at the adhesive/concrete interface were low compared to the tensile and shear strengths of the Sikadur 31 adhesive used in the ROBUST project. This confirms the test results showing the concrete to be the weak link in the concrete adhesive layer. No significant change in stresses in the plate and load/deflection behaviour was observed as a result of omitting the adhesive layer. In addition, stress distributions at the concrete adhesive interface and at the concrete/external plate interface (for the model with no adhesive) showed similar characteristics. Consequently, it was decided not to include the adhesive layer in subsequent analyses, as it had little effect on the results, but had a major effect on solution times and mesh sizes.

8.5 Prestressed 18.0 m concrete beams

As a result of the success of the FE models to predict accurately the load–deflection and load–strain history of the 1.0 m, 2.3 m and 4.5 m beams tested in the laboratory, the models were further developed to predict the behaviour of the 18.0 m post-tensioned prestressed concrete beams removed from the A34 Botley flyover in Oxford which were tested in four point bending on site at Kidlington as described in Chapters 4 and 5. The ten beams, which were 18.0 m long by 711 mm deep by 343 mm wide, were originally stressed with five tendons. The original bridge had a reinforced concrete deck slab on top of the beams. As the beams in the ROBUST test conditions had their top slabs removed, the five tendons in the case of the beams would have represented an over-reinforced section. Hence three of the tendons were cored at 1.5 m centres in four places within the middle-third section of the beams, thus weakening the beams and making them suitable for strengthening by plate bonding. Furthermore, as there were only ten suitable beams on which to perform the site trials, it was necessary to choose parameters that would give the most meaningful results. The success of the finite element analysis in predicting the behaviour of the beams tested in the laboratory was a major factor in using the method to design the tests to be carried out on the site beams. The key parameters investigated included the length of plate, thickness of plate, number of layers of plates, end anchorage conditions of plate, the effect of plate prestress and stress transfer from the internal reinforcement to the external plate during coring. Core samples of the concrete were taken and tested to provide information on the strength and stiffness of the concrete as input data for the FE analysis. The measured E value was 35 GPa and f_{cu} was 42 MPa. The yield stress of the prestressing tendons was 1090 MPa and the ultimate stress was 1300 MPa.

Figure 8.7 shows the finite element mesh used to model the 18.0 m beams. As in the laboratory tests, the beams were simply supported on concrete abutments and loaded in four point bending. As the beams were symmetrical about their length and their cross-section, only one-quarter of the beam was modelled, reducing computing time and resources. Advanced features of the ANSYS program were used successfully to model the effects of prestressing.

Finite element analyses were carried out on models having two and three tendons, respectively. Although three of the five tendons had been cored in the middle sections of the beams, prestressing strands on either side of the core holes were still grouted and there was a possibility of some of the cored tendons reanchoring. The tendons outside the cored areas would then still be under a state of stress. It was anticipated that two tendons would give a lower bound solution and three tendons an upper bound solution. Figure 8.8 shows load–deflection plots for six unplated beams tested on site and the corresponding FE predictions for the beams with two and three tendons, respectively. Figure 8.8 also shows clearly that the FE predictions effectively bound the experimental results. The results also show a small variation in stiffness of the test beams. This figure shows very good correlation between the numerical and experimental results, lending further credence to the use of calibrated FE models as predictive tools.

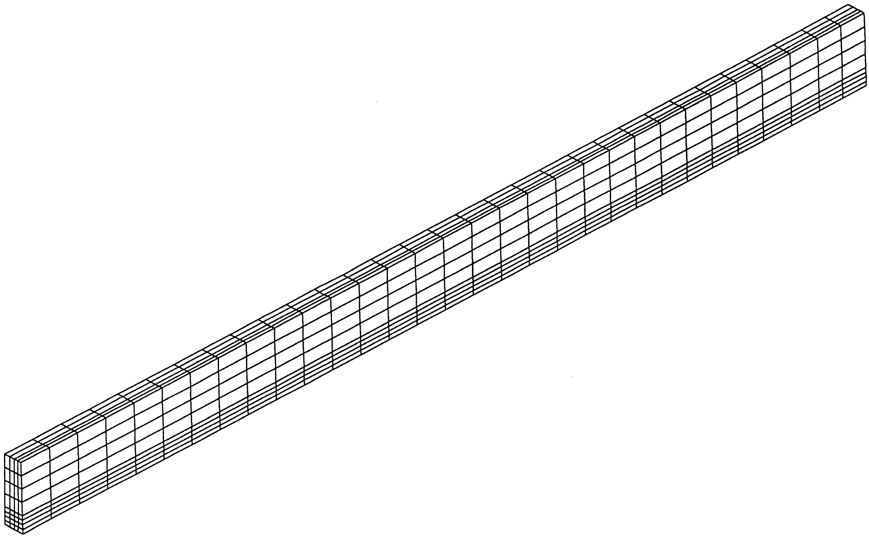

Figure 8.7 Three-dimensional mesh of an 18.0 m prestressed concrete beam (quarter model).

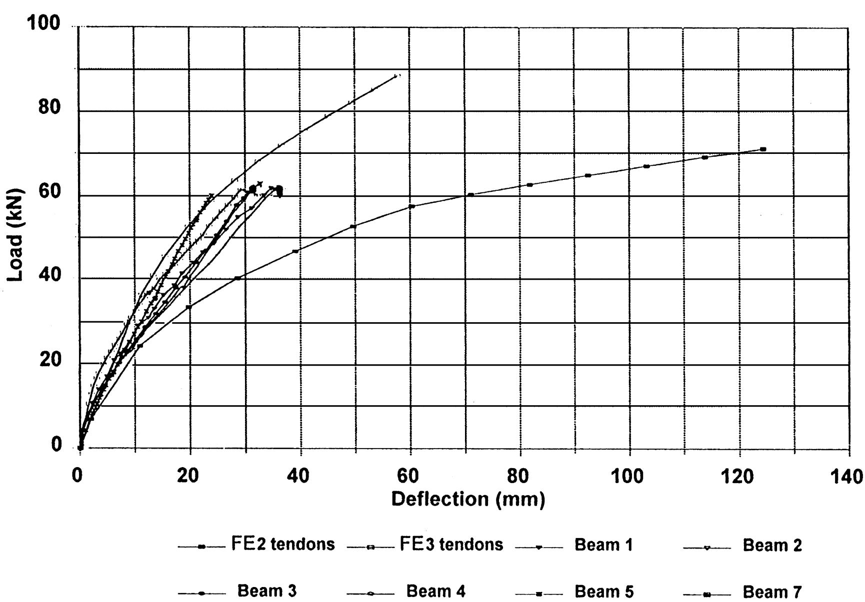

Figure 8.8 Experimental and numerical load deflection plots for unplated 18.0 m beams.

8.6 Beams with unstressed plates

Eight of the ten beams were strengthened with unstressed composite plates; beam numbers 1, 3, 4, 5 and 10 had three 90 mm wide by 1 mm thick plates, with various lengths and end conditions. Beam 1 had plates 12.5 m long and unanchored at the ends. Beams 3 and 5 had plates 6.0 m long; the ends of beam 3 were unanchored and those of beam 5 were anchored. The ends of beams 4 and 10 were unanchored and had composite plate lengths of 15.8 m and 9.25 m, respectively. FE analysis had revealed that peeling would not occur at the ends, even on the 6.0 m plates. Figure 8.9 shows load–deflection curves for beams 1, 3, 4, 5 and 10 and the corresponding FE predictions. The FE results for the variable length plates and the anchored plates showed little difference in strength and stiffness, hence only the worst response is presented here. This phenomenon is also borne out in the experimental results. An 18.0 m beam bonded with 6.0 m long unanchored plates provides approximately the same increase in stiffness and strength as beams having unanchored plates, 12.5 m or 15.8 m long, and as a beam with 6.0 m long anchored plates; all these beams had a single layer of three 90 mm wide by 1 mm thick plates side by side. However, stresses at the ends of the plates are higher in the shorter plates than in the longer plates, but not significantly so; a result predicted by the finite element analysis.

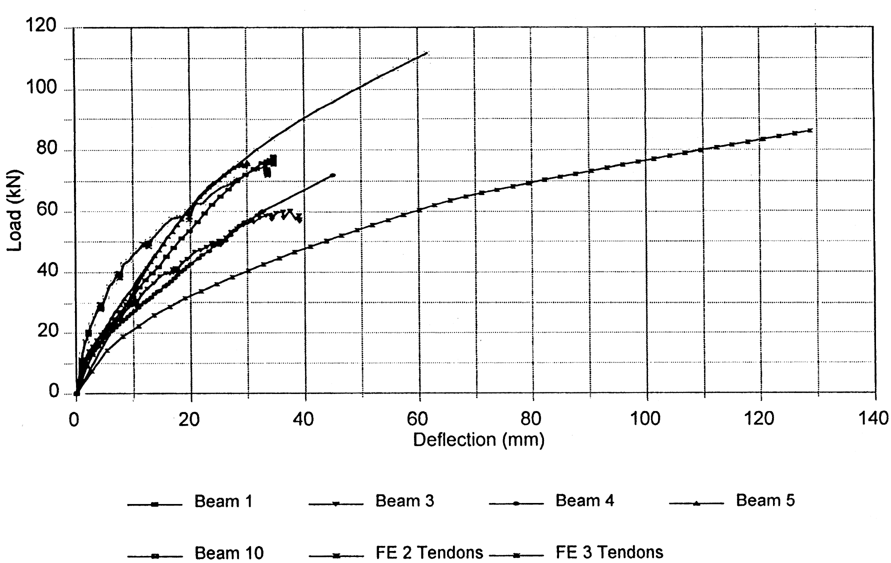

Figure 8.9 Experimental and numerical load deflection plots for beams with one layer of 1.0 mm thick plates.

Again, it is clear from the results that the numerical and experimental solutions are in close agreement, with the numerical results effectively bounding the experimental ones, with the exception of beam 10, which shows a stiffer response than the upper bound FE curve. This can be attributed to the fact that this beam was plated prior to coring and testing. Stresses from the cored tendons were transferred to the plate, thus pretensioning the plate. As a result the formation of cracks was delayed during testing, due to the prestress in the plate resulting in a stiffer initial response.

Experiments in the laboratory on smaller scale beams had highlighted the probability of peeling at the ends of unanchored short plates. This was not borne out in the large scale beams either experimentally or numerically, however beam 5 (6.0 m long by 1 mm composite plates) was anchored at its ends to cater for this possible eventuality. Plate strains at the ends of the unanchored 6.0 m long plated beam were much higher than the corresponding anchored beam, as the bolts were taking much of the shear force within this region. However, the unanchored plate did not peel off and the strengthening achieved was comparable. This end peel phenomenon on the beams tested in the laboratory could be due to scale effects, with the beams not having sufficient anchorage length in which to dissipate the large strains in the plate at the positions of cracks.

It was evident from all the tests carried out that coring significantly weakened the beams locally in addition to the removal of the tendons. All the beams which were cored prior to plating had vertical cracks at the core locations when loaded. Strain gauges were attached to the plates directly below the core holes and also midway between them. Figure 8.10 shows plots of plate strain versus distance from the plate for the numerical and experimental results for beam 5. It is clearly evident that at core locations, where discrete cracks are clearly visible, the strain is very high, dropping down to a much lower value between the cores. The figure shows two curves for the FE plots, one with the discrete cracks introduced directly into the model from site measurements and the other showing the original FE model with smeared cracks. The FE model with discrete cracks gives good correlation with the experimental results, but the FE model without the cracks appears to give reasonable average values for the stresses in the plate at various locations.

Beams 2 and 6 had two layers of plates 1 mm thick bonded to their soffits, the ends of beam 2 being unanchored and those of beam 6 being anchored. Beam 2 had composite plates 15.8 m long, whereas beam 6 had plates 6.0 m long. Figure 8.11 shows the load–deflection plots of the numerical and experimental results for these two plated beams. The numerical results for the two and three tendons of the beams are shown to bound beam 2, which was to be expected; this beam was cored prior to plating. Beam 6, which was

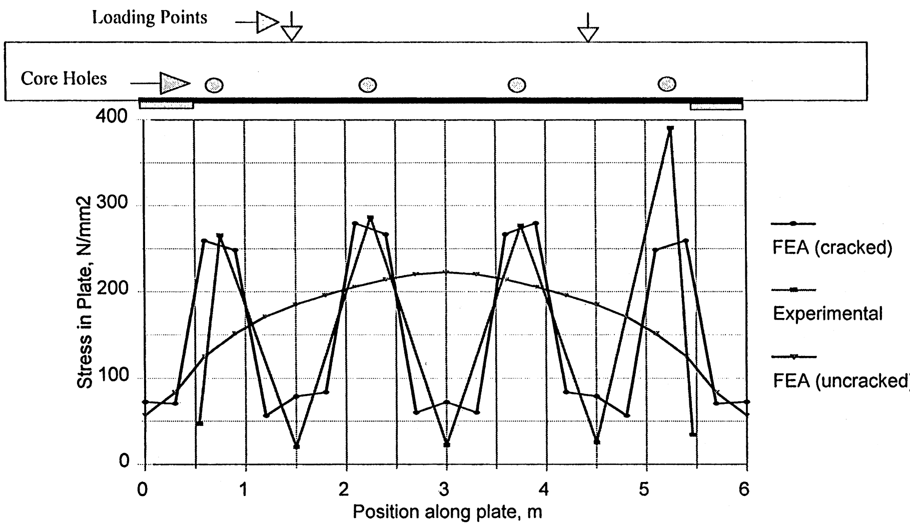

Figure 8.10 Plot of experimental and FE plate strains for beam no 5

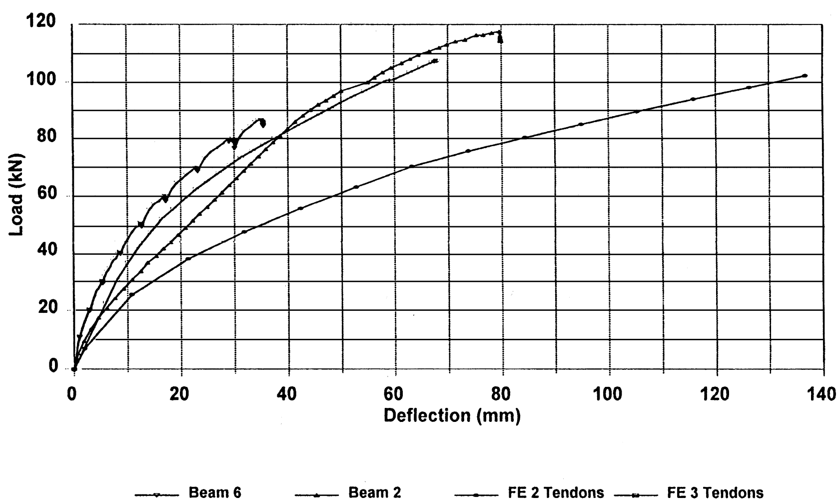

Figure 8.11 Experimental and numerical load deflection plots for beams with two layers of 1.0 mm thick plates

plated prior to coring, shows a similar type of response to that of beam 5 with regard to stiffness (Fig. 8.9); it shows clearly that the beams with short plates gave similar increases in strength and stiffness to beams with long plates. Figure 8.12 shows numerical and experimental load–deflection plots for beam 9, which was strengthened with three layers of 1 mm plates, 15.8 m long.

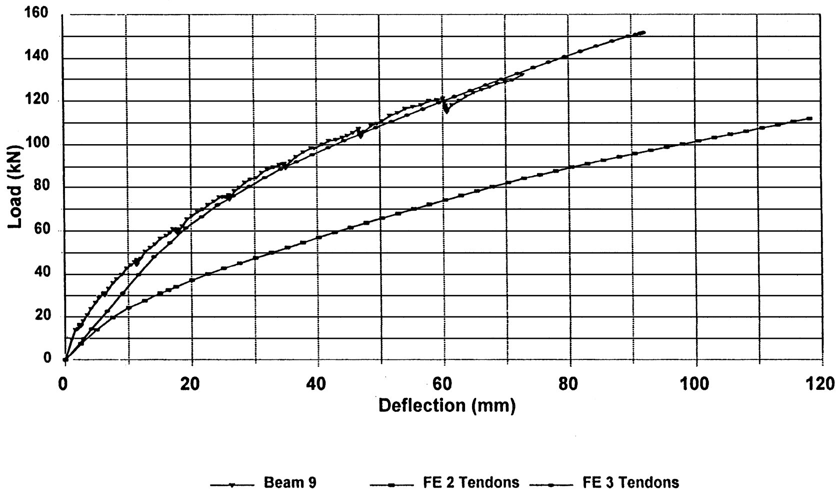

Figure 8.12 Experimental and numerical load deflection plots for beams with three layers of 1.0 mm thick plates.

Numerical analysis predictions gave very good correlation with the experimental results in all the beams strengthened with unstressed plates. Results also demonstrated that end anchorages were not required for any of the beams, another feature predicted by the FE analysis.

The majority of the beams were tested to 80% of their predicted ultimate capacity with two tendons intact, when unplated and subsequently when plated. Extrapolated results show that a single layer of 1 mm thick plates strengthened the beams by between 23–26%. The FE results predict a strengthening of 21–23%, much in line with the experimental results. This level of strengthening is sufficient to upgrade bridges from their current 38 tonnes load limit to the 1999 EC requirement of 40 tonnes.

FE and experimental predictions for beams with two layers of 1 mm plates showed increased strengthening in excess of 60% for the plated beam over the unplated beam, sufficient to strengthen bridges currently rated at 25 tonnes to the 40 tonnes EC directive. Beam 2 was actually loaded very close to failure, and failure was due to triangular flexural cracks, which caused local debonding of the plate over a 200 mm length near a core hole. Beam 9, which had three layers of composite plates bonded to its soffit, was also loaded very close to failure, with a similar type failure to beam 2, with an increase in strength of over 80%.

8.7 Beams with stressed plates

A method of stressing the plates was developed in the laboratory and is suitable for small span beams up to 8.0 m. The ROBUST project utilised experimental laboratory beams ranging from 1.0–4.5 m (see Chapter 5). In work undertaken by the University of Surrey, a number of 1.0 m, 2.3 m and 4.5 m beams were tested under four point loading in the laboratory with CFRP plates that were prestressed to 25%, 40% and 50% of their ultimate tensile capacity.

The following observations by the ROBUST team on beams prestressed with composite plates are:

1 The beams with plate prestress were clearly much stiffer than equivalent unstressed plates.
2 The action of stressing the plates is to reduce the onset of cracking, resulting in a stiffer member.
3 The in-service durability is enhanced due to reduced cracking.
4 Prestressing the plates can close cracks on structures.
5 The plate can be used as a safety net or reserve of strength against failure of tendons in prestressed beams as a result of corrosion.
6 The prestress also contributes to enhancing the shear capacity of the beam.

In summary, prestressing CFRP plates can have quite a beneficial effect on extending the service life of existing bridges and increasing their load carrying capacity in shear and bending significantly.

As the technique was shown to be viable in the laboratory, a purpose built prestressing device was designed and manufactured under the ROBUST project to be used to stress plates and bond them onto two of the ten 18.0 m prestressed beams on site at Kidlington. The device was capable of stressing one or two plates at a time, by anchoring one end of the plates and pulling from the other end. As each beam had three plates bonded side by side to the soffit, the sequence of operation was first to stress the middle plate, followed by stressing the two outer plates. The prestressing operation was implemented on site on beams 7 and 8 with three 90 mm wide by 1 mm thick plates stressed to 30% of their ultimate tensile capacity of 1500 MPa and then bonded onto the soffits of the beams. Beam 7, where the plates were 6.0 m long, had been cored prior to stressing and bonding, whereas beam 8, where the plates were 15.8 m long, was cored following this process, so that the transfer of stress from the tendons into the plates could be monitored subsequent to coring. One of the outer plates on beam 8 failed during the stressing operation at around 300 MPa. This was due to a manufacturing defect in the plate. In the event the beam was tested with only two plates.

Figure 8.13 shows load–deflection plots for beam 7 (6.0 m long composite plate) together with the corresponding FE plots for the beams with two and three tendons, respectively. The results suggest that this beam is initially

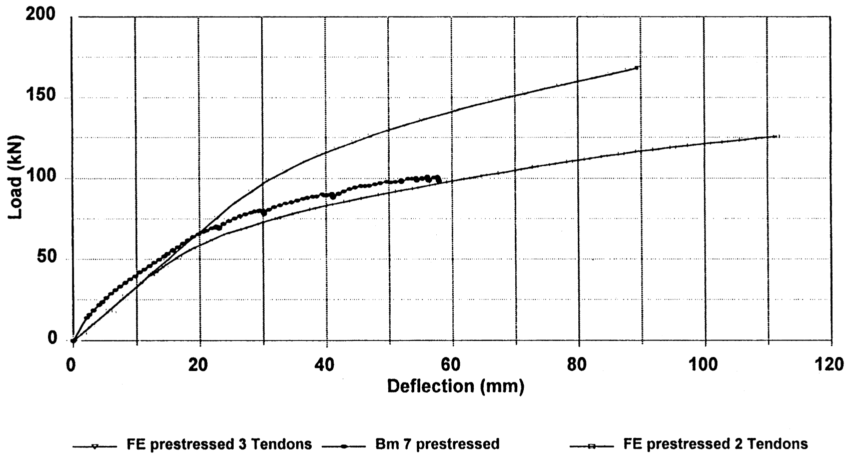

Figure 8.13 Experimental and numerical load deflection plots for beams with one layer of 1.0 mm thick prestressed plates.

stiffer than both the FE plots, but then tends towards the FE plot with two tendons. This could be attributed to the fact that as the prestressing device needed to be bolted to the vertical sides of the beam, drilling to accommodate the bolts damaged the two remaining intact tendons locally. This caused the steel to yield prematurely, further weakening the beam at this core location; a triangular crack distribution developed as the beam was loaded and this resulted in local debonding of the plate at this location. It is interesting to note that at the non-stressing end where a similar core hole existed, but with no bolting required for a stressing device, similar cracking did not occur, lending further credence to the possibility of damage to the uncored tendons. Figure 8.14 shows plots of stresses in the plate for the FE and experimental results. As mentioned previously, the FE models were modified to take account of the discontinuities at the core locations due to coring. The experimental curve shows peak stresses at the core locations, but reduced in value somewhat between cores. The core, which registered a peak stress of over 1000 MPa, was where the plate had debonded locally.

Numerical analysis predictions are in line with the experimental results, with the discrete crack model giving a similar curve to the experimental curve, and the smeared crack model giving a curve averaging the stress at these locations. This beam was tested very close to failure and strengthening of over 40% was achieved. This compares with 26% for an unstressed plate, highlighting the benefits of prestressing in terms of stiffness and strength.

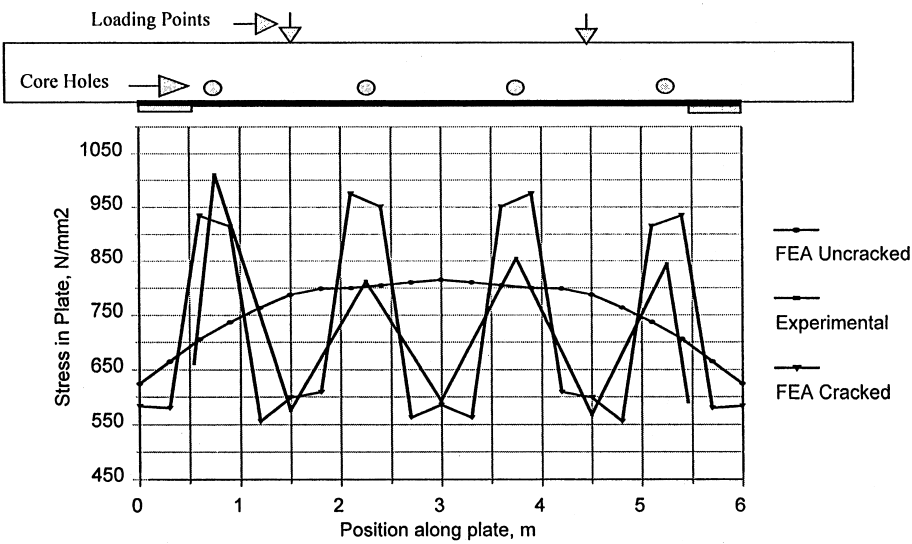

Figure 8.14 Plot of experimental and FE plate strains for beam no. 7.

8.8 Concluding remarks

Finite element methods utilising three-dimensional brick elements with a concrete material criterion have been used successfully in calibration of 1.0 m, 2.3 m and 4.5 m beams which were tested in the laboratory and have been employed to design subsequent tests on 18.0 m beams with encouraging results. Finite element methods utilising two-dimensional plasticity methods in ANSYS have proved to be unreliable as a predictive tool. Three-dimensional finite element methods have played a key role in developing design rules and guidelines for plate bonded solutions using advanced composite materials.

Accurate prediction of cracking of the concrete and yielding of steel reinforcement of a reinforced/prestressed concrete beam using the finite element method is not a simple task, but has been achieved satisfactorily under this programme.

Work done on small scale beams and on 18.0 m long beams with composite plates prestressed before bonding onto the beams has been shown to be viable both numerically and experimentally. Prestressing composite plates represents a significant contribution to the advancement of plate bonding practice. There is scope for increasing the shear resistance of reinforced concrete beams by bonding stressed plates to their soffits. Reduced cracking can thus enhance durability in service.

8.9 References

ANSYS (1995, 1996, 1997) *The Theory, Procedures and Elements Manuals*, ANSYS Inc, Houston, USA.

Deuring M (1993) *Post-strengthening of Concrete Structures with Pre-stressed Advanced Composites*, Report No. 224, CH-8600, EMPA, Switzerland.

Meier U (1987) 'Bridge repair with high performance composite materials', *J Mater. Technik* **4** 125–128.

Rots J G and Blaauwedraad J (1989) 'Crack models for concrete: discrete or smeared? Fixed, multi-directional or rotating?', *Heron* **34**(1).

William K J and Warnke E D (1975) 'Constitutive model for the triaxial behaviour of concrete', *Proceedings International Association for Bridges and Structural Engineering*, ISMES, Bermago, Vol 19, pp 174–190.

8.10 Bibliography

Fashole-Luke P S (1996) 'Strengthening RC and PC beams with advanced composite materials', *British Cement Association (BCA) Conference*, Sheffield.

Hutchinson A R and Rahimi H (1993) 'Behaviour of reinforced concrete beams with externally bonded fibre reinforced plastics', *5th International Conference on Structural Faults and Repair*, Edinburgh, pp 221–228.

Hutchinson A R and Rahimi H (1995) 'Flexural strengthening of concrete beams with externally bonded CFRP and GFRP plates', *Structural Adhesives in Engineering IV*, Institute of Materials, Bristol, pp 177–182.

Lane J S, Leeming M B and Fashole-Luke P S (1997a) 'Testing of strengthened reinforced and prestressed concrete beams', *Construct Repair J* **10** 10–13.

Lane J S, Leeming M B and Fashole-Luke P S (1997b) 'Using advanced composite materials in bridge strengthening: introducing project ROBUST', *The Struct Eng J* UK.

Peshkam V and Leeming M B (1994) 'Application of composites to strengthening of bridges: Project ROBUST', *Proceedings 19th International BPF Composites Congress*, Birmingham, 22–23 November, British Plastics Federation.

Peshkam V, Leeming M B and Fashole-Luke P S (1995) 'The role of finite element analysis in composite plate bonding', *Conference in Infrastructure Design and Rehabilitation*, NAFEMS Research Working Group, London.

9
Design and specifications for FRP plate bonding of beams

M B LEEMING AND J J DARBY

9.1 Introduction

The culmination of the research, the experimental testing in the laboratory, the modelling by numerical methods and the full scale beam testing on site, is the practical application of the technique to real situations. The first stage of this is calculating the extent of the strengthening required and determining the quantity of the plates to be bonded. The next aspect that concerns the designer is the specification of how the work has to be carried out. The practical aspects of undertaking the work on site have been covered in Chapter 10.

9.2 Practical design rules and guidelines

Throughout the design process the client has to be satisfied that the technique is sound and will give him the service life required. For techniques that have stood the test of time, a competent designer applies codes and design rules that have been formulated by experts and have had the approval of appropriate authorities. In the case of new techniques, however, authenticated design rules do not exist and an experienced designer must interpret the research carried out. It is necessary to understand the limits of the research and if there are areas where detailed knowledge is lacking, a conservative approach is necessary.

One of the objectives of the ROBUST programme of research was the development of practical design rules and guidelines for flexural strengthening of reinforced (RC) and prestressed (PC) concrete beams. These rules and guidelines were to be based on the experimental research and numerical verification carried out. The preliminary rules will need to be modified later by the results of further research and by taking account of the long term performance of actual structures. The accuracy of numerical verification has been demonstrated in Chapter 8 by comparing those results with the experimental ones reported in Chapters 4 and 5. The numerical methods used were developed for parameter studies of plate geometry and material properties and although the technique can be used for design it is generally considered by engineers to be expensive, time consuming and complex for practical design. Consequently, simpler methods based on empirical formulae which will be derived from the numerical and experimental results must be developed.

9.2.1 Failure modes

The first step in the formulation of design rules is to determine how the strengthened beam will fail under certain loading conditions and then to apply empirical formulae to predict that ultimate limit state of failure. The member may have to satisfy more than one ultimate failure mode. The application of suitable factors of safety will ensure that the failure mode is not reached in practice.

Three main failure modes have been identified in the literature and within the programme as follows (see Fig. 9.1):

- Flexural failure, modes 1, 2 and 3 in Fig. 9.1, occurring either in the carbon fibre reinforced polymer (CFRP) plate as tension failure, yield of the steel reinforcement in tension or in the concrete as a compression failure, the first being likely to occur when the beam is under-reinforced and the latter when over-reinforced. Yield of the steel reinforcement is likely to occur before either the concrete or the CFRP plate fails but while this may contribute to the ultimate failure of the beam it is not the prime cause of failure.

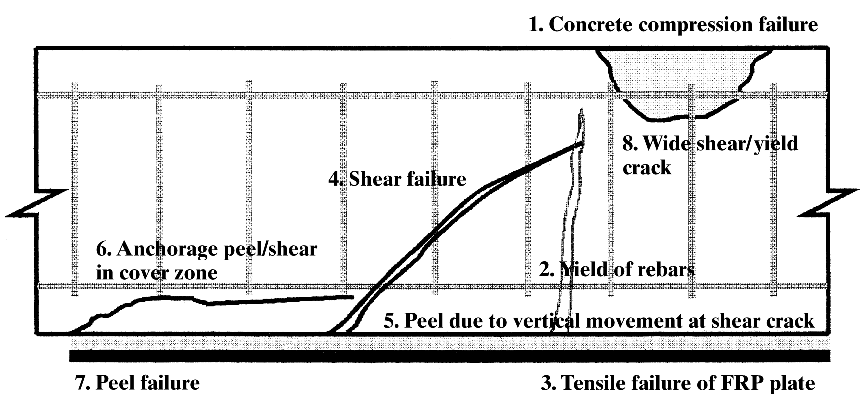

Figure 9.1 Typical failure modes for strengthened beams. Failure modes 1–8 on figure. Failure mode 8, adhesive failure at concrete/adhesive interface; failure mode 9, adhesive failure at adhesive/FRP plate interface; failure mode 10, interlaminar shear within FRP plate.

- End anchorage peel, modes 6 and 7 in Fig. 9.1. The abrupt termination of the plate can result in a concentration of stresses, some normal to the plate, which cause the plate to peel off towards the centre of the beam.
- Peel at a shear crack, modes 4, 5 and 8 in Fig. 9.1. This is a complex mechanism where debonding may occur due to strain redistribution in the plate at the crack and/or the formation of a step in the soffit of the beam causing shear peel. The delamination can then propagate towards the end of the plate. Whether mode 5 or 8 occurs depends on the amount of the shear reinforcement in the unstrengthened beam.

There are a number of other possible but unlikely modes of failure which have been identified in the literature such as delamination of the composite plate or in the glue line, but these have generally not been experienced in the research to date as the strength of these materials is higher than that of concrete. This type of failure could happen in practice if the installation has been less than perfect or there is a defect in the manufacture of the plate.

9.2.2 Failure in flexure

Meier (1987) states that with the substitution of appropriate properties, classical reinforced concrete beam theory can be applied when using advanced composite plates to strengthen beams in flexure. Strengthened full scale reinforced concrete beams tested by Kaiser (1989) at EMPA also seem to validate the strain compatibility method in the analysis of cross-sections. These statements were examined carefully in the light of the results from beam tests in the ROBUST programme. A major study was carried out using normal reinforced concrete theory to predict the load/deflection history, the deflection of the beams and the modes of failure of the beams.

In flexure, classical theory assumes that plane sections remain plane, that is, strain compatibility is assumed. Deflection of the beams can be calculated using simple formulae. Tracing the load/deflection history using normal reinforced concrete theory gives a reasonable fit to the experimental values, bearing in mind the inherent assumptions in the method. However, in most cases, while classical theory predicts that the plate will fail in tension, this phenomenon has not been experienced in any of the tests on beams with unstressed plates. The concrete compressive strain is usually limited to 0.35% for normal design purposes, whereas the value in the tests has been somewhat lower, in the range of 0.19–0.25%. Figure 9.2 compares the actual strains measured on one of the 2.3 m beams tested at Oxford Brookes University with the calculated strains at a load of 90% of the actual failure load in the test.

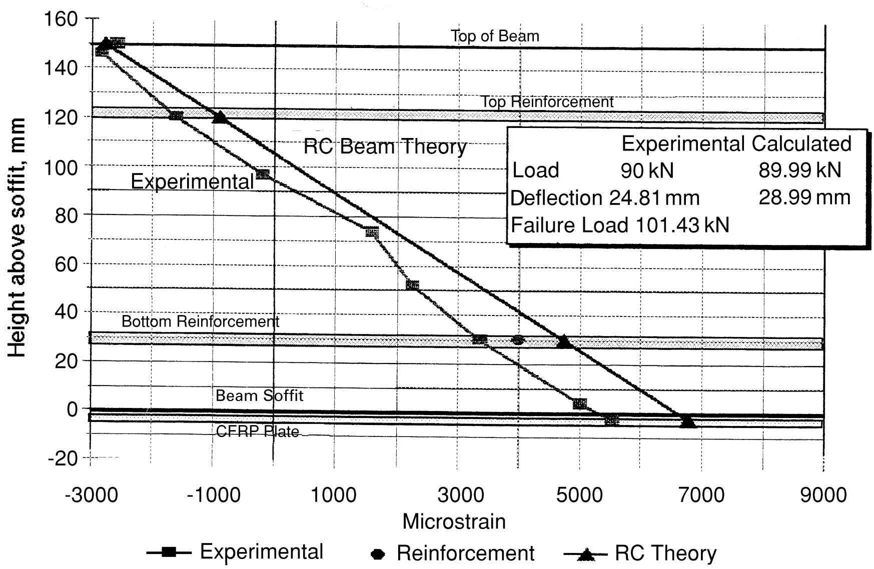

Figure 9.2 Comparison of measured strains on a 2.3 m beam with calculated strains from RC theory.

The strains on the concrete in the experimental work at Oxford Brookes University were measured by demountable mechanical (Demec) gauges on the side of the beams. It was not therefore possible to obtain readings at failure load. The correspondence between measured readings and those calculated becomes worse as the failure load is approached. This is probably due to the extensive cracking in the concrete that occurs in the tension flange and to the yield of the reinforcement. However, the prediction is on the safe side and will be more accurate at working load.

With the use of the classical reinforced concrete design theory and simple formulae for deflection, load deflection graphs can be drawn and the results compared with the experimental results. The correspondence is good with the calculated deflection following the changes in the slope of the experimental results as the reinforcing steel yields. The use of classical reinforced concrete theory is adequate for the purpose and has been found to predict stresses in strengthened beams better than the unstrengthened beams.

To apply reinforced concrete theory to strengthen beams it is necessary to know the ultimate strength of the plates and their modulus and use an appropriate material factor of safety. The ultimate strength of the plates and their modulus is obtained from the characterisation tests carried out on the plates. A material factor of safety, γ_m, of 1.5 is proposed for the ROBUST CFRP plates, based on properties derived from test specimens from the production of plates, assuming a pultruded plate fully cured at the works, and operating temperatures not in excess of 50 °C combined with a heat distortion temperature greater than 90 °C. With these parameters Eurocomp (1996) would advise that γ_m should not be taken as less than 1.5. This value can be compared with γ_m for steel of 1.15 and for concrete 1.5. The calculations can be carried out at the ultimate limit state, balancing the compressive and tensile forces which have been calculated from strain compatibility using appropriate modular ratios for the various materials.

9.2.3 End anchorage peel

The mechanism of end anchorage peel has been studied by a number of researchers: Jones *et al*. (1988, 1989), Swamy *et al*. (1989), Roberts (1989), Taljsten (1994), Jansze (1995), Jansze and Walraven (1996), Zhang *et al.* (1995), Raoof and Zhang (1996), Oehlers and Ahmed (1996). The theory of Jones *et al.* (1988) relates to relatively stiff steel plates and calculations within the programme have shown that stresses predicted by the theory are much reduced due to the thinner more flexible CFRP plates. These stresses are said to be due to:

1 The force resulting from making the plate conform to the curvature of the beam; this force is 1/370 th of an equivalent steel plate.
2 Plate curl due to the eccentricity of the plate force to the bondline; this force is 1/14 th of the equivalent steel plate.
3 Interface bond stress or longitudinal shear; this force is still important but is reduced due to the CFRP plate being thinner, since the closer the outermost fibre is approached the lower the longitudinal shear stress.

Swamy *et al*. (1989) suggested a design method of doubling the longitudinal shear stress and using an ultimate interface shear strength of $\sqrt{2}\times$ the tensile strength of the concrete.

The theory developed by Roberts (1989) is based upon partial interaction theory. Incorporating the results obtained from the 1 m beams into the theory, anchorage shear/peel stresses at failure in the order of 14–18 MPa are predicted. These are clearly not sensible, being considerably greater than the tensile strength of concrete.

Täljsten (1994) uses fracture mechanics for his predictions. Again the data obtained from the tests on 1 m beams at the University of Surrey and also from the 2.3 m beams at Oxford Brookes University were input into the theory. Anchorage shear/peel stress at failure in the order of 1.3–1.8 MPa are predicted for the 1 m beams and from 1.8–3.4 MPa for the 2.3 m beams which did not appear to fail by this mechanism. These results are more reasonable with respect to the expected shear or tensile strength of concrete although a value of at least double would have been expected for the 1 m beams.

Zhang *et al.* (1995) and Raoof and Zhang (1996) present a theory based on a series of cantilever teeth of plain concrete between cracks in the cover zone anchoring the plate and is limited by the tensile strength of the concrete. The spacing of cracking is difficult to determine but a minimum and maximum spacing is said to give an upper and lower bound solution which brackets the data.

A simplistic analysis was carried out by calculating an anchorage stress in a similar manner to the bond stress for a reinforcing bar by taking the force in the plate at the point of maximum moment at failure and dividing it by the length multiplied by the width of the plate beyond that point. Stresses from this exercise varied from 2.6–4.2 MPa which are of an order that might be expected although there is still a wide variation. Breaking these figures down into groups depending on plate width gives the results in Table 9.1.

None of the above theories presents an adequate method for general design. End peel stresses have been shown to be critical for thick and relatively stiff steel plates but it is thought that they are not significant for thin and flexible composite plates. There is no doubt that peel stresses occur at the end of a plate that is curtailed at a point some distance from the support but none of the above theories fitted the experimental results from the ROBUST programme.

The calculations for anchorage length suggested by European thinking are based on tests carried out when a CFRP plate is bonded to a concrete block and a force applied to pull the plate from the concrete in a direction parallel to the bond line. The tests are often done as a double shear lap test. In these tests the peel is initiated not at the end of the plate but at the front face of the concrete. The peeling mechanism is not therefore due to end peeling stresses but to high longitudinal shear stress at the leading edge of the concrete. This condition would be closer to the situation at the edges of a crack. The failure mode is considered to replicate FRP peeling off at the outermost crack in the uncracked anchorage zone. Calculations based on fracture mechanics give good agreement with experimental results. Usually when a plate peels under these conditions only a thin layer of concrete a millimetre or two in thickness adheres to the plate but in some cases its failure line may divert into the plate causing interlaminar plate failure.

Table 9.1 Anchorage stresses for the 1 m beams

Plate width (mm)	90	65	45
Plate thickness (mm)	0.5	0.7	1.0
Maximum stress (MPa)	2.62	3.34	4.22
Minimum stress (MPa)	2.53	3.09	3.99

Rostasy (1993) looked at this situation and cites work by Ranish (1982) who studied bond stresses in the anchorage length of steel plates bonded to concrete blocks. It was found that a maximum value of bond stress (max τ_{b1}) was reached which equalled four times the mean surface tensile strength of the concrete established by a pull-off test. The necessary anchorage length (l_{bl}) was found from equation [1]:

$$\text{nec}\, l_{bl}\ 5\ F_{lu}^2 / (E_1 . t_1 . b_1^2 . \max{}_{bl} . 15.10^{25}) \tag{1}$$

where l_{bl} is between 500–1500 mm, F_{lu} is the limiting plate force, E_l is the modulus of the plate (MPa), t_1 is the thickness of the plate for values between 5–15 mm and b_1 is the width of the plate (about 150 mm). This formula was introduced into early design procedures for CFRP plates where the thickness of the CFRP laminate was expressed as the thickness of an equivalent steel laminate in the ratio of their respective strengths. The max τ_{bl} was also given in a table with values according to Tausky (1993) based on the measured tensile strength of the concrete at the surface. This theory assumed a minimum bond length of 500 mm and the force to be anchored was the tensile force in the CFRP laminate at the design level in the location of the maximum moment.

Neubauer and Rostasy (1997) carried out 51 bond tests using CFRP plates bonded to concrete blocks where the bond length, plate width, plate thickness and concrete cube strength were varied. It was found that anchorage lengths of between 195–330 mm were required to sustain a plate force that could not be exceeded and that longer anchorage lengths were not effective. Fracture mechanics was used to analyse the results which were expressed in the formulae [2]–[4]:

$$l_{t.\max} = \sqrt{}(E_1 . t_1)/(2 . f_{ctm}) \tag{2}$$

$$T_{ck.\max} = 0.5 . k_b . b_1 . \sqrt{}(E_1 . t_1 . f_{ctm}) \tag{3}$$

$$\text{where } k_b = 1.06 . \sqrt{}((2 - (b_1/b))/(1 + (b_1/400)) \geq 1 \tag{4}$$

where $l_{t.\max}$ is the maximum bond length in mm, E_1 is the modulus of the plate, t_1 is the thickness of the plate in mm, f_{ctm} is the concrete surface tensile strength in $N\,mm^{-2}$ which should not be taken as greater than $3\,N\,mm^{-2}$, $T_{ck.\max}$ is the maximum plate force that can be anchored in Newtons, b_1 is the plate width in mm and b is the width of the beam soffit or the plate spacing in slabs in mm.

Chajes *et al.* (1993) first investigated adhesives with moduli from 5–0.2 GPa with various types of surface preparation in similar tests. The adhesive with the lowest modulus failed in the adhesive while the others failed in the concrete. Fusor together with a Chemglaze primer (an organofunctional silane) gave the best results and was chosen as the adhesive for further tests where three different concrete strengths were used. These few results were used to predict a shear stress failure in the concrete of 11.1 $\sqrt{f_c}$ psi, where f_c is probably the cylinder strength although not stated (0.92 $\sqrt{f_c}$ in MPa). Further lap shear tests were then carried out using 50 mm, 100 mm, 150 mm and 200 mm bond lengths with a 36.4 N mm^{-2} concrete. These tests showed that a maximum force of 12 kN could be anchored by a 25 mm wide strip with a 95 mm anchor length and that longer anchorage lengths did not increase the force that could be anchored. An average shear stress in the concrete was estimated at 4.945 N mm^{-2}. Chajes *et al.* (1993) therefore came to the same conclusion as Neubauer and Rostasy (1997) that there is a certain anchorage length that will sustain a plate force that cannot be exceeded. Table 9.2 compares the results from these two papers.

Table 9.2 Comparison of anchor lengths

	Neubauer and Rostasy (1997)				Chajes *et al.* (1993)			
Adhesive	Sikadur 31				Fusar			
Tensile strength (MPa)	24.8[1]				30.6			
Elongation (%)	0.4[1]				3.0			
Elastic modulus (GPa)	5.172[1]				1.584			
CFRP Plates	CarboDur				Prepreg			
Tensile strength (MPa)	2000–3000				1655			
Elongation (%)	3–5				1.5			
Elastic modulus (GPa)	150–230				108.5			
Results	Concrete strength (MPa)	Plate thickness (mm)	Anchor length (mm)	Plate force (kN)	Concrete strength (MPa)	Plate thickness (mm)	Anchor length (mm)	Plate force (kN)
	25	1.2	228					
	25	2.4	330					
					36.4	1	95.25	12
	55	1.2	194					
	55	2.4	275					

[1] Figures from Chajes *et al.* (1993) for Sikadur 31.

Detachment of the plate was a common failure mechanism in the ROBUST tests. However, the plate became detached so quickly that there was always doubt about whether the failure started at the end of the plate and propagated towards the centre or started peeling at a crack and propagated towards the end of the plate. Even recording the point of failure with a high speed video camera could not resolve the question. It is now thought that peel initiating from a shear crack is the more likely mechanism.

9.2.4 Peel at shear or at wide shear/yield crack

Where the shear span to effective depth of the beam is less than about 6, the normal mode of failure of a reinforced concrete beam is usually due to the formation of a diagonal flexure–shear crack. This effect is illustrated in Fig. 9.3 where the results of the tests on 1 m long beams at the University of Surrey are presented. The effect of the width of the plate is evident, the wider the plate the greater the ultimate moment. At low values of shear span to depth ratio, effective anchorage of the ends of the plates also increased the ultimate moment. The shear span to depth ratio is related to

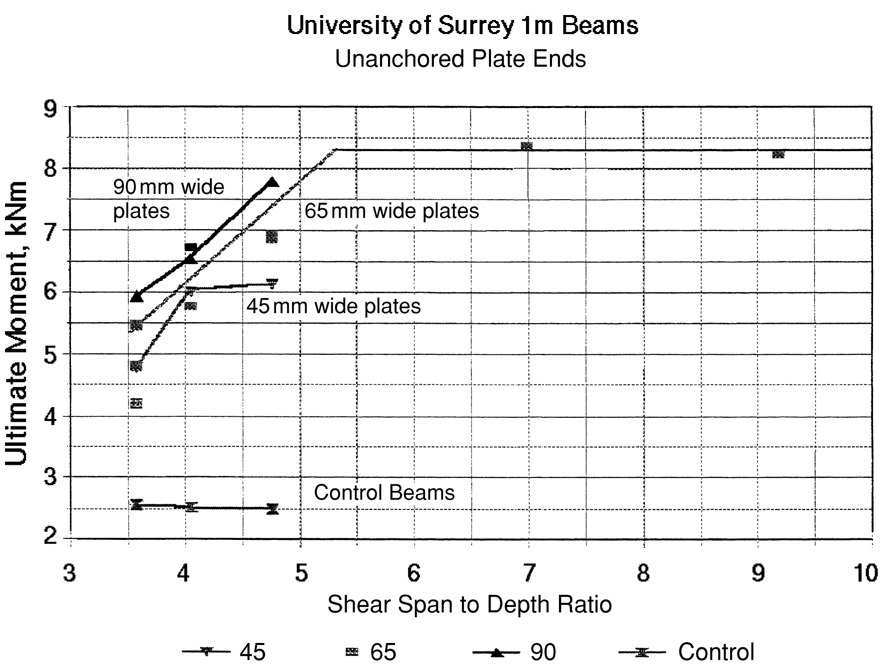

Figure 9.3 Effect of shear span to depth ratio and plate width on ultimate moment of beams strengthened with CFRP plates.

the longitudinal shear stress between the plate and the concrete but with the scatter of the results a lower bound longitudinal shear stress could not be assessed. Whether this reduction in ultimate strength was due to shear or end peel effects could not be ascertained. It is clear that where the shear span to depth ratio is low, calculating the required amount in strengthening by flexure alone is not sufficient and shear and end peel effects must be considered or the plate ends must be anchored adequately. At a point when a crack occurs at the end of the plate due to shear and bending, end shear peel is likely to occur and the whole depth of the cover concrete together with the plate will detach and peeling will occur at the level of the reinforcement.

If a shear crack in a beam strengthened with a CFRP plate approaches an angle of 45°, a step can occur in the soffit of the beam as the end of the beam rotates about the compression zone of the top flange (see mode of failure 4 in Fig. 9.1). This failure mode was identified by Meier and Kaiser (1991). This mechanism can cause the plate to peel due to stresses normal to the plate and may be aggravated by stress redistribution across the crack. Triantafillou and Plevris (1992) address the problem and propose the relationship $P = \lambda(G_s.A_s + G_p.A_p)$ where P is the debonding load, G is the shear modulus, A the area of the main reinforcement and the plate, respectively and λ is a constant determined by experiment. From the three experiments they found that λ approximates to 0.011 using a value of 4.4 GPa for the shear modulus of the CFRP plates.

Many failures within the ROBUST programme are now thought to have initiated from a near vertical crack at a point about a beam depth away from the load point associated with triangular cracking of the concrete followed by peeling towards the end of the plate. The vertical crack occurred between the shear link reinforcement. The beams were heavily reinforced in shear to avoid shear failure of the type described above. A possible mechanism for failure mode 8 in Fig. 9.1 is the formation of a wider crack at the point where the steel reinforcement yields, leading to debonding of the plate to redistribute the high strains across the crack which then propagate towards the end of the plate. This mechanism may be aggravated by some small vertical movement at the soffit due to shear strain across the crack. The factors influencing the failure will be:

- shear transfer in the compression zone of the concrete,
- shear transfer due to aggregate interlock in the crack below the neutral axis,
- dowel action of the steel reinforcement,
- dowel action of the CFRP plate, although this must be small and should be discounted,
- a shear step in the soffit of the beam causing plate peel. This latter

mechanism is dependent on the angle of the crack and the extent of the shear strain across the crack,
- the formation of the wide crack due to yielding of the steel reinforcement.

The main objective of the programme was to study CFRP plate bonding in flexure but it is clear that with low shear span to effective depth ratios used in some of the experiments, the mode of failure was a shear peeling causing the removal of the layer of concrete cover to the internal tensile reinforcement. All the beam tests in the programme were carried out under four point bending. This loading configuration gives constant shear in the shear span with a high bending moment at the loading point that diminishes to zero at the support. This form is commonly used in loading tests as it leads to simple analysis but is not typical of loading in practice where uniformly distributed loads are more common and point loads, when they occur, are frequently moving such as in wheel loads. With uniformly distributed loading there is no clear shear span, and anchorage and shear effects may be less pronounced. Shear effects are not well understood in reinforced concrete theory and the relative effects of shear transfer in sound concrete, aggregate interlock in cracked concrete and the dowel action of the reinforcement are difficult to quantify separately. Taylor (1974) gives the following proportions of the shear force taken by the above actions: compression zone shear 20–40%, aggregate interlock 33–50% and dowel action 15–25%. Regan (1993) gives a more recent overview of research into shear over the last 100 years. It is not thought that the dowel action of the CFRP plate is significant. The extent of a shear step in the soffit of the beam is difficult to quantify and is also probably of little significance. The situation will be on the safe side if the beam satisfies the normal code requirements for shear ignoring the contribution of the CFRP plate.

The stresses in the plates and in the steel reinforcement can be calculated throughout the length of a beam according to classical RC bending theory on the basis that the stresses vary in accordance with the moment at any point. This is illustrated in Figs. 9.4 and 9.5 for the half spans of two beams, one with a single point load and the other with uniformly distributed loading. The case of the single point load is similar to that of the shear span only of a beam loaded in four point bending as is usual in experimental beam tests.

Longitudinal shear stress between the plate and the concrete will be related to the rate of change of the force in the plates and hence to the stress in the plate for constant section. It can be seen from Fig. 9.4 for the case of a single point load that the greatest slope on the curve of the stresses in the plate is towards the middle of the beam where the steel reinforcement has yielded and can no longer contribute any greater force to balance the

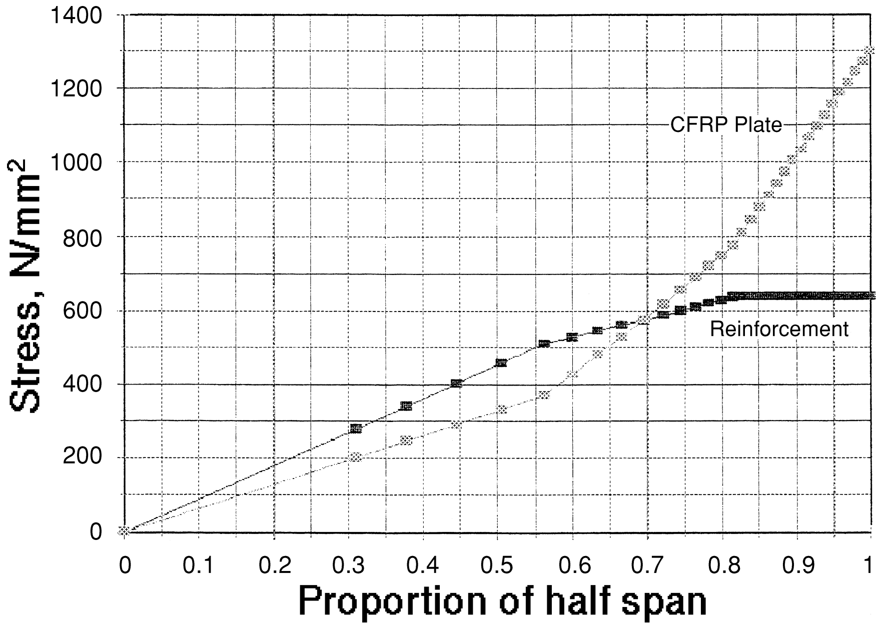

Figure 9.4 Tensile stresses under a single point load.

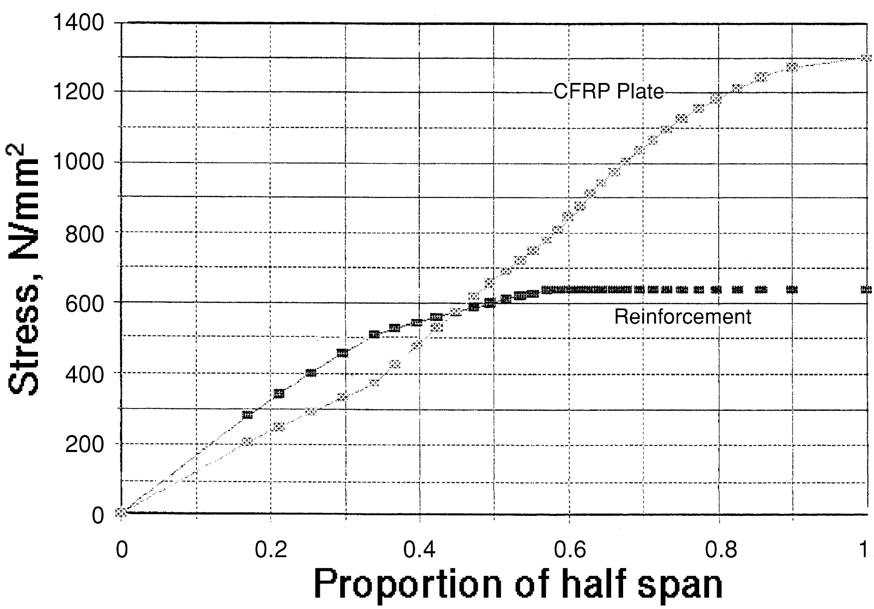

Figure 9.5 Tensile stresses under uniformly distributed load.

increasing moment. The longitudinal shear stress, at this point, can be shown to be approximately equal to $S/(by)$ where S is the shear force, b the width of the plate and y the distance from the middle of the concrete stress block which is effectively the distance of the centroid of the plate from the top of the beam less half the neutral axis distance. For the case of a uniformly distributed load the greatest slope is between about 0.6 and 0.7 of the half span. The former case is a coincidence of high moment and shear; the latter of high moment but low shear.

Combined with the above, higher stresses will occur in the CFRP plate at the location of the crack due to the increase in strain over the width of the crack. This instantaneous high stress can only dissipate by debonding either side of the crack, possibly causing a triangle of concrete to detach from one side of the crack. This phenomenon was found in the testing of the 18 m beams where the strains in the plates were found to be very high at the points where the prestressing cables had been cut by coring through the side of the beam. These strains were found to be 1.25 to 1.5 times the average strains. The debond will release the peak stress to a point where the rate of change of stresses in the plate no longer exceeds the longitudinal shear stress that the concrete can withstand.

Once the length of the debond is sufficient for the difference in the forces in the plate on either side of the debond to exceed the maximum longitudinal shear stress of the concrete, the debond will propagate further and will ultimately reach the end of the plate. The three factors involved in failure mode, namely, high longitudinal shear stress at the interface between the plate and the concrete, stress redistribution in the plate over the crack and shear step effects, do not lend themselves to simple analysis for design purposes. It is suggested, therefore, that in the design the strain in the plate is calculated at the point where the internal steel reinforcement starts to yield and the ultimate load for this failure mode should then be taken as the point of yield strain plus an additional limiting strain. The additional limiting strain has yet to be determined from experiment.

9.2.5 Limit states and partial safety factors relevant to composite plate bonding

The design of strengthened bridges should be considered for both the serviceability limit state (including the checking of stress limitations) and ultimate limit state, in accordance with the relevant clauses of BS 5400: Part 4 (1990), except where amended in this chapter. The appropriate loads and load factors should be taken from BS 5400: Part 2 (1990).

Partial safety factors to the characteristic strength of the CFRP plates of $\gamma_m = 1.5$ for ultimate limit state (ULS) and $\gamma_m = 1.00$ for serviceability limit state (SLS) should be used. Where the characteristic strengths of the existing concrete and reinforcement are not known characteristic strengths may be derived from test evidence.

The 'over-reinforcement' of a concrete section can result in a brittle failure. Sections to be strengthened should, therefore, be checked to ensure that this does not occur, in particular by using the requirements of Clause 5.3.2 of BS 5400: Part 4 (1990).

9.2.6 Plate dimensions and spacing

When determining the size and thickness of each individual plate for strengthening a slab, the effective width of the section should not be greater than the width of the plate plus twice the distance of the plate to the neutral axis of the section.

9.2.7 Fatigue and creep

Fatigue is discussed in Chapter 7, Part 2 'Fatigue Behaviour'. The stress range for fatigue purposes in the existing steel reinforcement of reinforced concrete beams should not be increased when the beam is strengthened by CFRP plate bonding. In prestressed concrete it should be ensured that stress transfer due to creep in the concrete will not result in excessive compressive forces being induced into the plates.

9.2.8 End and intermediate fixings

Intermediate fixings are not required as is the case with steel plate bonding as the plates are much lighter. The 'grab' of the adhesive is such that the plate remains in position after having been rolled into the adhesive layer without further support.

Fixings may be required at the ends of the plates to resist concentrations of peel and shear stresses in this region, where these stresses exceed the product of the pull-off strength of the concrete and γ_m. The fixings should be designed as described above. In the ROBUST programme of research, 15 mm thick $\pm 45°$ glass fibre reinforced polymer (GFRP) end tabs were specially bonded to the ends of the plates and stainless steel bolts were passed through both the plate and the end tab at suitable spacings and were anchored at a depth into the concrete below the steel reinforcement. The bolts should be supplied with large washers and tightened up to a predetermined torque to prevent crushing of the composite materials. It is not adequate to omit the end tabs and merely to drill through the composite plates and thence to anchor them at their ends with bolts. Drilling holes through unsupported composites will sever the unidirectional fibres and the concentrated compressive forces under the bolt head will weaken the plate further; it is not possible for the forces in the plate to be transmitted into the bolt. The bond between the ROBUST end tabs and the plates was designed to take the whole force to be anchored. This force is then transferred into the concrete by the fixing bolt. Other forms of fixing were not found to be so effective. Tests by Poulsen (1996) show that the ultimate anchorage force can be increased by up to three times by utilising three bolts, but increasing the number of bolts beyond this value is not effective. The effect of bonding short lengths of laminate across the end of the plate was also examined. One transverse laminate increased the ultimate force anchored by 75% but further transverse laminates gave little increase in anchorage force.

Plates should not extend into areas of compression without further verification about their adequacy, as plate buckling may occur causing tensile/peel stresses in the adhesive normal to the plate. The maximum compressive force in the plate is limited by the maximum permitted strain in the concrete at compressive failure, normally taken for design purposes as 0.0035. Where the plates are fully bonded they can be considered fully restrained against buckling. The out-of-plane forces can be calculated and even an out-of-straightness of 5 mm in 1 m produces out-of-plane forces that are well within the tensile strength of the concrete which is the weakest link. However, an unbonded length as short as 40 mm for a 1.4 mm thick plate could be liable to buckling failure. Clearly the length of voids in the bondline must be limited to well below the critical buckling length dependent on the thickness of the plate. During construction, very careful monitoring of voids in the bondline is required for sections of plate that will go into compression during any part of the loading cycle. This is an area that would benefit from further research.

9.2.9 Vandalism and fire

Where fire damage and vandalism are expected the plates may be covered by a suitable polymer-modified cementitious screed or by suitable intumescent coatings. Fire tests have been carried out at EMPA where CFRP plates were found to last considerably longer than steel plates. The reason for this is that the adhesive is the most vulnerable factor and steel conducts the heat rapidly to the glue layer. Carbon does not conduct heat to the same extent as steel and to some extent protects the glue layer. Composite materials are being increasingly used in offshore structures as panelling for blast and fire protection (Wu and Gibson, 1994).

9.2.10 Durability

At present there are no standard accelerated laboratory testing methods to predict the long term performance and durability of the system. The durability of composite plate bonding is discussed in Chapter 6.

9.3 Application of the technique

9.3.1 General

The technique of bonding CFRP plates to structures can be used in all locations where there is a requirement for additional flexural reinforcement in the tensile zone. Their use for strengthening in compression requires further justification. Careful consideration should be given where the headroom of bridges is critical or where there is evidence of frequent damage to the soffit from vehicles. Special provisions may be necessary to protect the CFRP plates under such circumstances. Where plating to soffits of bridges above carriageways is being considered the available headroom should be checked. Due allowance should be made for any fixings required.

The Highways Agency (1994) requires that, for steel plate bonding, any element of a bridge structure to be strengthened should be capable of supporting nominal dead load, superimposed dead load factored by the partial safety factor for loads, $\gamma_{fL} = 1.2$ and nominal HA (normal) live load (i.e. unfactored) when checked at ULS. There seems no reason to alter this requirement for CFRP plate bonding at present.

Where plates are bonded to the top surfaces of slabs and beams and subsequently buried by the road surfacing it would be impractical to provide inspection facilities for the plates. In such cases, special care should be taken during bridge inspections to identify any areas of plates that may have debonded as indicated by local breakup or reflective cracking of the surface in the location of the plates. It is essential that accurate drawings indicating the location of all plates are maintained and are readily available for such inspections.

9.3.2 Investigations and tests

Before plate bonding is considered for a structure, investigations should be carried out to ensure that the risk of corrosion in the existing member is low and that the structure is sound enough (including any repaired areas) for strengthening by plating.

Surfaces that are damp or subject to leakage, particularly if contaminated with chlorides, should only be plated after satisfactory remedial measures have been taken. If the reinforcement is corroding, the expansive rust products may disrupt the concrete and eventually cause debonding of the plate. Therefore, unless repairs have been carried out, plate bonding should only be considered for members where chloride values are generally less than 0.3% by weight of cement and half-cell potential measurements are numerically generally less than −350 mV (e.g. −200 mV) with respect to a copper/copper sulphate electrode. CFRP plates are less vulnerable to durability problems than steel plates and therefore surfaces that are damp and subject to leakage are less critical. However, steps should be taken to rectify the dampness and prevent the leakage. Tests in ROBUST carried out by Oxford Brookes University showed that there was no loss in pull off strength when Sikadur 31 was used to bond the dollies to saturated surface dry concrete. Nonetheless, it is advisable to avoid these conditions to ensure that the highest standards are used in installation of the technique. While dampness may not affect the composite plate and adhesion to the concrete significantly, it could cause further corrosion of the steel reinforcement leading to spalling of the cover concrete to which the plate may be bonded and would also be susceptible to freeze/thaw damage causing delamination of the concrete cover.

9.4 Materials

9.4.1 Composite plates

A variety of composite plates have been used in the ROBUST and other research programmes, using glass, carbon or aramid fibres manufactured by the pultrusion process or made up from prepreg material. In the UK and in Europe the use of plates containing about 60% by volume fraction of unidirectional carbon fibre manufactured by the pultrusion process in an epoxy or a vinylester resin matrix is favoured. The plate in ROBUST was 90 mm wide by 1 mm thick and used T300 carbon fibre in a vinylester resin (BASF A430) combined with a peel-ply inbuilt surface finish on both sides of the plate. This method of surface finish provided a suitable surface for bonding without an extra manufacturing process and kept the surface to be bonded clean until just before the adhesive was applied. The choice of T300 fibres was made on cost and availability at the time with a penalty of a lower strength and modulus. T700 fibres are much more widely available now and there is no reason why this superior strength of fibre should not be used in plates of different widths and thickness. Other plates from Europe have used T700 fibres and are available in various sizes but require abrading and wiping clean before use to obtain a satisfactory bonding surface.

The modulus of glass is too low to give satisfactory performance in plate bonding requiring at least three times the thickness of an equivalent CFRP plate to give the same stiffness and is therefore not being used economically. Aramid has no special advantages over carbon and is more expensive.

The use of any plating material must have adequate data to support the quoted properties resulting from characterisation tests carried out in accordance to appropriate standards such as BS2782 Part 3 (1996) and Part 8 (1994), ASTM D3039/3039M (1995) and Crag (1988). These standards require a test sample 25 mm wide. Tests have been carried out on full width samples which gave significantly lower results. This size effect must be resolved before reliance can be placed on results from coupon specimens. The material factor of safety used in design should reflect this uncertainty and will depend on the number of tests carried out and their variability.

9.4.2 Concrete – suitability for bonding

A range of tests is available to determine the suitability of the concrete surface for bonding plates. The most suitable test method involves bonding a metal dolly to the concrete using the same adhesive used to bond the plate, and subsequently pulling the dolly off. Examination of the failure mechanism and failure load gives useful guidance. The preparation is considered suitable when a cohesive failure occurs in the concrete. This test should be carried out in accordance with BS 1881 (1992) or with the latest edition of prEN 1542 (awaiting translation before ratification). A minimum value of $1.5\,\mathrm{N\,mm^{-2}}$ should be obtained.

9.4.3 Adhesive

On the basis of long term experience epoxy resin adhesives have been found to be suitable for steel plate bonding. Their durability has been established by use over a period of twenty years. The same epoxy resin adhesives have been used in research and in practice for CFRP plate bonding with acceptable results. An adhesive must demonstrate an acceptable track record of use in steel and CFRP plate bonding or must be subjected to additional tests to prove its acceptability for strengthening proposals detailed below.

Epoxy resin adhesives require care in use. Manufacturers or formulators commonly supply two-part resins in containers suitably proportioned for mixing. It is important that all the hardener is added to the resin in its container and mixed with a slow speed mechanical mixer. High speed mixing entrains air and is less efficient. The resin and hardener should be of different colours and adequately mixed to produce a uniform colour. The speed of the chemical reaction increases with the temperature generated.

Sikadur 31PBA was used in the ROBUST programme of research. This is a two-part cold cured, epoxy-based structural adhesive composing a resin which is white in colour and a black coloured hardener. They should be mixed together to form a uniform grey colour. The adhesive should be applied onto the CFRP and/or concrete surfaces within 20 min. Sikadur 30 has been used more extensively in Europe. The properties of Sikadur 31PBA and Sikadur 30 are given in Table 9.3.

Table 9.3 Properties of typical adhesives used in plate bonding

	Sikadur 31PBA	Sikadur 30
Heat distortion temperature (°C)	43	62
Flexural modulus ($kN\,mm^{-2}$)	8.6	12.8
Tensile strength ($N\,mm^{-2}$)	21.8	
Moisture resistance	<0.5% after 28 days	
Density ($kg\,m^{-3}$)	1500	

9.5 Workmanship

9.5.1 Surface preparation of concrete surfaces

It is important that preparation of the concrete substrate is carried out well to ensure an adequate bond with the adhesive. Minor protuberances and imperfections can be removed by scabbling or grinding the surface. Grit blasting is the preferred method of general surface preparation and should be carefully carried out to provide a laitance and contamination free surface resembling coarse grained sandpaper with exposure of minor aggregate. Consideration should be given to filling any cracks or fissures (including open construction joints) wider than 0.2 mm or liable to leakage, by injection of a suitable, compatible low viscosity resin. The prepared surface should be dust free and surface dry. If moisture is picked up by absorbent paper pressed onto the concrete it is likely to be too damp for bonding. It may be necessary to provide temperature and humidity control to dry out the concrete sufficiently prior to bonding.

9.5.2 CFRP surfaces

The ROBUST CFRP plate has an in-built peel-ply surface treatment installed as part of the pultrusion process. The peel-ply provides a clean surface, free from contaminants such as release agents, which is wettable and has an appropriate surface texture with a freshly fractured matrix resin surface with a surface free energy value in excess of $55\,mJ\,m^{-2}$. The peel-ply sacrificial surface layer is removed immediately before application of the adhesive. Plates which do not have this in-built peel-ply surface treatment should be abraded and cleaned and degreased with a suitable cleaning agent such as Sika Colma Cleaner. The plates should be maintained in a dry condition before application of the adhesive.

9.5.3 Application of adhesive

The bonding of CFRP plates to concrete members is carried out by applying the structural epoxy resin to the plate and/or concrete, offering the plate to the soffit of the member and applying pressure to ensure intimate contact with the concrete. It is important to spread the adhesive immediately after mixing to dissipate the heat generated and extend its workability time. Common practice is to spread the adhesive slightly more thickly in a domed profile along the centre line of the plate than at the sides of the plate. This reduces the risk of forming voids when pressing the plate loaded with adhesive against the concrete surface. Excess adhesive squeezed out of the plate can be scraped away and a fillet formed at the plate edge. Small glass beads (ballotini) dropped onto the epoxy adhesive surface prior to fixing can be used to maintain the minimum required adhesive thickness of 1–2 mm. Procedure trials should always be carried out to prove the method of application and acquaint the operatives with the material. The ambient temperature and dew point temperature should be determined and if the ambient temperature is less than 10 °C or dew point is more than 10 °C, artificial heating and dehumidifiers may be required to maintain the ambient temperature and humidity at acceptable levels for installation and curing. The manufacturer’s instructions on safe use of resins should be followed.

Based on experience in ROBUST up to three plates may be bonded in layers to give the required total thickness, each plate being bonded separately.

9.6 Quality control

9.6.1 Adhesive bond/shear tests

In bonding to CFRP, assurance of the quality of adhesive bonded joints should take place prior to bonding by ensuring adequate control of all the principal steps in the bonding process; these steps include surface preparation, mixing, application and curing of the adhesive. The purpose of the test is to check adhesion and the adhesive material properties. Simple single lap shear tests are recommended, with a 10 mm overlap, see below. Three single lap shear tests should be performed for each batch delivered to the site. The CFRP adherends should be prepared in the same manner as the plates to be bonded. The load and locus at failure is to be recorded and reported and the test should be deemed acceptable if the bond strength of the adhesive to the CFRP is greater than 12 N mm^{-2}.

It is recommended that representative samples of the prepared surface be used to make single lap shear joints generally in accordance with ASTM

D3163 (1992). The dimensions and arrangement of the test are shown in Fig. 9.6. The failure load will indicate whether or not the adhesive material has been mixed and cured adequately, while the locus of the joint failure can provide information on the soundness of the bond. Interfacial failure and low joint strengths indicate poor adhesion, whilst failure of the joint in the composite material itself can at least provide some lower bound confidence level in bond performance.

An alternative to the single lap joint test is the wedge cleavage test illustrated in Fig. 9.7 and carried out generally in accordance with ASTM D3762 (1979). The purpose of the test is also to assess adhesion and toughness of the adhesive material. The procedure involves first bonding together two strips of composite 150 mm long and then, once the adhesive has cured, driving a wedge between them to provide a self-stressed specimen. This induces a cleavage stress on the bondline and if adhesion is poor a crack will grow between the composite and the adhesive layer. If the crack grows through the adhesive only, then satisfactory adhesion is assured; the fracture energy of the adhesive material can additionally be calculated from a knowledge of specimen geometry and material properties, enabling a check on the toughness properties of the cured adhesive. The length of the crack and its position should be recorded.

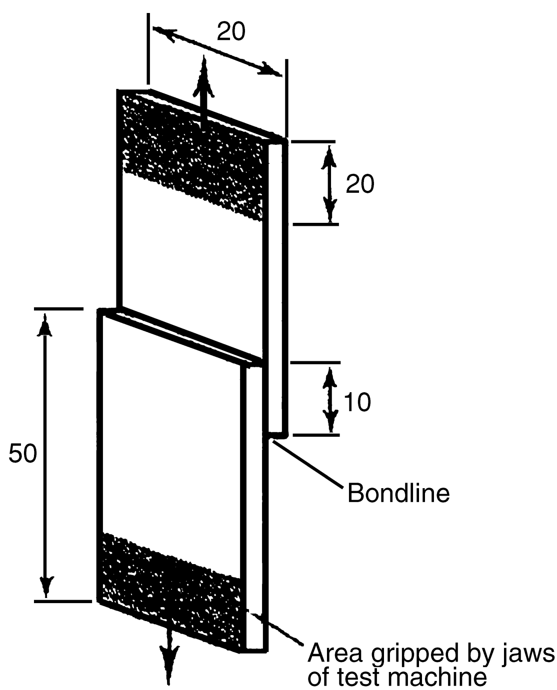

Figure 9.6 Single lap joint (all dimensions in millimetres).

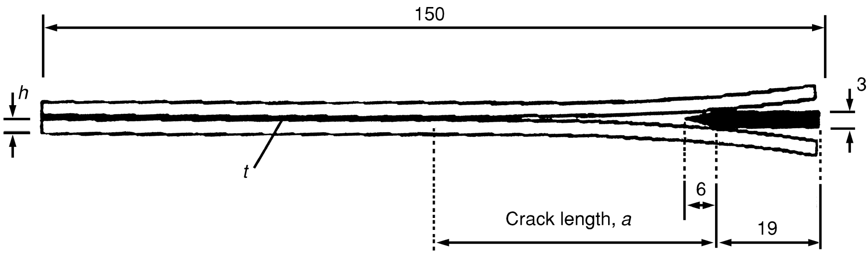

Figure 9.7 Wedge cleavage test (all dimensions in millimetres).

Tests should be carried out over a range of temperatures specifically including −25 °C, +20 °C and +45 °C using CFRP adherends. The temperatures are measured by means of thermocouples attached to the CFRP surface of the joint. The minimum average lap shear stress is 12 N mm^{-2} at 20 °C.

9.6.2 Tests for new materials – epoxy resin adhesives for CFRP plate bonding

9.6.2.1 Materials

The material consists of a two-part cold cured, epoxy-based adhesive comprising resin and hardener, the resin being based on the diglycidyl ether of ‘bisphenol A’ or ‘bisphenol F’ or a blend of the two. The hardener or curing agent should be from the polyamine group in order to achieve a better resistance to moisture penetration through the adhesive layer. Inert fillers may also be incorporated with the resin component to improve the application or performance characteristics of the adhesive. The filler should be an electrically non-conductive material, be highly moisture resistant, be able to withstand temperatures up to 120 °C without degradation and have a maximum particle size of 0.1 mm. The materials should be supplied in two packs, in correct weights for site mixing. The two components should have different colours in order to aid thorough mixing.

Other adhesives offering performance equivalent to that required by this specification for long term durability will be acceptable. In the absence of satisfactory laboratory accelerated durability tests, however, the assessment of the durability of an alternative adhesive should be based on the provision of evidence that it has, in practice, performed satisfactorily in conditions similar to the use proposed for a period of not less than 15 years after installation.

9.6.2.2 Mixing

Mixing should take place in accordance with manufacturer's instructions.

9.6.2.3 Placing

The adhesive should be capable of being applied readily to both concrete and CFRP surfaces in layers from 2–10 mm thick.

9.6.2.4 Cure time and temperature

The adhesive should be capable of curing to the required strength between 10 °C and 30 °C in relative humidities of up to 95%. For repairs and strengthening works the adhesive should cure sufficiently to give the specified mechanical properties at 20 °C in not more than three days. On curing the adhesive should undergo negligible shrinkage.

9.6.2.5 Usable life

Mixed adhesive, before application to the prepared surfaces should have a usable life in excess of 40 min at 20 °C. Test method as in BS 5350 : Part B4 (1993).

9.6.2.6 Open time

The time limit in which the joint should be made and closed should not exceed 20 min at a temperature up to 20 °C. Test method as in BS 2782 : Part 8 Method 835 (1994).

9.6.2.7 Storage life

Shelf life of both the resin and the hardener should be not less than 6 months in original containers at 5 °C and at 25 °C.

9.6.2.8 Mechanical properties of hardened adhesive

Cure and storage temperature, as stated in the appropriate standards, should be used for all tests and a minimum of five tests should be undertaken for each test requirement and the average of the results taken.

9.6.2.9 Moisture resistance

Minimum moisture transport through the adhesive should be ensured, with water content not exceeding 3% by weight after 28 days immersion in distilled water at 20 °C. Specimens should be 40 mm by 40 mm and between 1–2 mm thick. The procedure and calculations are used in accordance with BS 6319: Part 8, Clause 6.2.7 (1984) to determine water content.

9.6.2.10 Heat distortion temperature

The adhesive should have a heat distortion temperature (HDT) of at least 40 °C. The sample under test should be placed in a temperature controlled cabinet at 20 °C and loaded in order to achieve a maximum fibre stress of $1.81\,\mathrm{MN\,m}^{2}$ in accordance with ISO 75 (1987). The HDT should be taken as the temperature attained, measured by a thermocouple attached to the specimen, after undergoing a further 0.25 mm deflection while subject to a surface heating rate of $0.5\,°\mathrm{C\ min}^{-1}$.

9.6.2.11 Flexural modulus

Flexural modulus, without creep effects, at 20 °C should be in the range of $4–10\,\mathrm{kN\,mm}^{-2}$. A specimen 200 mm by 25 mm by 12 mm deep tested in four point bending should be used. The sample should be loaded at the third points to achieve a deflection at a rate of $1\,\mathrm{mm\,min}^{-1}$ and the central deflection recorded. From the resultant load–deflection curve, the secant modulus at 0.2% strain is calculated.

9.6.2.12 Tensile strength

A minimum value of $12\,\mathrm{N\,mm}^{-2}$ at 20 °C should be achieved. A dumb-bell specimen should be manufactured in accordance with BS 2782: Part 3 (1996) with a cross-section of 10 mm by 3 mm. The specimens should be cast in polytetrafluoroethylene-lined moulds. Adhesive ductility may be measured with appropriate strain monitoring equipment.

9.6.3 Concrete

The suitability of the preparation of the concrete surface should be demonstrated by a 'pull-off' test using the adhesive to be used in the bonding of the plates. The test should be deemed a failure unless the failure plane is within the concrete.

The bond of the plate to the concrete may be demonstrated after bonding by suitable tests such as coin tap, ultrasonic pulse velocity or thermography.

9.6.4 Trial panels and procedure trials

A trial panel and procedure trial for approval by the engineer should be provided for the strengthening with a plate of the same width and thickness and incorporating the same fixing details as those designed for the structure. This should demonstrate the following:

- preparation of the plate
- preparation of the substrate surface
- use of spacers
- application of adhesive
- installation of fixings
- finishing of installation
- compliance with quality control items

9.7 In-service inspection and maintenance

9.7.1 Manual of procedures

A manual of procedures for inspection and maintenance for the strengthened structure should be prepared for the maintaining authority. A typical summary of inspection procedures is given below but this should not be regarded as exhaustive. A draft list prepared during design can be extended during construction should it become necessary.

It is suggested that inspections should take place every six months for at least two years after completion of the work. The frequency of further inspection should be agreed with the maintaining authority after a review of the inspection reports.

The inspection procedure manual should include all relevant technical literature relating to products used in the work. Photographs of critical details and maps of cracking are also useful. Procedures to be adopted if deficiencies occur should also be clearly defined.

The deck soffit should be checked to ensure:

- There is no visible evidence of sealed cracks in the concrete opening.
- There is no visible evidence of the plates debonding.
- There is no audible evidence of the plates debonding by tapping the plates with a coin or suitable light hammer.
- The plate fixing bolts are not loose.

The top surface of the deck should be checked to ensure:

- There is no visible evidence of the road surfacing cracking or deforming.
- There is no visible evidence of any cracks in the edge parapet detail increasing in width or new cracks forming.

Hammer tapping must not damage the system or any nuts slackened on the fixing system.

9.8 References

ASTM D3163 (1992) *Standard Test Method for Determining Strength of Adhesively Bonded Rigid Plastic Lap-Shear Joints in Shear by Tension Loading*, ASTM Philadelphia 15.06.

ASTM D3039/3039M (15.03) (1995) *Test Method for Tensile Properties of Polymer Matrix Composite Materials*, ASTM Philadelphia.

ASTM D3762 (1979) Standard test method for adhesive bonded surface durability of aluminum (Wedge test), ASTM, Philadelphia.

BS 1881 : Part 207 (1992) *Recommendations for the Assessment of Concrete Strength by Near-to Surface Tests*, British Standards Institution, London.

BS 5400 : Part 4 (1990) *Code of Practice for Design of Concrete Bridges – Steel, Concrete and Composite Bridges*, British Standards Institution, London.

BS 5400 : Part 2 (1990) *Specification for Loads*, British Standards Institution, London.

BS 5350 : Part B4 (1993) *Determination of Pot Life – Method of Test for Adhesives*, British Standards Institution, London.

BS 2782 : Part 3 (1996) Mechanical Properties 1976, Method 320 B/C, *Tensile Strength, Elongation and Elastic-Modulus Method of Testing Plastics*, British Standards Institution, London.

BS 2782 : Part 8 (1994) Other Properties 1980, Method 835C, *Determination of Gelation Time of Polyester and Epoxide Resins using a Gel Time*, British Standards Institution, London.

BS 6319 : Part 8 (1984) *Method for the Assessment of Resistance to Liquids*, British Standards Institution, London.

Chajes M J, Finch W W, Januszka T F and Thomson T A (1993), 'Bond and force transfer of composite material plates bonded to concrete', *ACI J* **93**(2) 208–217.

CRAG (1988) *Test Methods for the Measurement of the Engineering Properties of Fibre Reinforced Plastics*, ed. P T Curtis.

EUROCOMP (1996) *Structural Design of Polymer Composites – EUROCOMP – Design Code and Handbook*, ed. J L Clarke, E & FN Spon, London.

Highways Agency (1994) *Strengthening of Concrete Highway Structures Using Externally Bonded Plates*, BA 30/94.

ISO 75 (1987) *Plastic and Ebonite – Determination of Temperature of Deflection under Load*.

Jansze W (1995) 'Anchorage of externally bonded plates under static flexural loading', *Progr Concrete Res* **4**, February.

Jansze W and Walraven J C (1996) 'Anchorage of externally bonded steel plates', *International Concrete Congress, In Service of Mankind*, Dundee, Scotland, 24–28 June 1996.

Jones R, Swamy R N and Charif A (1988) 'Plate separation and anchorage of reinforced concrete beams strengthened by epoxy-bonded steel plates', *The Struct Eng* **66**(5/1) 85–94.

Jones R, Swamy R N and Charif A (1989) 'Plate separation and anchorage of reinforced concrete beams strengthened by epoxy-bonded steel plates', *The Struct Eng* **67**(13/4) 250.

Kaiser H (1989) *Strengthening of Reinforced Concrete with Epoxy-Bonded Carbon-Fiber Plastics*, Thesis submitted to ETH, Zurich, Switzerland, in partial fulfilment of the requirements for the degree of Doctor of Philosophy (in German).

Meier U (1987) 'Bridge repair with high performance composite materials', *J Mater Technik* **4** 125–128.

Meier U and Kaiser H (1991) 'Strengthening of structures with CFRP laminates', *Advanced Composites Materials in Civil Engineering Structures Proceedings*, MT Div/ASCE/Las Vegas, Jan. 31 1991.

Neubauer U and Rostasy F S (1997) 'Design aspects of concrete structures strengthened with externally bonded CFRP-plates', *Structural Faults & Repair – 97*, Edinburgh, July 1997, Vol 2, pp 109–118.

Oehlers D J and Ahmed M (1996) 'Upgrading reinforced concrete beams by bolting steel side plates', *Bridge Management 3, Inspection, Maintenance and Repair*, Guildford, 1996, E&FN Spon, pp 412–419.

Poulsen E (1996) *Anchorage of Sika® CarboDur Laminates Bonded to Concrete*, CEN TC104 SC8 WG3 meeting in Paris 5/6 Dec 1996.

prEN 1542 *Products and Systems for the Protection and Repair of Concrete Structures – Test Methods, the Pull-off Test*, Awaiting German translation before ratification, not available publicly.

Ranish (1982) *Zur Tragfähigkeit von Verklebungen zwischen Baustahl uns Beton – geklebte Bewehrung*, Institute for Construction Materials, Massive type Construction and Fire Protection, Technical University Braunschweig, Issue 54.

Raoof M and Zhang S (1996) 'Peeling failure of reinforced concrete beams with externally bonded steel plates', *Bridge Management 3, Inspection, Maintenance and Repair*, Guildford, 1996, E&FN Spon, pp 420–428.

Regan P (1993) 'Research on shear: a benefit to humanity or a waste of time?' *The Struct Eng* **71**(19) 337–346.

Roberts T M (1989) 'Approximate analysis of shear and normal stress concentrations in the adhesive layer of plated RC beams', *The Struct Eng* **67**(12/20) 229–233.

Rostasy F S (1993) 'Strengthening of R/C and P/C structures with bonded steel and FRP plates', *Proc. 5th Int. Conf. On Structural Faults and Repair 1993*, Vol 2, pp 217–224.

Swamy R N, Jones R and Charif A (1989) 'The effect of external plate reinforcement on the strengthening of structurally damaged RC beams', *The Struct Eng* **67**(3/7) 45–56.

Täljsten B (1994) *Strengthening of Existing Concrete Structures with Epoxy Bonded Plates of Steel or Fibre Reinforced Plastics*, Doctoral Thesis, Luleå University of Technology, Division of Structural Engineering, Sweden. August 1994.

Tausky (1993) *Betontragwerke mit Aussenbewehrung*, Birkhäuser Verlag.

Taylor H P J (1974) 'The fundamental behaviour of reinforced concrete beams in bending and shear', *Proc. ACI-ASCE Shear Symposium*, Ottawa, 1973 (ACI Publication SP42-3) American Concrete Institute, Detroit, 1974, pp 43–77.

Triantafillou T C and Plevris N (1992) 'Strengthening of RC beams with epoxy-bonded fibre-composite materials', *Mater Struct* **25** 201–211.

Wu Y S and Gibson A G (1994) 'Composites offshore: applications and opportunities', *Composites 94*, Birmingham 22/23 November 1994, British Plastics Federation Publication No 293/10 pp 45–56.

Zhang S, Raoof M & Wood L A (1995) 'Prediction of peeling failure of reinforced concrete beams with externally bonded steel plates', *Proc Inst Civ Eng Struct Building* **110**(Aug) 257–268.

9.9 Bibliography

BD2 (DMRB 1.1) Technical Approval of Highway Structures on Motorways and Other Trunk Roads.

BA 35 (DMRB 3.3) The Inspection and Repair of Concrete Highway Structures.

SB 1 (DMRB) (Scotland) The Inspection of Highway Structures.

DB 24 (DMRB 3.1) Design of Concrete Highway Bridges and Structures, Use of BS5400 : Part 4: 1990.

DB 37 (DMRB 1.3) Loads for Highway Bridges.

BD 21 (DMRB 3.4) The Assessment of Highway Bridges and Structures.

10
Site construction techniques

A P RIMOLDI

10.1 Introduction

The bonding of plates to civil engineering structures has been recognised as useful since the end of the 1970s. The work is usually carried out in order to enhance the load-bearing capacity of the structure, either because the designed loading is likely to be exceeded or because modification or deterioration of the structure demands it. This chapter describes the techniques and problems associated with plate bonding in real commercial situations on construction sites.

Concrete Repairs Ltd is a well established specialist contracting company with a wealth of experience in the repair, maintenance and modification of reinforced concrete structures. It has been carrying out plate bonding since the early days of acceptance of the technique in the UK marketplace. Since 1996 the company has carried out several projects using carbon fibre reinforced polymer (CFRP) plates as a strengthening medium for reinforced concrete structures.

Experience gained in the technique of externally bonding CFRP plates to concrete structures, by undertaking commercial contracts over a two year period, is summarised in the following sections. This technique is the one adopted by the ROBUST system.

10.2 Steel plate bonding

Before discussing the use of CFRP plates it is important to consider briefly the more traditional technique of externally bonding steel plates.

Since the strengthening in 1975 of the Quinton Bridge on the M5 motorway, steel plate bonding has been increasingly used in the UK for upgrading concrete beams. The concrete in which steel is embedded as reinforcement can serve to protect the steel because the highly alkaline nature of concrete provides an environment that is beneficial to the embedded reinforcement. If the concrete is of good quality, well compacted and with minimal cracking, it is difficult for the process of corrosion to take place. In such circumstances, reinforced concrete can support the tensile and shear stresses experienced by the structure.

However, corrosion of steel reinforcement through the process of carbonation can be one of the major limitations to the longevity of a reinforced concrete (RC) structure, with the presence of chlorides, poor detailing and workmanship all contributing to the corrosion process. In extreme cases of degradation a structure may become unsafe or beyond economic repair. There is a continuing worldwide problem regarding corrosion of embedded reinforcement, and of course, a worldwide industry maintaining structures and preventing corrosion.

In most cases, externally bonded reinforcement (in the current context bonded plates) is required to add to the capabilities of the existing embedded reinforcement. This can be to enhance strength or effectively to replace corroded reinforcement.

In the past, upgrading structures was achieved by literally adding a further layer of reinforcement to that of the existing steel and then encasing the whole within a concrete jacket. This was a very expensive process, tantamount to rebuilding a large section of the structure.

The idea of adhesive bonding steel plate to a suitably prepared concrete surface was then developed. This appeared to be a relatively easy method for construction sites, but in practice handling up to 6 m long plates, usually on to soffits of beams, and in addition providing multiple end anchors without damaging the applied coating on the plates can be an extremely difficult operation. The survey and setting out of the end anchor bolts is fraught with difficulty especially on a heavily reinforced structure. The protective paint coatings to the plates need to be applied under almost clinical conditions and, of course, the coatings require maintenance during service life. Lack of maintenance of the plates will result in their rapid corrosion as they will not then have the benefit of being embedded in concrete.

Steel plate bonding is a perfectly viable means of providing extra strength. However, the logistics of handling plates weighing several hundred kilograms makes the operation a difficult one. Furthermore, fabrication and preparation processes need to be allowed for when programming works and the slow application often means lengthy restrictions in the use of the structure. This could result in long traffic delays when dealing with road bridges.

The use of CFRP plates simplifies and shortens the whole process of plate bonding and its adaptability allows a wide range of options to be considered.

10.3 Adhesive bonding of carbon fibre composite plates – site requirements

The following sections will discuss the management, design and site construction details which are associated with adhesively bonding CFRP plates to RC beams.

10.3.1 Construction Design Management Regulations 1994 (CDM)

Virtually all commercial construction projects in the UK need to be carried out under CDM (1994) regulations (this requirement became active from March 1995). These specific legally enforceable regulations need to be considered and followed at every stage in the design and construction process. The regulations concern the management of health and safety. It is assumed that all works described in this chapter properly comply with the Regulations.

10.3.2 Health and safety

It is imperative that all construction site operations are carried out in such a manner that risk to site workers and others that may be affected by the work is either eliminated or reduced to a safe level. It is assumed therefore that in all the following descriptions risk assessments have been made and that where necessary personal protective equipment (PPE) is supplied and used. It is also assumed that a safe means of access and materials handling is employed and that suitable measures to protect passers-by are maintained.

10.3.3 Site set-up

Before carrying out any other works the site should be visited to determine its location, its relationship with the immediate environment, ease of access for personnel and deliveries, provision of services, security and other special considerations to enable a work site to be established.

Following this, a suitable works compound should be built providing accommodation and facilities for personnel, secure weatherproof storage for materials and, of most importance, a clear unobstructed area for laying out the plates. Consideration must be given to the prevailing weather conditions, time of year and any other physical factors that may affect the works.

Finally a safe means of access to the working area if necessary, must be built. This must take due account of risk of falling, work over water or live railway tracks and must be constructed accordingly.

10.3.4 Loading

The designer of the strengthening system may need to take due account of dead and imposed loadings during the bonding process. For example, a highly trafficked road or rail bridge may introduce stresses into the plates during the curing process. If the designer finds that this is unacceptable, several other options are available. Traffic can be removed from the structure either by lane restrictions, night-time possession or total closure. Alternatively, propping may be installed to accept the loading during the bonding process. Other forms of preload, such as jacking or kentledge, may be applied as a method of inducing a stress into the plates as part of the design and installation process.

10.3.5 Condition survey

Once the structure can be easily inspected a comprehensive concrete condition survey should be carried out. This starts with a visual inspection to check for obvious defects (cracks, spalls, poor compaction, uneven surfaces) and is followed by a hammer sounding survey to identify hollow areas. Additionally, chemical tests (depth of carbonation, chloride levels) may be necessary.

Prior to carrying out the bonding works it is important that a sound structure is available. The previously mentioned tests will identify defects and deterioration. If necessary a package of repair can be carried out to restore the structural concrete to its original state and thereby create an undamaged structure for the bonding works. If anchor bolts are required, an electronic covermeter can be used to identify the position, size and depth of embedded reinforcement. All the information gained from the condition survey should be carefully drawn up and recorded for future reference and use.

10.3.6 Setting out

Should the design require it or if there are special considerations (e.g. prestressing) then setting out should be considered at this stage. This will enable the anchor plates to be bonded into position and the positions of the anchor holes on relatively flat surfaces to be marked. It may be necessary to clean the surfaces by high pressure water jetting to remove dirt and contaminants to make the marking up process easier. The setting out is only required at this stage if hole drilling and coring is to be carried out before plates are bonded.

10.3.7 Hole drilling

Where predrilled or prestressed plates are to be used it is advisable to consider hole drilling at this stage. Using the information from the covermeter survey and suitable templates, the positions for end anchorage bolts or prestressing fixings can be marked accurately. If the design allows the existing reinforcement to be cut, then slow speed diamond coring can be used. This process uses water as a lubricant and will provide an accurate core hole. Any intervening reinforcement will be cut in the process.

Percussion drilling is usually quite satisfactory, provided that accuracy can be maintained. Intervening reinforcement will not be removed by this process, so careful positioning and planning is required. This is especially so where a group of bolts (often 6 or 9 no.) is required in a small area. The perpendicularity of the holes is also extremely important. Although diamond drilling is usually carried out using a jig, percussion drilling is often hand held. Slight deviations or inaccuracies at this stage will mean that it becomes difficult or impossible to position the anchor bolts through the plates. Particular care must be taken when drilling into prestressed or post-tensioned structures to avoid damage to the tendons.

10.3.8 Surface preparation

To ensure successful bonding of the plates, a high standard of surface preparation is essential. Concrete surfaces should be free of all contaminants including grease, oil and existing coatings. Surface laitance must also be removed, as should any unevenness or sharp ridges and formwork marks on the concrete surface. Sudden changes in level should be less than 0.5 mm.

A well-weathered concrete surface could be suitably prepared using high pressure water jetting techniques. All other surfaces (which, if recently renovated should be at least 6 weeks old) should be prepared by grit-blasting followed by vacuum cleaning (or vacuum blasting if available).

If gritblasting is not possible, scabbling, needle gunning or grinding may be utilised, although these techniques are not recommended as they will almost certainly leave a much rougher surface than gritblasting. Such surfaces may require remedial works before plate bonding can begin. Significant surface imperfections can therefore be dealt with at this stage. Usually an epoxy mortar would be used, which provides a fast gain in strength and will allow overbonding to take place quickly.

As a final check on the surface preparation, it is usual to perform a minimum of three pull-off tests on the prepared substrate; these tests will also provide an indication of the concrete strength. The test method involves bonding a 50 mm diameter steel dolly to the concrete surface after preparation.

The load at pull-off and the location of the failure plane should be recorded. This simple test will help to ensure good surface preparation and to give an indication of the tensile strength of the concrete substrate.

10.3.9 Working area

At this point in the proceedings it is worth considering the working area. The CFRP plates are extremely light and often very long (20 m). When handled they will be coated with adhesive which will pick up any contaminants that may come into contact with them. A clear unobstructed working area is therefore essential. Consideration should be given to adverse weather conditions that may affect the work (e.g. wind, rain, low temperatures). If necessary, temporary roofing or heating should be provided. It must be possible to transport the coated plate to the structure without any risk of it becoming contaminated.

A suitable source of electricity should be available together with cleaning and washing facilities for the plant and personnel. Lighting should also be available if necessary.

10.3.10 Materials and plant

CFRP plates come in varying widths (e.g. 50 mm, 90 mm, 100 mm), varying thickness (1.0 mm, 1.2 mm) and any required length. CFRP plate is very expensive and therefore minimisation of waste is an important factor. Roll lengths of 250 m are available. Wastage from rolls must be considered when ordering cut lengths. Plate can be easily cut to length on site using a guillotine.

Some CFRP plate is delivered in its raw state and will require cleaning to remove contaminants. Other types of plate are supplied with a peel-ply protective coating which is removed immediately prior to bonding to ensure a completely clean surface. Plate should be checked for twist. A twisted plate will not lay flat on the surface and should be rejected.

CFRP plate should be handled carefully. The unidirectional longitudinal weave of the fibres means that it can easily split along its length. This characteristic is far less of a problem on peel-ply plates.

Prestressed plates may well be supplied with glass fibre, predrilled anchor blocks already bonded to cut lengths of CFRP. Again, the dimensions and quality of these must be checked before use as an incorrectly supplied plate can cause delay to the contract and be costly to remedy.

Epoxy adhesives are available from several sources. They are two part (resin and hardener) for site mixing. Again material choice is based on several factors. Client or contractor preference, track record, technical performance and cost. Special considerations such as pot life, ambient temperature and environmental issues may also influence the choice. Special care must be taken when disposing of the used containers.

It is usual, during the course of a project, to take samples from each batch of the adhesive used. These specimens are usually 200 mm long by 12 mm deep by 25 mm wide and formed in steel moulds. They should be cured in the same ambient conditions and for the same length of time as that used for the bonding operation. After demoulding, these samples are tested and a load/deflection graph is recorded. Tensile strengths of adhesive can be checked after manufacturing a dumb-bell specimen which is loaded to failure. These tests are further detailed in BA 30/94 Part 1 (1994). All material testing should be carried out by an independent testing authority (Cranfield University, 1996). The ROBUST project recommends that lap shear tests should be undertaken (see Chapter 3, Section 3.3.6).

Ancillary equipment such as anchor bolts, sleeves and studs are usually chosen from manufacturers' catalogues after considerations of exposure (stainless steel, plated steel) and technical specifications.

Plant is generally confined to fairly small items. Slow speed mixing drills with paddles for the adhesive are usually employed together with handtools for adhesive application. Drilling machines will be necessary for anchor bolt holes if they are to be formed after the plates are bonded.

As in all parts of the project the correct personal protective equipment must be available and used at all times.

10.3.11 Bonding of unanchored plates

The simplest form of CFRP bonding is a single layer unanchored plate. These are installed when the designer considers that end anchorages are unnecessary.

The plate is selected and laid out upside down on the adhesive table. If it has a peel-ply protective coating this is removed from the adhesive side at this stage. If it is an unprotected plate, its surface is lightly sanded and cleaned using a solvent wipe. The plate is visually inspected for defects and imperfections. From this point on cleanliness is vital and operatives wear surgical-type gloves. The concrete surface is finally checked and lines are marked to show the approximate position of the plate.

Adhesive mixing can now take place. The cold cured epoxies usually have resin and hardener in differing colours (e.g. white and black). When mixed together with a slow speed paddle drill the mixture turns into a grey paste (Sika CarboDur, 1998). Mixing is complete when there are no inconsistencies in colour.

Mixing usually takes place in the larger of the two delivery pails. The slow speed mixing paddle avoids air entrainment. Depending on ambient conditions the pot life is about 30 min. Before applying adhesive a final check of all surfaces and components should be made.

The mixed adhesive is applied by hand trowel to the selected position on the concrete structure (Fig. 10.1). Using skilled labour, conventional plastering and scraping techniques will ensure that all voids are filled and a 1 mm thick skim adhesive is left on the surface.

During this operation it is recommended the mixed adhesive is also applied to the plate. It is suggested that a 'dome'-shaped profile of adhesive is formed, some 2 mm thick in the centre of the plate, thinning to 0.5 mm at the edge. This can be achieved using a dome-shaped spatula fixed at one end of the adhesive reservoir. The whole assembly can be drawn along the length of the plate, thus extruding the required profile of adhesive to the plate. This is a quick and easy method (Fig. 10.2).

Spacers may be required to maintain adhesive thickness, although these are not essential. Conventional plastic washers of known thickness can be set into the adhesive. Alternatively, single sized glass balls or ballotini can be sprinkled over and embedded into the adhesive. Both systems are successful at maintaining a minimum adhesive thickness.

The plate is then ready to be applied to the concrete surface. This operation usually requires three operatives if the plates are more than 3 m long. The coated plate is lifted from the table and taken to the structure. One end is placed on the surface using only hand pressure. Working away from that end and using hand pressure only, the plate is bonded to the structure.

Figure 10.1 Applying epoxy adhesive to prepared concrete surface.

Figure 10.2 Setting dome-shaped spatula into adhesive reservoir prior to application of epoxy adhesive to plate.

Moving along the plate the technique continues until the far end is reached. Even when working on soffits the adhesive is strong enough to take the very light weight of the CFRP plate without risk of sagging or debonding (Fig. 10.3).

Once in place a hard rubber roller is applied to the plate to squeeze out excess adhesive and ensure firm adherence to the substrate. A trowel is used to tidy up the edges and remove excess adhesive. This adhesive should not be reused (Fig. 10.4).

The bonding of one plate is thus completed. The exercise, after thorough planning, is very straightforward and can then be repeated for multiplate applications.

After 48 h the fitted plate should be checked for voids. One very simple and effective method is to 'coin-tap' the surface of the composite plate. More sophisticated methods would be the thermography, 'woodpecker'

Figure 10.3 Fitting CFRP plate to concrete surface.

Figure 10.4 Using a hard rubber roller to squeeze out excess adhesive and ensure firm adhesion.

and ultrasonics techniques. If voids are found, these can be injected with epoxy resin through the sides of the exposed adhesive. This method is satisfactory for steel plates but is not recommended for CFRP plates. It must be stressed, however, that voids in the adhesive layer are extremely rare. If peel-ply is fitted to both sides of the composite plate the outer peel-ply layer can be removed after the composite plate has fully bonded to the adherend.

10.3.12 Bonding of anchored plates (predrilled)

There are circumstances when, for design considerations, the CFRP plates are supplied with anchor bolt plates in fibreglass already bonded to the ends and predrilled. As before the chosen plate must be offered up to the predrilled concrete to check dimensions and hole positions. Once approved, the anchors for one end only can be fitted. These may be single anchors or a group.

Again, depending on design and anchor characteristics, expanding or resin anchors can be used. The former expand and grip the predrilled concrete using expanding wedges. The latter use epoxy or polyester resin in premixed or capsule form to set, either a threaded sleeve or a stud into the concrete.

The setting of these anchors is critical. They are often of at least 12 mm diameter and sometimes 25 mm or 30 mm. They must be set centrally in their holes and perpendicular to the concrete surface. Extra care must be taken where multiple anchors are fitted to ensure that they are correctly spaced in relation to one another (Fig. 10.5).

Figure 10.5 Detail showing bolted anchorage of a multiple plate installation.

Once the anchors are set for one end of the plate the bonding of the plate can take place. The techniques are similar to those for unanchored plates. The coated plate is placed onto its anchor bolts and the nuts and washers are hand tightened. Using hand pressure only, and working away from the anchored end, the plate is bonded to the structure. When the holes in the plate and structure are aligned, the anchor bolts at the other end can be installed and again be hand tightened. Once in place pressure is applied to the plate using a hard rubber roller to squeeze out the excess adhesive and ensure firm adhesion to the substrate. After 48 h the plate can be checked for soundness and voids and the anchor bolts can be tightened to the required torque.

10.3.13 Bonding of anchored plates (undrilled)

If anchor systems are required the most frequently used technique during the installation of CFRP plates is to apply end anchor bolts to undrilled plates. (ROBUST does not recommend this method as it can weaken the composite plate. Section 4.3.3 of Chapter 4 discussed the ROBUST investigative work on anchor plates.)

In this procedure the plate length and the setting out of the anchorage system is not nearly so critical as when using a predrilled plate. The technique for bonding is identical to that of the unanchored plates and is, therefore, much quicker than a predrilled plate system. Any site tolerances can also be accommodated. Once the plate has been successfully bonded to the structure it can be drilled to accept its anchors. Care must be taken not to hit embedded reinforcement. A covermeter should be used to ascertain the position of the embedded reinforcement. The CFRP plate is ‘invisible’ to a covermeter thus making the setting out relatively straightforward.

The CFRP plates are easily drilled *in situ* using a hand-held rotary drill set at slow speed. Again, the setting of the anchors, checking and testing is the same as for the previous sections.

10.3.14 Bonding of multiple plate thicknesses, anchored and unanchored

There are projects that will require a multithickness plate installation to provide the required increases in structural strength. Whether these installations are anchored or unanchored, the techniques are similar to single thickness work.

After bonding the first layer to the concrete structure the installed plate becomes the substrate for the next layer, and so on. The peel-ply to the exposed surface of the composite plate is removed. If a plate without peel-ply has been fitted then the surface of that plate should be lightly

Figure 10.6 Detail showing spliced plate application.

sanded to provide a key. The same resin adhesive is used and the next plate is bonded.

If anchors are necessary they can be installed through predrilled holes or holes can be drilled through the multilayers of plates at the end of each group of plates; however, ROBUST does not recommend this procedure. The technique can also be used to strengthen a structure in two or more directions by criss-crossing the plates (Fig. 10.6).

If drilling the main plates is not possible then short lengths of CFRP plate can be used to provide an end anchorage as a strap perpendicular to the main plate.

10.3.15 Prestressing of CFRP plates

The final option available to both the designers and installers of CFRP plates is prestressing. This is a new technique that has only been carried out once on a full size structure. However, the success of the process will allow the use of CFRP plate strengthening techniques to be considered for a whole range of projects.

The prestressing machine is essentially a custom built plate framework that is securely bolted to the concrete substrate. Attached to the frame are two manual screw jacks which in turn are connected to a roller mounted stressing bar. This bar is locked to a stressing block at one end of the CFRP plate using a shear pin (Fig. 10.7).

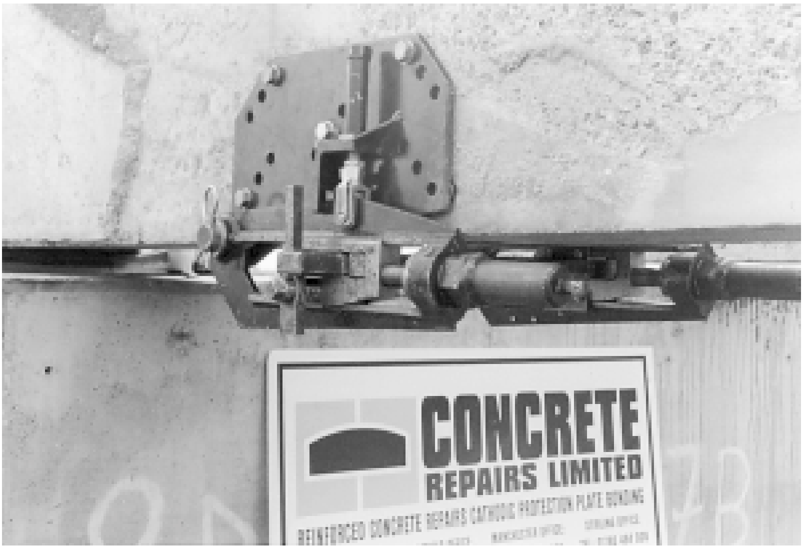

Figure 10.7 Prestressing machine bolted in place to a reinforced concrete beam.

Figure 10.8 Connecting the CFRP plate to the prestressing machine.

A predetermined axial stress is applied to the plate using simple load/extension considerations, that is, a known extension will correspond to a calculated load. Adaptations of the machine are possible for clamping to beams or soffits and for stressing two plates simultaneously (Fig. 10.8).

The prestressing machine has to be accurately and securely fixed to the structure. The anchor bolts for the machine have to be carefully set out and fitted, not only to facilitate secure mounting of the machine under load but to maintain the correct relationship with the anchor bolt that will secure the stressed plate.

After calculating the elongation of the plate under load, the end anchor position after stressing can be calculated and set out. The end anchor bolts can be set into the concrete substrate and the adhesive applied to the plate and substrate as before. The anchor block on the plate is then connected to the shear pin on the prestressing machine.

After checking that all the connections are secure the stressing process can begin. The slack is taken up using the screw jacks and other adjustments. The starting position of the taut plate in relation to a fixed scale is recorded. Using the two screw jacks the plate is carefully tensioned (Fig. 10.9).

The load required in the plate will be reached at a calculated extension. This is measured on a fixed scale. When the required extension (and therefore load) is reached the anchor bolt holes on the plate will correspond to those predrilled on the structure. When they coincide the anchor bolts can be inserted and tightened (Fig. 10.10). The screw jacks can then be released, the stressed plate being securely bolted to the structure. Bearer beams and props may need to be fitted against the plates at this stage to ensure firm adherence to the substrate (Fig. 10.11).

Figure 10.9 Using the twin screw jacks to tension the plate and apply prestressing.

Figure 10.10 Inserting the anchor bolts after applying the required prestress load.

Figure 10.11 Props and bearer beams installed to prestressed plate.

Finishing and testing of the installed prestressed plate will be as before. Again, multiple thickness installations are possible with this technique.

10.3.16 Maintenance and protection

Once installed the CFRP plates require no maintenance. There is obviously no risk of corrosion of the plate itself. The use of stainless steel anchor bolts will ensure a very long life in even the harshest of environments. There are few circumstances that would prevent the design life of the strengthening system equalling that of the structure.

After removal of the access required for the installation, the plates are usually inaccessible. However, there are circumstances where the plates themselves may need protection or hiding for purely aesthetic reasons. It may be necessary to provide fire protection to the installation or to cover it up to avoid the risk of vandalism.

The installed plates can easily be covered with a cementitious overlay applied by spray or hand placed. To ensure adhesion of the overlay, an epoxy bonding coat is applied prior to the cementitious material. The overlay can be left 'as sprayed' or trowel finished smooth and, if required, subsequently painted. The strengthening system can therefore be hidden from view very successfully and with great confidence that future maintenance inspections are unnecessary.

It is important that the installation is noted in the site safety file (a requirement of CDM (1994)), especially if it is covered up. This will prevent unauthorised drilling or damage to the plates from subsequent works and trades.

10.4 Economics

In comparison to the steel alternative, CFRP plates are certainly not cheap. The raw material cost is often four times that of steel. However, the advantages are found in installation costs and whole life costings.

The costs of installation in terms of labour are much lower than for steel. Transport and handling costs are lower and, of most importance, the installation of CFRP plate is very fast. A reduced contract programme obviously lowers the ancillary costs of access and plant hire, propping and site set-up. Even more significant are the reduced timescales for road closures or traffic management.

With negligible planned expenditure on maintenance, the economics of CFRP plates become very attractive. Currently, the high cost of the carbon fibre composite is offset by the advantages detailed above. This results in CFRP plates being a viable and realistic alternative to the utilisation of steel.

Figure 10.12 Overall view of completed installation of plates to a reinforced concrete bridge beam.

10.5 Conclusion

Structural strengthening using externally bonded plates is a well-established process in construction and repair. The acceptance of CFRP plate as an alternative to steel has opened up the marketplace (Fig. 10.12). It is a straightforward and elegant technique whose performance can be easily monitored and controlled in a site environment. The flexibility of the system allows the designer a greater scope to achieve his goals. The simplicity of the system allows the installer to work in a risk free environment without the problems associated with handling and maintaining heavy steel plate. The economics of the system now compare favourably with those of steel and must surely be attractive to the client when the need for future maintenance is removed.

10.6 References

BA30/94 (1994) *Design Manual for Roads and Bridges*, Vol 3, Section 3, Part 1, BA30/94, Strengthening of Concrete Highway Structures Using Externally Bonded Plates. Department of Transport, Feb 1994.

CDM (1994) *Construction (Design & Management) Regulations* (CDM), SI 1994/3247. HMSO.

Cranfield University (1996) *Quality Control Testing Associated with Plate Bonding Works*, Internal Report, Royal Military College of Science, Shrivenham. January 1996.

Sika CarboDur (1998) *Data Sheet 1.95*, Sika, Welwyn Garden City, Herts, UK.

11 Case studies of carbon fibre bonding worldwide

M A SHAW AND J F DREWETT

11.1 Introduction

The following case studies utilising the carbon fibre/polymer composite strengthening concept have been incorporated into this book to illustrate some practical examples of different strengthening applications. The ROBUST project which has recently been completed has not, as yet, any proven practical examples of the strengthening of existing bridge or building structures.

Sika Ltd, Hertfordshire, UK, one of the members of the ROBUST Consortium, has developed over a number of years a procedure for strengthening, using a system approach comprising an advanced epoxy structural adhesive and a range of carbon fibre/epoxy polymer laminates. This combined system is known as the Sika CarboDur system.

Sika have been in the forefront of structural adhesive engineering in the construction industry for many years. Structural adhesives have been specifically developed to transfer stresses from one substrate to another for a range of different situations. Typical applications include segmental bridge construction and strengthening structures using external plate bonding techniques. Each material has undergone intensive research and development and site trials to comply with the industry's demands of cost effectiveness, performance and durability.

In the past external strengthening had been successfully carried out with steel plates and epoxy-based structural adhesives. There are limitations to both the design and practical aspects when using plates and these have been highlighted in the earlier chapters. Sika have been involved with over 200 CarboDur projects worldwide, many of which would not have been possible using traditional strengthening techniques. The case histories selected show the adaptability and cost effectiveness of the technique and the global acceptance of the CarboDur system.

11.2 System properties

The general properties of the system are shown below. The carbon fibre laminates are manufactured by the pultrusion process to the Swiss Federal Laboratories for Material Testing and Research (EMPA) specification.

11.2.1 Sika CarboDur laminates

Sika CarboDur laminates consist of:

- carbon fibre reinforced with epoxy matrix
- fibre volumetric content >68%.

The laminates are available in a range of different grades. The minimum and maximum properties are shown in Table 11.1. Intermediate grades lie within this range.

Table 11.1 Minimum and maximum properties of Sika CarboDur laminates

	Minimum properties	Maximum properties
Thickness (mm)	1.2	1.4
Width (mm)	50	150
E-modulus ($N\,mm^{-2}$)	165 000	300 000
Ultimate tensile strength ($N\,mm^{-2}$)	1450	3050

11.2.2 Structural adhesive, Sikadur 30

The properties of Sikadur 30 are

- two-part solvent free epoxy adhesive
- density 1800 kg/m^{-3}
- glass transition point about 62 °C
- flexural modulus 12 800 $N\,mm^{-2}$
- compressive strength about 90 $N\,mm^{-2}$
- tensile strength about 30 $N\,mm^{-2}$
- tensile slant shear strength about 18 $N\,mm^{-2}$
- moisture uptake <0.5%.

For the case histories included in this chapter, the application of the CarboDur system has generally followed the procedure outlined below:

- The substrate surfaces were prepared to remove all contaminants.
  - Concrete: fine and coarse aggregate exposed and fine gripping texture achieved by blast cleaning,

  - Timber: surfaces planed or grit blasted,
  - Masonry: fine gripping texture achieved by blast cleaning or scabbling.
- Substrate surfaces vacuum cleaned to remove dust.
- CarboDur carbon fibre laminates cut to length at factory or on site.
- Preprepared laminate bonding surface cleaned.
- Sikadur 30 structural adhesive applied to laminate and substrate.
- Carbon fibre laminate pressed into position on substrate by hand.
- Carbon fibre laminate bedded into adhesive using hard hand rubber roller to extrude excess adhesive and produce void free bondline.
- Surplus adhesive removed from substrate and laminate.

11.3 Case histories

11.3.1 Kings College Hospital, London, UK

11.3.1.1 The problem

In June 1996, this was the first project in the UK to use a carbon fibre strengthening system for external poststrengthening. The project involved the overall refurbishment and extension of the Normanby College suite for the new Joint Education Centre. As part of the refurbishment brief, an extra floor was required for accommodation. This was achieved by converting the roof slab to a floor slab thereby increasing the live load capacity to $3.0\,kN\,m^{-2}$. The construction of the existing building consisted of a reinforced concrete frame with cast *in situ* troughed floors. The upper storey extension was designed as a lightweight steel frame structure, however calculations on the existing roof slab established that in its present form, the slab could not sustain the additional live load.

To resolve the problem, three options were considered:

- Demolish the existing roof and construct a new floor.
- Provide a secondary steel frame to support existing slab or new separate floor.
- Externally poststrengthen the roof slab by external plate bonding.

The first option was considered time consuming and expensive. The second option was not feasible owing to the long spans. Therefore the option of poststrengthening was preferred, based on its cost effectiveness and speed of application. The initial design showed that steel plates 75 mm wide and 6.0 mm thick were required to provide the additional reinforcement.

11.3.1.2 The solution

The original roof slab was formed from 400 mm deep tapered ribs 80 mm wide at the bottom located at 600 mm centres spanning 11.0 m. Because the plate width to thickness ratio was less than 50 and it would be necessary to slice the plates for handling purposes within a restricted working area, the use of the CarboDur strengthening system was specified by the structural engineers, Lawrence Hewitt Partnership. To achieve the desired strengthening requirements, the laminates used were 50 mm wide, 1.2 mm thick with an E-modulus of 155 000 N mm^{-2}.

In competitive tender, Concrete Repairs Ltd (CRL) secured the subcontract to supply and install the CarboDur system for the main contractor John Mowlem Construction Plc.

CRL prepared the floor rib soffits by needle gunning and vacuuming; this method of preparation was checked on site by performing pull-off tests using a 'limpet'. The pull-off values achieved were in the region of 3.0 N mm^{-2} with a failure in the concrete. The 50 mm wide CarboDur laminates arrived on site in 250 m long rolls, preboxed for protection (Fig. 11.1). The individual laminates were then cut on site using a guillotine, cleaned to remove surface contaminants and coated with the epoxy adhesive.

At the same time the concrete bond surface was coated with adhesive (Fig. 11.2). The laminates were then offered up to the beams (Fig. 11.3) and

Figure 11.1 The complete CarboDur strengthening system.

Figure 11.2 Application of adhesive to CarboDur laminate.

Figure 11.3 Positioning an 11.0 m laminate to underside of rib.

Figure 11.4 Bedding CarboDur laminate into adhesive with hand roller.

rollered into position (Fig. 11.4) to ensure good adhesive contact and to eliminate voids. The next day antipeel bolts were installed at the ends of the laminate by drilling through the CarboDur laminate into the concrete and using chemical anchors. The strengthening work was completed within the four weeks programme at a cost of around £60 000 using a total of 1100 m of CarboDur laminates.

11.3.2 Strengthening of the Rhine bridge Oberriet, Meiningen, Switzerland

11.3.2.1 The problem

11.3.2.1.1 Introduction

The three span bridge was built in 1963 and crosses the border over the River Rhine between Austria and Switzerland, linking the towns of Meiningen (Vorarlberg) to Oberriet (St Gallen) (Walser and Steiner, 1997). The end spans are 35.1 m in length with a central span of 45 m (Fig. 11.5).

A thorough investigation and structural analysis according to current SIA (Swiss Engineers and Architects) load standards had shown that besides normal maintenance, the bridge deck was in need of transverse strengthening. This was due to the fact that in 1963, the bridge deck had been designed for standard 14 tonne truck loads.

Figure 11.5 General view of Oberriet bridge.

11.3.2.2 The solution

11.3.2.2.1 Strengthening options

Different solutions were available to guarantee the future structural safety under today's traffic loads.

- replacing the entire bridge deck
- improving the cross-section characteristics by providing a concrete overlay
- strengthening by bonding additional reinforcement to the existing deck.

Because the existing concrete slab was in good condition and the chloride content exceeded the critical values only in the outer 10 mm, it was decided, for economical reasons, not to replace the total deck slab. The additional concrete thickness necessary to reach full flexural capacity would, however, have caused unacceptable longitudinal stress. Therefore bonding additional reinforcement to the deck became the preferred solution. Structural elements strengthened with bonded reinforcement, according to general practice, should have a residual total safety of $d \times R = 1.2$ after failure of the added strengthening. The fact that the required strengthening factor was about 2.15 meant that the sectional area of the deck slab still had to be increased.

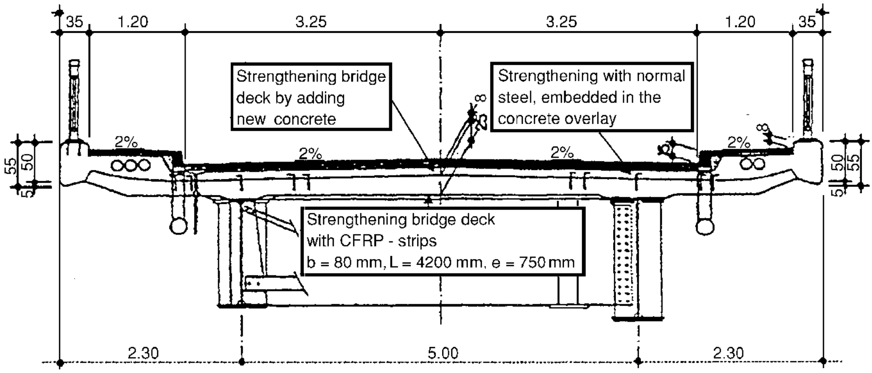

Figure 11.6 Cross-section of Oberriet bridge structure.

It was possible to meet all the structural requirements by bonding transversal strengthening strips and adding 8 cm to the slab thickness (Fig. 11.6). Adding new concrete also allowed the removal of the chloride-contaminated concrete layer by hydro demolition. CarboDur laminates 80 mm wide and 1.2 mm thick were chosen for strengthening. A total of 160 strips, 4 m long, bonded at 75 cm intervals were used.

Thanks to this concept, the bridge today is as good as new and fully meets today's stringent safety standards.

11.3.2.2.2 Structural analysis and design

The stress results for the strengthened cross-section are represented in Fig. 11.7. The zones with negative bending moments were strengthened with normal steel reinforcement embedded in the concrete overlay. The stress transference between the old and new concrete was assured by shear connectors. For the dead loading, before bonding of the CarboDur laminates, stress results indicated an almost zero stress situation in the middle of the deck slab, which leads to the assumption that the existing strain is zero.

Assuming the following values for the calculations:

- Steel II: yield stress, $f_{sy} = 350\,\mathrm{N\,mm^{-2}}$
- Steel S 500: $f_{sy} = 460\,\mathrm{N\,mm^{-2}}$
- Concrete 1963: compressive stress, $f_c = 32.5\,\mathrm{N\,mm^{-2}}$
- Concrete 1996: B45/35: $f_c = 23\,\mathrm{N\,mm^{-2}}$
- CarboDur laminate: $f_{Lu} = 2000\,\mathrm{N\,mm^{-2}}$, $f_{tk} = 3000\,\mathrm{N\,mm^{-2}}$.

The results in the middle of the deck slab were as follows:

- Ultimate bending moment before strengthening, $M_{RO} = 75\,\mathrm{KN\,mm^{-1}}$
- Ultimate bending moment with added concrete, $M_{R1} = 106\,\mathrm{KN\,mm^{-1}}$

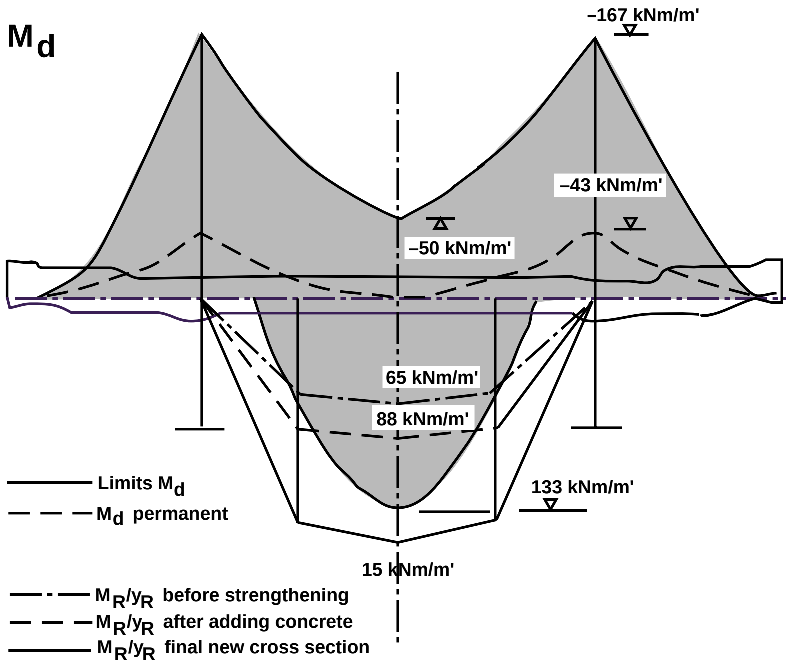

Figure 11.7 Transversal bending moments in the deck slab.

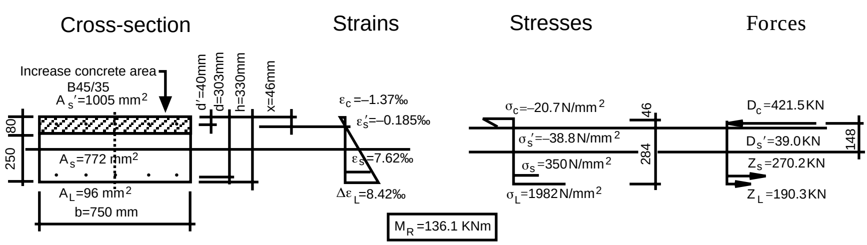

Figure 11.8 Determination of the ultimate bending moment.

- Ultimate bending moment with additional CarboDur strengthening, $M_{R2} = 181.4\,\text{KN mm}^{-1}$.

The results for M_{R2} are represented in Fig. 11.8. The planes of elongation were determined on the basis of mean elongation values, whereas tensile stresses of steel and CarboDur laminates correspond to maximum elongation for the formulation of the conditions of equilibrium. The following coefficients were taken as a ratio of mean to maximum elongation: CFRP strip $K_L = 0.7$/steel $K_s = 0.9$.

Figure 11.8 also shows that failure of the CarboDur laminate strip occurs during the yielding of steel but before failure of the concrete. The total

Figure 11.9 Application of the Sikadur epoxy adhesive.

strengthening factor of 2.4 is the result of increased concrete thickness load factor of 1.4 × CarboDur laminate safety factor of 1.7. Under maximum service load of $m_{ser} = 88.9\,\mathrm{KN\,mm^{-1}}$, the requirement that total safety $d \times R = 1.2$ after failure of the CarboDur laminates is met. The fact that the CarboDur laminates do not have any plastic deformation capability is accounted for by the choice of the value $f_{Lu} = 2/3\ f_{tk}$.

11.3.2.2.3 Application of the CarboDur strengthening system

The underside of the bridge deck was prepared by blast cleaning to expose the coarse and fine aggregate and to give the proper roughness of 0.5–1.0 mm peak trough amplitude for bonding. To assure a good bond of the epoxy adhesive to the concrete immediately prior to the bonding operations, the concrete surface was cleaned by vacuum cleaner to remove all dust.

The pretreated substrate was uneven and full of cavities due to debris within the original cast concrete. Substrate reprofiling work was carried out with a compatible epoxy mortar along the proposed bond line, executed on the day preceding the actual bonding operations. Immediately after the final cleaning of the substrate, the adhesive was applied by trowel onto the concrete (Fig. 11.9).

Owing to the lightweight nature of the CarboDur laminates only two operatives were required to carry the 4.0 m long laminates to the area of application. The two operatives then applied them onto the underside of

Figure 11.10 Installation of CarboDur laminate.

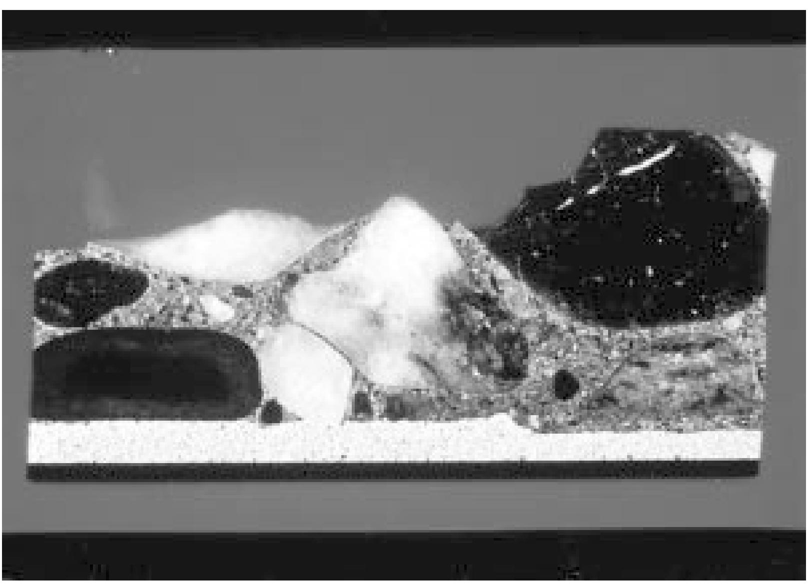

Figure 11.11 Photographic cross-section of core taken from bonded laminate showing adhesive interface and omission of voids, Rhine bridge, Oberriet.

Figure 11.12 Determining the concrete surface preparation by a 'pull-off' test.

the bridge deck (Fig. 11.10). The CarboDur laminates were then carefully pressed on by means of a hard rubber roller. This method of pressing on by roller has successfully been tested on concrete beams at EMPA (Fig. 11.11). Entrapped air in the adhesive interface was checked by means of infrared thermography.

11.3.2.2.4 Quality assurance

After preparation of the substrate, the surface was inspected visually for weak areas, cracks and inclusions in the concrete such as wood. Tensile bond strengths were used to assess the adequacy of the surface preparation by means of pull-off tests with glued-on steel disks (Fig. 11.12). Preliminary investigations had already shown that the concrete of the deck of the Oberriet bridge was of excellent quality. Readings for the tensile bond strength were in the range of 3.3–3.7 N mm^{-2}.

The evenness of the concrete surface was checked with a metal straight edge (Fig. 11.13). The maximum allowable deviation of 5 mm over a length of 2 m and 1 mm over a length of 0.3 m was exceeded for 10% of the surface. Such areas were reprofiled by levelling with a compatible Sika epoxy mortar to the prescribed admissible tolerances.

To assess site mixing during the bonding operations, prisms of the epoxy adhesive, two per day, 12 in total, were prepared for laboratory testing to

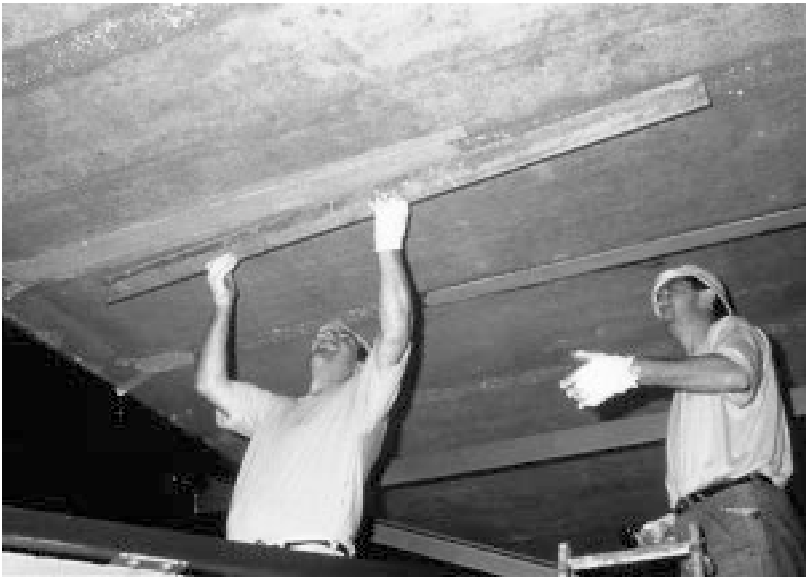

Figure 11.13 Checking the evenness of the reprofiled surface.

Figure 11.14 Completed strengthening work on Rhine bridge, Oberriet.

measure the compressive strength and flexural modulus. A final inspection of the applied plates incorporated a visual inspection and a check for hollow spots by tapping the surface with a small hammer (Fig. 11.14).

11.3.3 Strengthening of masonry walls in an office building, Zurich, Switzerland

11.3.3.1 The problem

11.3.3.1.1 Introduction

Two existing six storey apartment houses built in the 1930s were converted into a large office building. Consequently, a complete redesign of the structural load bearing system was necessary. Furthermore, many items of the present building codes had to be taken into consideration because they differed considerably from those at the time of the original construction, in particular with respect to the earthquake and wind load standards.

Amongst many other alterations, old wooden floors and all of the inner load bearing walls and one entire façade had to be removed and replaced by reinforced concrete slabs and columns. Only parts of the interior unreinforced masonry (URM) fire wall remained in place. These alterations changed both the stiffness and the load bearing capacity of the whole structure.

In the longitudinal direction, two new concrete walls were calculated to resist the earthquake loads. But for the critical transversal direction, only two internal concrete load bearing walls of the staircase and parts of the URM fire wall were available to transmit the horizontal loads down into the foundation. The interior URM fire wall had, therefore, to be strengthened considerably.

11.3.3.2 The solution

11.3.3.2.1 Strengthening options

Three options were considered:

1 demolishing and reconstructing a new fire wall,
2 strengthening the existing wall by applying a reinforced shotcrete skin,
3 strengthening the wall using the CarboDur carbon fibre strengthening system.

The carbon fibre option was chosen based on the following advantages:

- creates minimum interference with other construction work

- no dimensional changes in wall thickness
- cost effective solution to resist earthquake loads
- maintenance free system
- no special tools or heavy equipment required on site
- short duration time on site resulting in a reduction in programme time.

11.3.3.2.2 Strengthening details

The CarboDur strengthening system was utilised on one side of the wall for three storeys (Fig. 11.15) using 100 mm wide strips, 1.2 mm in thickness laid diagonally across the wall (Fig. 11.16). The practical advantage of using the CarboDur laminates was lightness of the material. One long length of laminate was used, therefore, eliminating the need for lap joints. In addition the crossover detail was very simple.

The existing render was removed from the wall and the surface was grit blasted in the areas to be bonded to achieve an open textured profile. Local protuberances were removed mechanically. Prior to the application of the adhesive, the surfaces were vacuum cleaned to remove dust.

The CarboDur laminates were anchored in the adjacent new reinforced concrete column. In order to achieve optimal adhesion between the carbon fibre laminate and the grouting mortar, the anchorage zone of the laminates was slightly curved and provided with a special bonding bridge. In addition to this, steel ties were placed across the laminates and fixed into the concrete with an epoxy resin. All anchorage zones were then grouted with a compatible epoxy mortar.

To ensure the highest quality of work, specialist contractors were used together with the following on site quality assurance programme:

- continuous visual over-all inspection by a supervising engineer,
- adhesion tests on the prepared surfaces,
- dewpoint control of the substrates prior to bonding and grouting,
- sampling of all epoxy batches, as used and mixed on the site, to measure compressive strength and flexural modulus,
- recording of all delivery documents, including production numbers and expiry dates.

11.3.3.2.3 Conclusion

With this strengthening, the lateral resistance and the ductility of the interior URM fire wall could be increased many times over at reasonable costs. It took no more than four days to carry out all the strengthening work. This was the first ever project where carbon fibre laminates had been used to strengthen a masonry wall (Fig. 11.17).

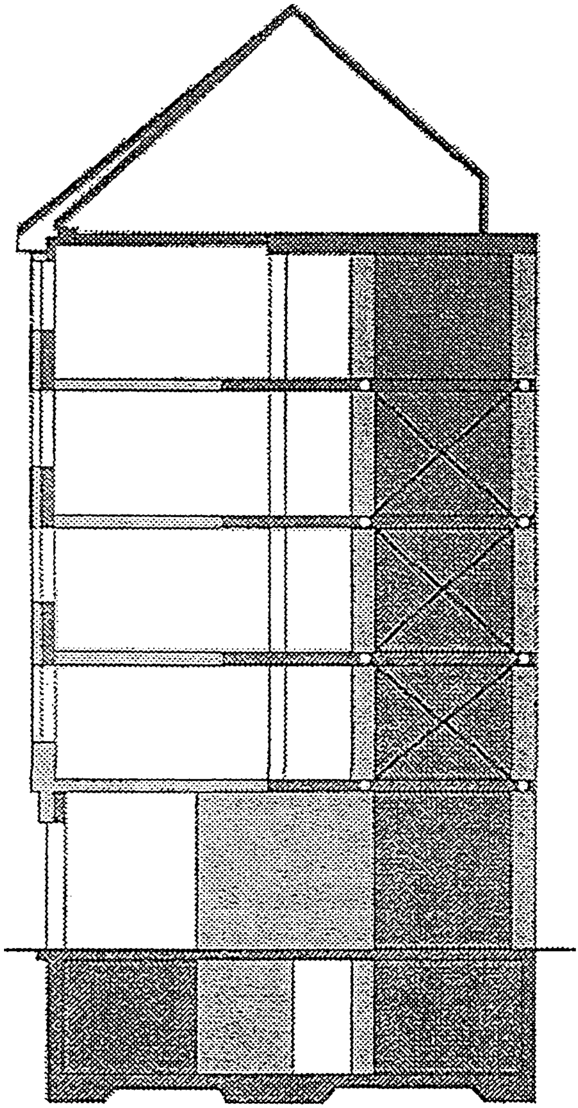

Figure 11.15 Cross-section of the Zurich office building.

Figure 11.16 Application of the CarboDur plates to the masonry wall.

Figure 11.17 URM fire wall strengthened with CarboDur carbon fibre strengthening system.

11.3.4 Strengthening of historic wooden bridge, Switzerland

11.3.4.1 The problem

11.3.4.1.1 Historical background

In 1807 a covered wooden bridge near Sins in Switzerland was designed and constructed to allow horse drawn vehicles to cross over the river (Fig. 11.18). The original supporting structure design consisted of arches strengthened by suspended truss members. On the western side of the bridge this construction can still be seen and is currently in good condition.

During the Civil War in 1847 because the bridge was identified as a strategic crossing point, the eastern side was partially destroyed. In 1852, this section of the bridge was rebuilt with a modified supporting structure made up from a combination of suspended truss members with interlocking tensioning transoms. The present permitted vehicle load carrying capacity of the bridge is 20 tonnes.

During its life the bridge has been rehabilitated in different ways. The most recent investigation which consisted of a load test identified that the wooden pavement and several cross beams were incapable of carrying the current vehicle loading requirements.

Figure 11.18 General view of wooden bridge, Sins, Switzerland.

11.3.4.2 The solution

11.3.4.2.1 Strengthening details

In 1992, strengthening work commenced on the bridge and the original wooden pavement was replaced with 20 cm thick transversely prestressed bonded wooden planks. The most highly loaded cross beams were strengthened using carbon fibre plates bonded to the external surfaces. Each cross beam was constructed from two oak beams placed upon each other separated by wooden blocks to increase the depth (Fig. 11.19). The dimensions of the upper beam were 300 mm by 300 mm and of the lower beam 370 mm deep by 300 mm wide.

To achieve the required bonding surface, the beams had to be prepared. The most suitable method of preparation for this project was achieved by using a portable planing system (Fig. 11.20). The cross beams were strengthened with 1.0 mm thick plates at widths of 250 mm at the upper level and 200 mm at the lower level (Fig. 11.21). Once the preparation was carried out and dust removed, the plates were bonded to the beams (Fig. 11.22).

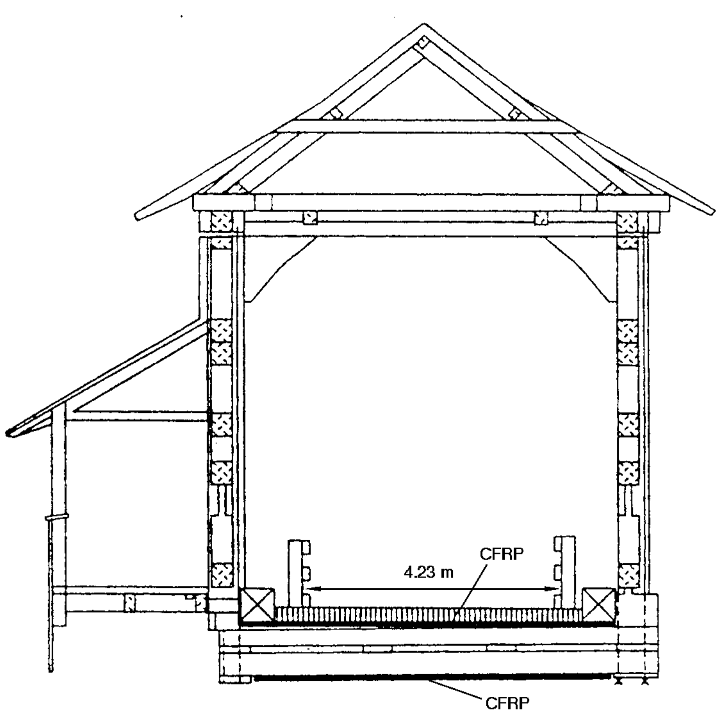

Figure 11.19 Cross-section of the historic bridge near Sins. Selected cross beams were strengthened with carbon fibre plates.

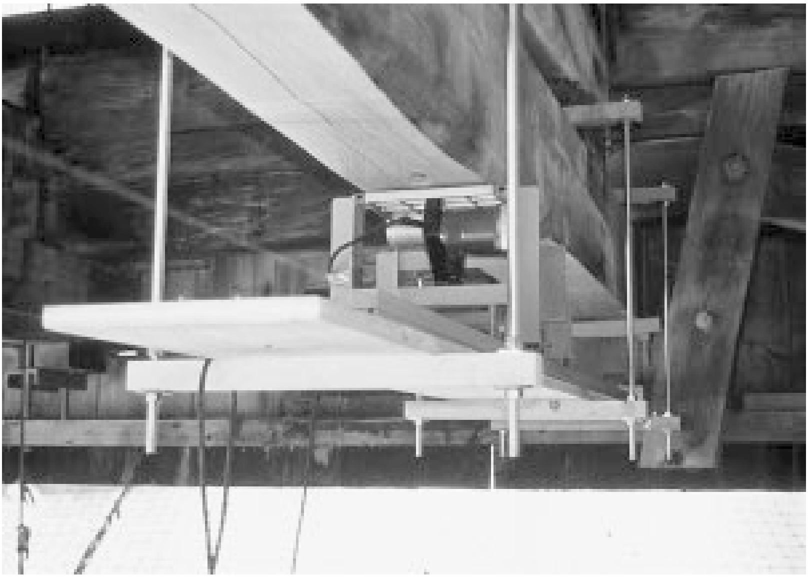

Figure 11.20 Portable system to plane the surface of the wooden cross beams.

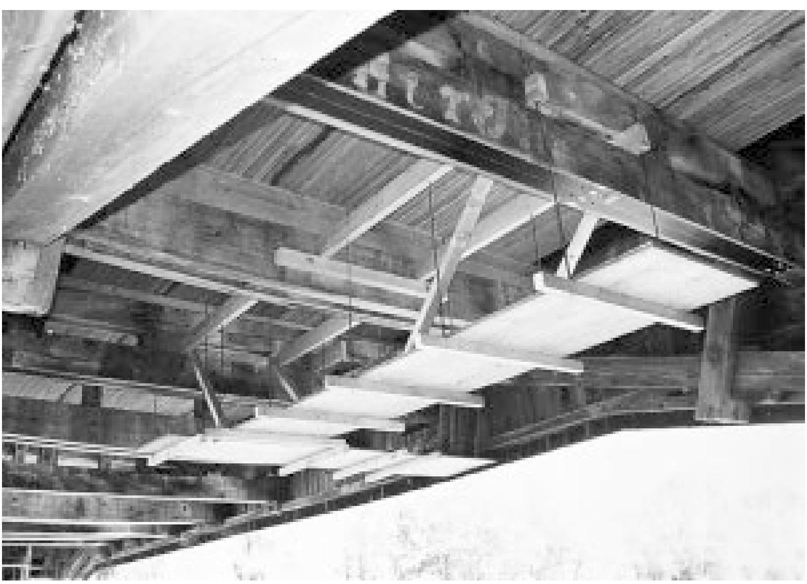

Figure 11.21 View of strengthened cross beams.

Figure 11.22 A close-up of the bonded plate to the underside of the beam.

Figure 11.23 The use of bonded gauge studs to monitor long term performance.

The Swiss Federal Laboratories for Material Testing and Research (EMPA) used pulse infrared thermography to assess the *in situ* suitability of the bonding operation. The use of strain gauges and gauge studs to monitor the long term performance of the strengthening technique is currently being used on selected cross beams (Fig. 11.23).

11.3.4.2.2 Conclusion

The success of this project has given confidence and practical experience in this method for poststrengthening timber structures. For the future preservation of historic bridges and similar structures, this poststrengthening technique offers many advantages over traditional strengthening methods. Because carbon fibre plates are thin, strong and flexible, they can be designed and installed to provide a cost effective solution which does not detract visually from the original design of the structure.

11.3.5 Strengthening subways, Tyne and Wear, UK

11.3.5.1 The problem

Following an assessment of the structure by South Tyneside MBC it was necessary to upgrade the flexural loading capacity of subways to accommodate 40 tonne vehicle loadings.

Figure 11.24 Applying CarboDur laminate to underside of underpass, Tyne and Wear.

Figure 11.25 View of completed bonding operation, subways, Tyne and Wear.

11.3.5.2 The solution

To increase the flexural capacity, 100 mm wide, 1.2 mm thick carbon fibre laminates were bonded to the roof of the subway. The CarboDur laminates and adjacent concrete roof were overlaid with 15.0 mm of prebagged polymer modified Gunite and coated with a high performance protective concrete coating. The total length was 130 m (Figs. 11.24 and 11.25).

11.3.6 Underpass at Great Missenden, Buckinghamshire, UK

11.3.6.1 The problem

The underpass was understrength to sustain current loading requirements of 40 tonnes.

11.3.6.2 Solution

Cracks in the soffit of the deck were first injected with an epoxy resin. Minor concrete repairs were also carried out prior to bonding the 100 mm wide, 1.2 mm thick CarboDur laminates. The total length was 172 m (Figs. 11.26 and 11.27).

Figure 11.26 General view of the underpass, Great Missenden.

Figure 11.27 Completed bonding operation of underpass, Great Missenden.

11.3.7 Strengthening of loggia slabs (balconies), Magdeburg, Olvenstedt, Germany

11.3.7.1 The problem

All the slabs in a multistorey block of flats were sagging due to fatigue and insufficient reinforcement. This reduced the safe use of the balconies. To enable levelling of the upper surface and to compensate for the new dead load and existing live loads, the balconies needed strengthening.

11.3.7.2 The solution

The underside of the loggia slabs was strengthened with three 50 mm wide, 1.2 mm thick carbon fibre laminates. The total length was 1140 m (Figs. 11.28 and 11.29).

Figure 11.28 View showing block of flats, Magdeburg.

Figure 11.29 Underside of deck showing completed strengthening of balconies, Magdeburg.

11.3.8 Strengthening of floors in an old apartment house, Budapest, Hungary

11.3.8.1 The problem

The owner changed the use of the building from an apartment house to an office building. The existing floor slab construction was not sufficient for the new loading requirements and had to be strengthened.

11.3.8.2 The solution

The floor slab soffit was strengthened with 50 mm wide, 1.2 mm thick CarboDur laminates. The total length was 170 m (Figs. 11.30 and 11.31).

Figure 11.30 Apartment house, Budapest.

Figure 11.31 Application of CarboDur laminates.

11.3.9 Strengthening of floor slabs in children's hospital, Brno Hospital, Czech Republic

11.3.9.1 The problem

The hospital had planned to install a new tomograph and it was found that the existing reinforcement was not sufficient for this additional load. The ceiling needed a poststrengthening system to increase the load capacity.

11.3.9.2 The solution

CarboDur laminates 50 mm wide, 1.2 mm thick were applied crosswise to the underside of the floor. The total length was 30 m (Figs. 11.32 and 11.33).

Figure 11.32 Application of CarboDur laminates.

Figure 11.33 Completed installation.

11.3.10 Floor strengthening of town hall, Auckland, New Zealand

11.3.10.1 The problem

During the course of the construction work on the town hall it was found that the two mezzanine floors in the main entrance area did not have sufficient reinforcement to comply with current codes. The engineers needed to design a poststrengthening system that could be installed to increase the live load capacity of the existing floors (Fig. 11.34).

11.3.10.2 The solution

The floor slab soffits were strengthened with 50 mm, 1.2 mm thick CarboDur laminates installed at 600 mm intervals. The total length was 200 m (Fig. 11.35).

Figure 11.34 General view of town hall, Auckland.

Figure 11.35 View showing plates bonded to structural floor slab substrate.

11.3.11 Strengthening of apartment balconies due to high deflections: multistorey flats in Loano, Genova, Italy

11.3.11.1 The problem

The cantilever balconies on a multistorey block of flats had an end deflection in excess of 12 mm, together with associated cracks in the concrete. The balcony slabs had been underdesigned and required strengthening.

11.3.11.2 The solution

Poststrengthening of the balconies was carried out using four plates, 50 mm wide, 1.2 mm thick and 1.2 m in length on the upper surface of the slab and beam. A load test was carried out and the deflection was reduced to 2.0 mm (Fig. 11.36).

Figure 11.36 Load testing of balcony at Loano.

11.3.12 Strengthening of a concrete waffle slab, Stuttgart, Germany

11.3.12.1 The problem

To upgrade the college complex another floor was to be added to the building. To bear the additional load, the original roof slab had to be strengthened to comply with the new floor loadings.

11.3.12.2 The solution

The slab was strengthened with 500 m of 80 mm wide, 1.2 mm thick laminates and 250 m of 50 mm wide, 1.2 m thick laminates. The thin section of the plates allowed the simple detailing of crossovers at the intersections of the beams (Figs. 11.37 and 11.38).

Figure 11.37 Offering CarboDur laminates up to the roof beams, Stuttgart.

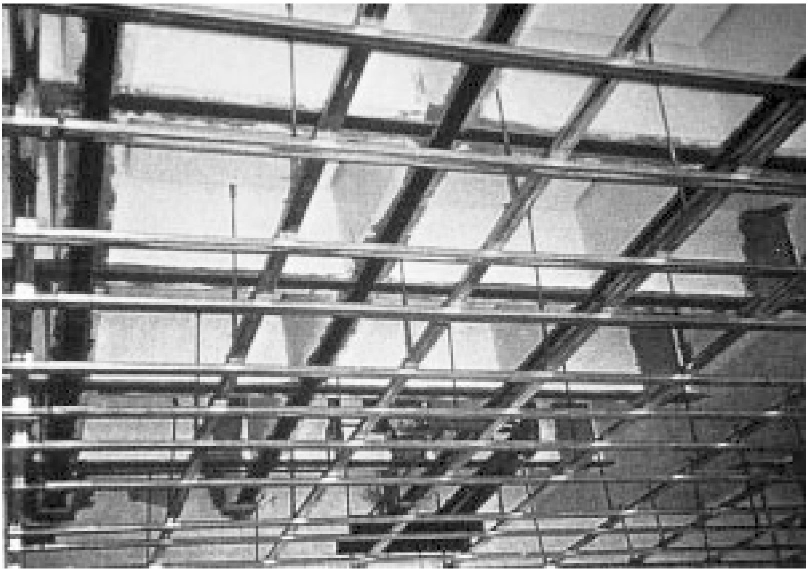

Figure 11.38 The completed project with simple crossover details shown, Stuttgart.

11.3.13 Strengthening of longitudinal concrete bridge beams, Niederwartha near Dresden, Germany

11.3.13.1 The problem

The whole concrete road bridge had to be repaired due to fatigue and environmental influences. An assessment of the bridge found that the longitudinal beams required strengthening to increase the live load capacity (Fig. 11.39).

11.3.13.2 The solution

The beams were strengthened on the underside with a total of 250 m, 80 mm wide, 1.2 mm thick CarboDur laminates and 250 m of 50 mm wide, 1.2 mm thick CarboDur laminates (Fig. 11.40).

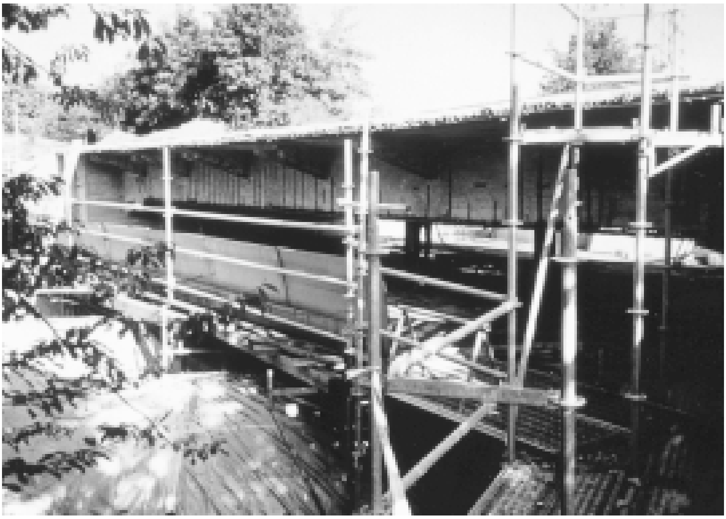

Figure 11.39 General view of repair work, concrete bridge, Niederwarthar near Dresden.

Figure 11.40 View of post-strengthened beam.

11.3.14 Horgen ferry bridge, Switzerland

11.3.14.1 The problem

A deficit of lateral reinforcement in the top deck of this reinforced concrete bridge necessitated a strengthening programme. The lack of reinforcement and resultant moment diagram is shown in Fig. 11.41. The shaded area of the diagram shows the reduced moment 4.0 m from the edge of the bridge parapet.

11.3.14.2 The solution

To overcome this loading problem, strengthening of the deck transversely was carried out in 1997 from the top surface using over 700 m of CarboDur laminates (Fig. 11.42). The thickness of the CFRP plates resulted in minimum alteration to the structure.

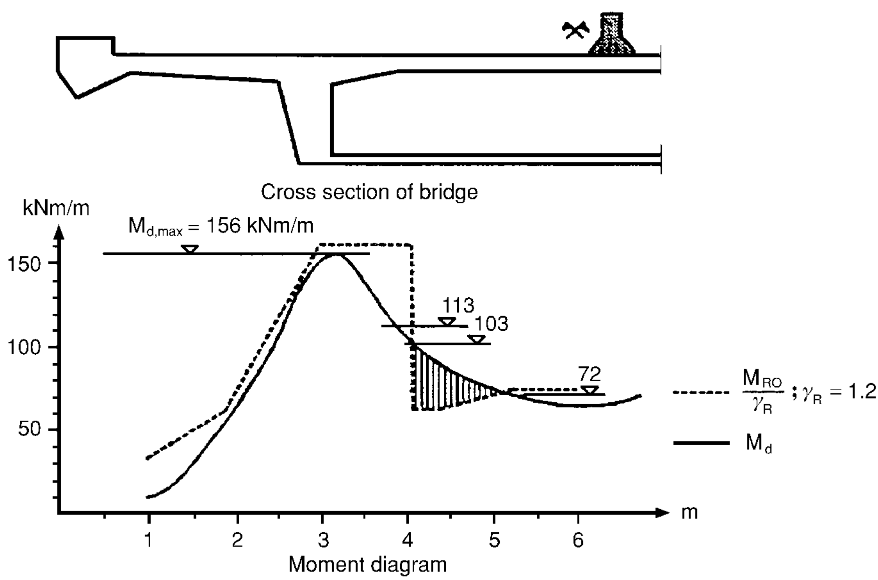

Figure 11.41 Moment diagram for Horgen ferry bridge: lateral moment, solid line; movement of resistance of steel reinforcement, dashed line; moment deficit, shaded area.

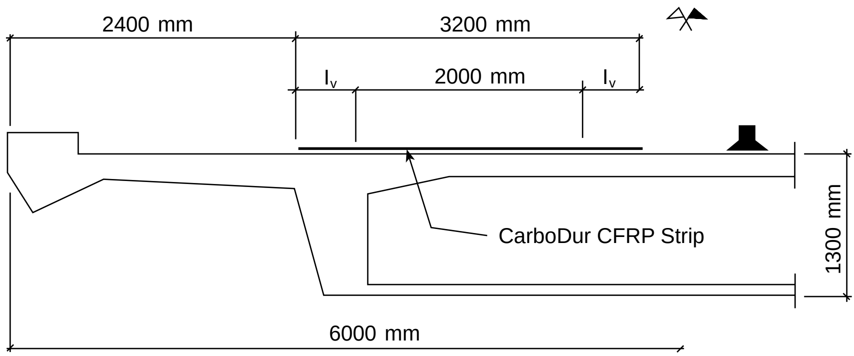

Figure 11.42 Lateral poststrengthening of the deck with a total of 700 m of CarboDur CFRP strips.

11.3.15 Devonshire Place bridge, Skipton, UK

11.3.15.1 The problem

Mouchel was appointed by North Yorkshire County Council to repair Devonshire Place bridge in Skipton, North Yorkshire. The bridge has a precast prestressed concrete hollow section edge beam. A number of the tendons in the edge beam were damaged during an inspection, weakening the edge of the bridge in flexure.

11.3.15.2 The solution

Using knowledge gained from the extensive work done on ROBUST, a single sheet of CarboDur laminate plate was bonded to the underside of the bridge to replace the lost flexural capacity. The traditional approach of bonding steel plates was clearly not suitable for this bridge due to access restrictions. In addition, the existing concrete was not thick enough to support anchor bolts which would have been required if steel plates were used. The CarboDur laminate was bonded to the bridge with Sikadur 30 adhesive. The plate bonding required no bolts or scaffolding and the bridge remained open during the process which was completed within one day.

11.3.16 Nestlé chocolate factory, Tutbury, UK

11.3.16.1 The problem

Mouchel was commissioned by Nestlé to inspect and access the capacity of a number of main beams supporting a factory floor in Tutbury to cater for a 30% increase in floor loading due to the installation of new plant and processing equipment. The brief also stipulated that there was to be minimal disruption to the factory operations.

11.3.16.2 The solution

Mouchel designed the strengthening scheme utilising the CarboDur laminate plate. In total 11 beams were required to have their flexural capacity increased by 30% to cater for the extra loading. This was achieved by bonding one strip of CarboDur laminate plate, 120 mm wide by 1.2 mm thick, centrally along the soffit of each of the beams. Around 100 m of CFRP plate was required for the entire operation. No scaffolding equipment was required, due to the lightness of the CarboDur plates, which meant minimum disruption to the factory operations and short contract duration. Sikadur 30 adhesive was used to bond the plates to the soffits of the beams. The carbon fibre composite bonding operation was completed in less than two days.

11.4 References

Farmer N (1997) 'Strengthening with CFRP laminates', *Construct Repair* **II**(7) 2–4.

Midwinter K N (1997) 'Plate bonding carbon fibre and steel plates', *Construct Repair* **II(**7) 5–9.

Meier U (1995) 'Strengthening of structures using carbon fibre/epoxy composites', *Construct Building Mater* **9**(6) 341–351.

Schwegler G (1994) 'Masonry construction strengthened with fibre composites in seismically endangered zones', *10th European Conference on Earthquake Engineering*, Vienna.

Shaw M (1997) 'Structural strengthening with external plate bonding', *International Conference on Building Envelope Systems & Technology*, Bath, UK, April 1997.

Walser R and Steiner W (1997) 'Strengthening of the Rhine bridge Oberriet-Meiningen', *Proc. Recent Advances in Bridge Engineering – Advanced rehabilitation, durable materials, non-destructive evaluation and management,* ed. U Meier and R Betti, July 1997, pp 127–133.

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