M12: Civil Engineering Applications

Learning Unit-1: M12.1

M12.1: Typical Applications of FRP Composites in Civil Engineering

M12.1.1 FRP Composites Technology Brings Advantages to the American Bridge Building Industry

Introduction

The highway infrastructure continues to face numerous challenges, i.e., increasing growth demands and heavier trucks as well as trying to preserve aging and rapidly deteriorating highway bridges. As we enter into a new millennium, our strategy is to stay ahead of the bridge deterioration curve by focusing on the use of emerging high performance structural materials and innovative quality designs for more durable and reliable structures.

The Transportation Equity Act of 21st Century legislation (TEA-21) legislation launched an important initiative and established the Innovative Bridge Research and Construction (IBRC) Program, which provided materials in bridge applications. The IBRC Program was one of the largest Federal Government funded initiatives in the world; it was crafted to seek new and innovative material technologies for building more durable and effective bridges as well as extending the service life of the continually aging bridge inventory.

M12.1.1.1 Fiber Reinforced Polymer Composites Met the Challenge

Through this pursuit and among many other emerging new materials, the fiber reinforced polymer (FRP) composite technology has been demonstrated with great success for bridge applications. FHWA has been developing research in FRP composites materials over the past 25 years. The development of the advanced FRP composite technology from the aerospace stealth aircraft and commercial industries is an engineer's dream for innovative structural design and application. It has been found that the characteristics of a composites element or system can be tailored and designed to meet any desired specifications. The highly corrosion and fatigue resistance composites materials are making inroads into the civil infrastructure industry. These outstanding composites are among the leading materials in structural engineering applications today.

In the six-year period, the IBRC program funded 246 proposals of high performance materials and concepts in bridge design and construction. Of these applications, 127 are constructed with FRP composite materials. Some of the applications have been or are being demonstrated consistently in several states to capture the performance of the FRP composites under variable environments and to spread the wealth of knowledge gained.

This paper not only discusses the different methods of applying the FRP composite materials but also lists the characteristics, advantages, disadvantages, and opportunities. The author will offer some closing thoughts and recommendations to the engineering community of what additional effort is needed to advance FRP composites into conventional practice. For the purpose of discussions in this paper, bridge applications using FRP composite materials can be categorized into four general groups:

- New bridge construction with bridge deck systems
- New bridge construction with hybrid materials
- Bridge strengthening and/or repair
- Seismic column retrofit
- New Bridge Construction with FRP Deck Systems

About 30 percent of 600,000 bridges are being classified as either functionally obsolete or structural deficient. That does not mean that the deficient bridges are unsafe; they are classified administratively to indicate they require some form of maintenance or major rehabilitation to restore them to their original condition or to their original load carrying capacity. Many bridges have a superstructure consisting of concrete or steel girders topped with a concrete deck. Typically, a concrete bridge deck has a 25 to 40-year life span. In states where deicing salt is heavily used during winter operations, concrete bridge decks that were reinforced with unprotected steel reinforcement are deteriorating rapidly. The FRP composite deck systems have a potential to fill the need of bridge deck replacement and extend the service life of existing structures.

In the new bridge construction category under the IBRC Program, 44 projects employed FRP composite bridge deck systems that come in different shapes and forms. Most of these deck systems are proprietary products that were made of glass fibers and polyester or vinyl ester type resins. Some of these systems are into their third or fourth generation developments and improvements. They were made into either full depth sections to conveniently match the typical existing concrete bridge deck or partial depth sections.

Full depth (minimum 203 mm) panels are formed with pultruded structural shapes and shop-fabricated into modular panels for easy transport and rapid deployment. The cross section geometry takes the form of a truss configuration (top and bottom chords with diagonals, triangles, delta frames, "X-shaped" or other similar variations). The panels are spliced together in the field and are normally fastened onto a floor-beam and/or stringer structural support system. Another deck system is of built-up pre-cured sections consisting of orthogonal, honeycomb cells that act as the core element and sandwiched between two face sheets. A third system consists of a single stage large piece fabrication involving lay-up glass fibers, wrapped hollow cores, and vacuum assisted resin transfer molding. A fourth system, which is a shallower section, is constructed with side-by-side, parallel (127 mm X 127 mm) pultruded tubes acting as the core element and sandwiched between two face sheets. A fifth system is a FRP composite sheet panel stiffened with pultruded tubes placed transversely to the traffic direction. The tubes serve as tension reinforcement in the positive moment region, and the panel serves as a permanent bottom form for a cast-in-place full depth concrete deck. This system requires additional FRP composite reinforcing bars in the top mat.

An excellent example of an effective application with the FRP composite deck system is the replacement of an existing conventional concrete deck on a 60-year old, Warren steel truss. The 34.7-meter simple span truss was posted with a 12.7-metric ton weight restriction. Over the years of resurfacing the roadway and deck, the structure had accumulated numerous layers of asphalt cement, thus reducing its live load capacity. By replacing the deck with FRP composite deck panels, the existing 830-kg/meter^2 -superstructure dead load was reduced to 171 kg/meter^2. The bridge was immediately upgraded to carry more than the current legal load. The removal and replacement of the deck system took less than a month to complete. The cost of the rehabilitation ($876,000) was about one-third the cost of total replacement ($2.34 million) and was fully funded by the New York DOT.

The advantages of an FRP composites deck are lightweight, high strength and high performance, chemical and corrosion resistant, easy construction and handling, rapid project delivery, and in most cases, high quality shop fabrication. Its lightweight (88-171 kg/meter^2 without a wearing surface overlay) reduces the overall superstructure weight and foundation requirement. In areas of high seismic zones, a reduced mass may be highly desirable. Although the composite materials are of high tensile strength, the current deck design is governed by stiffness requirements. The stiffness modulus of glass FRP composites is about one-fifth that of steel. Except for high or ultra high modulus carbon materials, the stiffness modulus of typical carbon fibers is slightly higher than structural steel.

Another important feature of the FRP deck panel systems is its ability to be rapidly deployed and installed at the job site. In reducing congestion in the work zone and improving safety, FRP bridges that had been built from a relatively few short hours to over a weekend are highly desirable and sought after. Bridge owners are willing to pay some premium upfront cost over conventional method of construction that requires prolonged duration. The author predicts that in 10 years from now, long delays through routine construction work zones will no longer be tolerated.

There remain some challenges in the use of FRP composites for deck replacement. The design of an FRP deck system requires finite element analysis. Its lightweight in the superstructure can become aerodynamically unstable, especially for long span structures. As in any new innovation and being an anisotropic material, the composite components and system would require validation testing while building up a good database for each specific system. Depending on how a deck panel is fabricated, consistency and quality may vary. For field installation, connections and some other construction details would need to be developed, improved and tested. A well-designed and properly installed thin bonded overlay can improve traffic traction and extend the service life of the deck panels. Traffic railing connections would need to be designed and tested for crash worthiness. Nondestructive testing/evaluation devices should be incorporated into inaccessible parts of the deck panels to monitor short and long-term performance and to facilitate maintenance inspection. These disadvantages should not be viewed as hindrances. Rather, they should be welcomed as development opportunity for the engineering community and industry. As structural engineers learn more about the behavior of the composites, these problems will be resolved through proper applications, detailing and further research.

The FHWA desires to advance the FRP composites deck applications into conventional practice and is working with the AASHTO Technical Committee to develop guide specification for testing and acceptance of the deck systems. This will include testing standards and protocol to establish strength and stiffness requirements, connections and joints, and manufacturing variations and acceptable defects. The goal of this study is to develop performance specifications that will help bridge owners gain confidence in using composites. The research has been completed and a draft guide specification has been undergoing some peer reviews. A final draft will be submitted to the AASHTO Technical Committee for further review beginning 2004.

M12.1.1.1.1 New Bridge Construction with FRP Hybrid Systems

The United States bridge construction technology and philosophy is based largely on a first-cost basis. Since FRP composite materials have a higher first-cost, hybrid FRP systems that combine the high stiffness and/or high compression strength of conventional materials have been proven effective. Hybrid systems may be classified into two categories - structural composites product with hybrid fibers and structural systems consisting of hybrid composites and conventional materials. The first category involves a product-level definition that is made by combining carbon and glass and/or aramid fibers and resin to form a unidirectional structural element such as laminate or thin plate, rod or tendon, and strand. The second category involves a system-level that is defined by incorporating FRP composite components into a structural member made of conventional materials.

When designing hybrid structures, the key is to strategically place the FRP composites where its high tensile strength can be capitalized, while taking advantage of the high compressive strength and/or high stiffness of conventional materials. The high tensile strength of FRP composites is derived from its unidirectional fibers parallel to the axis of the applied load. In contrast to the high compression strength in concrete, the FRP fibers contribute to the composites' high tensile strength but the resin matrix contributes only a fraction of its compression strength. The use of hybrid systems would help build confidence in conservative bridge engineers who are more comfortable with conventional materials.

In hybrid product systems, the FRP composites are fabricated to form reinforcing bars, tendons, laminates or two-dimensional grids, gratings or flat plates using carbon and/or glass fibers. In bridge applications, the author has seen hybrid fibers being used in wet lay-up fabrics more often than in unidirectional elements. A three-dimensional fabric structure would include grid or cage systems. Although they exist, the author has not yet seen these types of systems being used for bridges.

FHWA is committed to promoting durable bridges. The short life span of conventional steel reinforced bridge decks in a highly corrosive environment attributed to heavy deicing salt use during winter weather operations is unacceptable with today's traffic capacity demands in developing communities. On Federal-aid bridge projects, there is a requirement for corrosion protection strategy (e.g., water proofing membranes and overlays, epoxy coated rebar, concrete cover) for concrete bridge decks. FRP composite reinforcing bars could have a niche for those who are satisfied with the reinforced concrete decks. The unidirectional fiberglass rod may either be coated with sand and/or treated on the surface to mimic the deformed steel bar for enhanced bonding with the concrete matrix. Carbon fiber rods and grids are being demonstrated in some projects. The National Cooperation of Highway Research Program under the Transportation Research Board (NCHRP-TRB) has completed a research study to develop material specifications on FRP composite materials as concrete reinforcing elements. This project has been completed and a report is available through the NCHRP-TRB.

There had been some reported corrosion problem in steel tendons that caused bridge failures in conventional prestressed concrete applications in the U.S. and Europe. Some states have reported corrosion and broken steel tendons in segmental concrete and pedestrian bridges recently. Others have detected voids in grouted post-tensioning ducts. The highly corrosion resistance FRP composites, high strength tendons are showing great potential in prestressed applications. A unidirectional carbon fiber tendon coupled with high performance concrete would eliminate corrosion problem and enable concrete members to last 100 years. One such successful first generation application in pre- and post-tensioned concrete Tee beams is proving that the FRP composites can be designed to overcome its non-ductile behavior. Although an FRP composite component behaves linearly elastic until rupture, it can be designed to behave in a ductile manner in a structural system. By combining both bonded and unbonded tendons in double-Tee prestressed concrete beams, Professor Grace, Lawrence Technological University in Michigan, has demonstrated that the hybrid concrete/FRP composite tendons concrete structural system can be designed to fail with large deflections.

The challenge is to develop efficient and effective anchorage systems for prestressed applications. Current proprietary anchorage systems consisting of various bonded sockets or metal sleeves served as a good start to demonstrate their applications. However, these first generation systems may have undesirable creep in both the resin matrix and adhesives, potential corrosion in the metal sleeves, and shear lag phenomena. Improvement in these areas would propel the advancement of prestressed applications. The FHWA has completed a research study that includes design guide and specifications. The goal of this study is to develop recommendations and guidance for applying FRP prestressing tendons and anchorages in bridges. More information will soon be made available with the release of an FHWA report. ACI 440 has an active working subcommittee for prestressed applications and some guidelines development may be already at work.

In the past few years, there has been some interesting research and development in the glulam timber bridge technology. A thin GFRP laminate consisting two percent of the cross sectional area of a glulam timber beam placed at the extreme tension fibers can increase the flexural strength 170 percent. This hybrid FRP composites/timber beam would reduce the amount of timber material and overall first cost. When vertical clearance is not a problem, the hybrid beam of similar depth can be made to span longer. FRP composite laminates will allow designers to take advantage of the composites' high tensile strength with conventional but lower quality lumber.

The disadvantage in using unidirectional FRP composite rod and tendon elements as substituted materials is a weak commercial incentive. The Germans had developed a high quality prestressed fiberglass tendon, but it could not compete with the steel strand on a first cost basis. As a result, the GFRP tendon never had a chance to go beyond the handful of demonstration bridge projects built in Germany. It is hoped that a potential success in prestressed applications would advance the prestressing tendons into long span cable-stayed and cable suspension bridges. The FRP composites offer the potential to eliminate the problem of excessive cable dead load and corrosion problem on long span bridges. The Swiss constructed a cable-stayed structure replacing two of its 24 stay cables with CFRP materials to demonstrate that it could be done.

Other CFRP and GFRP cylindrical shells are also being used in pilings, crash cushion, and dolphin construction in brackish waters and land structures along the Eastern Coastal states. There is much research needed on design, installation, and connection details as well as durability with these applications in constant wet-dry cycle environment.

M12.1.1.1.2 Bridge Strengthening and Repairs

The second leading application thorough the IBRC program is the bonded concrete repair using FRP laminates, rods and wet lay-up fabrics. The surface mounted composites have been used in numerous concrete bridge strengthening and repair applications. This technique is cost effective, easily to design, install and inspect. Composites applied to the soffit of existing concrete decks have been demonstrated and tested in Europe, Japan and America.

In general, when a structural member is being repaired using FRP composites, it will be much stronger than its original undamaged condition. The FHWA tested to failure a repaired Type II, AASHTO prestressed concrete girder. The repair was hand-wrapped using CFRP fabric and epoxy adhesive system. The repaired beam was 130% stronger than its original design. In bridge strengthening, a reinforced concrete beam's capacity can be increased for shear and/or flexural loads. Concrete slabs could be strengthened for flexural loads. When properly designed and applied, repairs can be done successfully without having to completely replace the whole structure. The work completed by Professor Urs Meier, Swiss Federal Laboratories for Materials Testing and Research (EMPA) in Dubendorf, Switzerland, has provided some excellent guidance for strengthening of structural members with FRP composites [5]. To ensure structural integrity when strengthening existing structural members, it is important to maintain a factor of safety for the slab greater than or equal to 1.0 for the pre-existing reinforcement. The failure mechanism for bonded repairs takes on numerous paths and the effort to quantify the failure modes would require time and further research.

A good repair program should include an evaluation of the preexisting condition and structural integrity of the overall concrete member to establish a baseline reference. For surface mounted applications, the concrete surface requires some preparation to ensure the substrate is sound and of good structural integrity. Deteriorated concrete or delaminations must be removed. Spalled surfaces must be built up to provide a level and flat surface for bonding the aligned fabric sheets or laminate. Sharp edges and corners must be rounded to prevent a knife-edge action on the fabric. When using rods, grooves must be saw-cut in the concrete surface. Moisture problem must be corrected before using composites bonded repairs because ambient-cured organic material adhesives are problematic in wet environments.

For bridge decks exhibiting extensive cracking, covering the entire soffit completely is not advisable. Moisture can accumulate inside the cracks and freezes, aggravating the concrete deterioration. Bonded laminates spaced a distance apart or fabric sheets in grid pattern would allow moisture to pass through a cracked deck; however, long term durability of the bonded repair could be compromised if the unprotected steel reinforcement inside the concrete continues to corrode.

The author has seen surface mounted bonded repairs and strengthening in unreinforced masonry structures. Composites rods were embedded in saw-cut grooves in mortar joints of masonry walls to increase its shear and bending capacities. And if needed, vertical or skewed-angle rods were added to form a grid pattern. Earlier studies have shown that wet lay-up fabrics are effective. Open grid systems are more desirable where inspection access is unhindered.

The author believes that all structural repairs should be accessible for inspection, and some form of monitoring must be instrumented for inaccessible areas. After a structural member has been repaired, the in-service condition of the concrete substrate as well as the performance of the composites should be continuously monitored.

The Japanese used bonded repairs for strengthening their tall masonry, round smoke stacks. As those chimneys aged, they required some repair and strengthening to continue their safe operation today. FRP chimney liners in service up to 20 years has proven FRP survival at high temperature, resistance to chemicals, structural reliability, low life-cycle cost, and low maintenance.

A technique using filament winding consisted of a machine equipped with spools of prepreg carbon tows and mounted on a guide system that winds around the chimney wrapping layers of carbon fibers onto the surface of the structure. The machine travels up and down to form a continuous shell and the prepreg is then cured in place. California used this similar technology to strengthen numerous bridge columns for seismic retrofitting. There will be more discussions of this topic further along in this paper.

The fiber wrap systems are also being used to repair deteriorated concrete piers, pier caps, concrete arch, and damaged beams. This technique is preferred for small and easily accessible locations because it requires only light duty equipment and small work crew. Furthermore, this technique indicates that numerous potential applications in civil engineering structure can be economical feasible and effective.

The FHWA has commissioned a research study through the University of Missouri at Rolla, Missouri to develop guide specifications. The American Concrete Institute, Committee 440 has developed and published design criteria and guidelines. In general, a beam or slab can be effectively strengthening to a greater capacity than its original design without significantly affecting its structural integrity from its secondary effects.

M12.1.1.1.3 Seismic Column Retrofit

The development of the United States seismic design codes has been an evolving process, usually through the real test of time and seismic events. As a result of the Loma Prieta Earthquake in Oakland, the California Transportation Department (CALTRANS) has been leading the retrofit of concrete bridge pier columns using FRP composite materials. The piers that were designed under pre-1971 seismic design codes were found inadequate for shear capacity, ductility and confinement in plastic hinges. The CALTRANS had since retrofitted thousands of concrete pier columns using FRP composite materials. The manufactured products can be classified into three categories: filament winding, hand-wrapped fiber sheets, and pre-cured cylindrical shells. The seismic column retrofit using FRP composite materials has been through a great deal of testing and development, and this application has been accepted as an established method for column strengthening. A well-defined accelerated testing protocol of up to 10,000 hours of exposure to various environmental conditions has been developed by CALTRANS to estimate degradation over the projected service life. Other states are beginning to adopt it in their bridge column retrofit program.

Some Closing Thoughts

We have made excellent progress with the FRP composites technology for bridge applications. The demonstration projects from the IBRC Program gave us a good start in adding FRP composites technology to our toolbox. The high strength, high fatigue resistance, lightweight, and corrosion resistance of composites are highly desirable characteristics for bridge applications. The author considers the sample applications are only a prelude as to what is more to come.

The FRP composite strengthening technology has proven successful, and we will probably see more applications in the future. When we have design and construction specifications developed and adopted bridge owners will be able to extend the service lives of numerous bridges, landmark and historic as well as routine structures at a fraction of the bridge replacement costs.

We need to take research, development and applications into higher levels of exploitations. Long-term durability issues need to be defined because the materials do not have sufficient historical performance data in bridge applications. The last six years of demonstration in bridge applications in the U.S. and accumulative work of others worldwide will serve as a continuous study laboratory in the field. Bridge owners need assurance of long-term integrity of bonded joints and components under cyclic fatigue loading, and they have seen and like what composites have to offer. Improper curing of the resins and moisture absorption and/or ultraviolet light exposure of composites that may affect the strength and stiffness of the structural system should be addressed. Certain resin systems are found ineffective in the presence of moisture and the author believes inorganic resins may be the solution to this problem. We need to follow up and verify their performance and durability.

Currently, most of these new materials are a direct technology transfer from the aerospace industry, and they are far more advanced than those required by civil applications. Most of the advanced composite materials that are cured at high temperature produce high quality and possess excellent characteristics. In bridge applications, resins as the binders for the fiber and adhesives for joints and connections that can adequately cure at ambient temperature and still offer comparable quality and characteristics are more desirable and practical. Standardized bridge components and systems design would allow more focused research, development and competitiveness. More efficient manufacturing and effective production methods for large volume panels and higher modulus materials are needed to make it more cost effective for composites to compete in the civil infrastructure. The direct technology transfer of fiber composites from the aerospace industry is not cost competitive when compared to conventional materials in bridge applications.

M12.1.1.2 Rebuilding the U.S. Transportation Infrastructure

Those challenges mentioned above should not be viewed as barriers but progress to make good on what the materials have promised to deliver. They will serve as opportunities to improve the materials to ensure that the product will be durable and reliable. The current focus for FHWA is to advance the FRP composite technology to rebuild the American transportation infrastructure in new bridge construction as well as the rehabilitation and maintenance of the existing bridge inventory. The rebuilding of the Nation's highway system presents a tremendous market opportunity for those high performance materials in bridge applications well into the 21st Century.

Conclusion

The author has presented numerous successful examples of FRP composites bridge applications. FHWA is advocating structures that not only will last 100 years but also reliable and consistence in performance. The FHWA believes there is a great future with the composite materials and will continue to support research and development in future generations of FRP materials. ACI Committee 440 has taken up on the challenge and is developing some guidance in the use of FRP under its various subcommittees. Numerous publications capturing the success of the FRP composites used in the civil infrastructure are available through the ACI.

The FHWA is developing a database to capture the performance history, design guidelines and specifications used in the demonstration projects. Information is made available to anyone through our Website: (www.fhwa.dot.gov/bridge/frp). We welcome your input and contribution to help us capture and disseminate the knowledge.

The future is bright for FRP composite materials. It is an exciting time for bridge engineers, consultants, researchers and the FRP composites industry. With FRP composites, the Americans are already changing the way they build and maintain their bridges.

M12.2. Adhesively Bonded FRP Composites in Strengthening Of Civil Engineering Structural Components Such As Beams, Columns, Masonry Etc

M12.2 Role of Bonded fibre-Reinforced Composites in Strengthening of Structures

M12.2.1 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. 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.

M12.2.1.1 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 vinyl-ester or epoxy. These materials are preformed to form plates under factory conditions, generally by the pultrusion process. For experimentation, strengthening of reinforced concrete structures 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 pre-stress the structure. Most experimental work has been undertaken by applying composites to concrete, but timber, stone, steel or cast iron may also be strengthened.

M12.2.1.2 The Market for Strengthening

Modern civilization 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 recognize a number of common features:

- Structural deterioration perhaps increased by environmental factors
- Changes in use or imposed loading
- The need to minimize closure or disruption during repairs
- The need to extend useful life whilst minimizing 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.

M12.2.1.3 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.

M12.3 Strengthening Techniques and Miscellaneous Issues

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 M12.1.

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

Table M12.1 Applications for composite plate bonding Structural

Case study: Roof structure warehouse

In 2002, the supporting beams of a roof structure of a stockroom at Brussels in Belgium were strengthened using a hybrid strengthening technique, see Brosens et al. Due to a calculation error, the bearing capacity of the main supporting beams was only half as much as needed. The amount of internal steel tensile reinforcement was unsatisfactory. The beams have a span of 14 m and a rectangular cross section of 400 mm x 800 mm. The roof structure consists of two adjacent spans of these simply supported beams. The concrete beams were prefabricated and already positioned on site. To prevent a delay of the construction works, the option of strengthening was chosen instead of replacing the beams.

The first idea was to provide additional steel plates at the bottom side of the beams. To attain the desired bearing capacity, very long steel plates (8 m) with a relatively large steel cross section (200 mm x 20 mm) would be needed. To avoid such heavy steel plates, which are very difficult to apply and to anchor to the concrete beam, another strengthening concept was chosen. Above the mid support, the two beams were connected to each other in order to obtain a statically undetermined structure. In that way, the bending moments at mid-span of the beams can be reduced and consequently also the amount of additional external reinforcement in the mid-spans.

The quality of the concrete was C30/37 and the steel bar grade was BE500. The bending capacity of the original beams was 460.6 kNm which is sufficient to carry the own weight of the structure (concrete beams and steel deck) but insufficient to carry an eventual maintenance loading of 75 kg/m^2. The design of the additional reinforcement was made, assuming that the permanent weight of the structure is taken isostatically by the beams, whereas the maintenance loading is taken hyperstatically. This leads to an additional bending moment of 475.6 kNm at the mid -span and of 120.8 kNm at the support.

To obtain the required bearing capacity and stiffness of the structure, a steel plate with a cross section of 150mm x 8mm is applied at the bottom side in the mid-span of the beams. To prevent premature delamination failure and plate end shear failure, the steel plate is anchored into the concrete beams by applying 3 bolts M16 at each end of the plate.

At the mid support, the two beams are connected by providing a steel plate (length 7 m, cross section 200 mm x 8 mm) at the top side of the beams. This steel plate will transfer the tensile stresses, originating from the negative (hyperstatic) bending moment at the support. Both ends of the steel plate are anchored to the concrete beam with 4 bolts M16. To enable the transfer of compressive stresses, the opening between the two beams is filled with an epoxy repair mortar. To prevent additional cracking due to thermal forces in the lower part of the beam above the support, additional tensile reinforcement is added in this area. Flexible CFRP sheets were used for that purpose. At each side of the beam, 3 layers of CFRP (length 4 m, equivalent fibre thickness 0.167 mm/layer, width 200 mm) were applied. Finally two stirrups made of CFRP sheets (2 layers, equivalent fibre thickness 0.167 mm/layer, width 300 mm) were provided to take up the increased shear forces due to the connection of the two beams.

Figure M12.4 gives an overview of the hybrid strengthening of the supporting beams of the roof structure. The complete strengthening work took only 2.5 weeks, which obviously is much faster than the alternative of replacing the beams by new ones. Some details of the application of the externally bonded reinforcement are shown in Figure M12.5.

Figure M12.4: Overview of the hybrid strengthening work

Figure M12.5: Detail of the connection of the two beams, showing epoxy mortar filling of original joint between beams, and continuity reinforcement above middle support

Conclusions

A good understanding of the force distribution in externally bonded reinforcement is essential for the design and the application of the technique in practice. The choice of the strengthening material, steel plates or CFRP laminates, is very important. Steel plates are suitable for the enhancement of both strength and stiffness properties whereas CFRP laminates are mostly only suitable for the enhancement of the strength properties.

For the design of the cross sectional area, bending as well as shear forces have to be considered. It is very important to take into account the original stress situation before applying the external reinforcement. Only in that case an appropriate design is possible. According to the eurocode standards, both the ultimate limit state and the serviceability limit state have to be checked.

An important issue is the design of the end anchorage. Since the plate end is a discontinuity, high shear stress concentrations might cause premature peeling failure in the end zones of the ext ernal reinforcement. Therefore, the maximum transferable force and the anchorage length have to be determined. If the force capacity of the connection is insufficient, an additional mechanical anchorage, such as bolts or external stirrups, has to be provided.

M12.4 Review of materials and techniques for plate bonding

M12.4.1 Introduction

This section 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.

M12.4.2 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. Nondestructive 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).

M12.4.2.1 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 polymerization, 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.

Advantages of Epoxy Resins over Other Polymers

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.

M12.4.2.2 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) have 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 is 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
- Post-bonding 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).

M12.4.2.3 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). 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 abrade 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.

M12.4.3 External Strengthening Using Steel Plates

M12.4.3.1 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.

M12.4.3.2 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 post-cracking 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 pre-cracked 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 la 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 width increased (wide, thin plates), the anchorage length will be decreases.

M12.4.3.3 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 1.0×10^{3} N mm^{-2} gave slightly improved strengths when failure occurred by plate separation than strengths given by an adhesive with a modulus of around 1.0×10^{3} 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.

M12.4.3.4 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 recognized 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 minimize 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.

M12.4.4 External strengthening using composite materials

M12.4.4.1 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 M12.2, 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 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 nondestructive 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.

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

Table M12.2: Comparison of characteristics of FRC sheet produced from different fibres (Meier, 1995)

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

M12.4.4.2 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.

M12.4.4.2.1 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 emphasize 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 analyzed 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 N mm^{-2} and the peel stress 6.37 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 N mm^{-2} in shear and 8.10 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 points 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.

M12.4.4.2.2 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 post-strengthening 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 points 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 remainders 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 points 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 over-designed 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.

M12.4.4.3 Prestressing composite plates for strengthening concrete beams

The utilization 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 pre-tensioning 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. Pre-tensioning 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).

M12.5 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.

M12.6 Advantages and Disadvantages of FRP Composite Laminated 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 pre-stressing of the plates.

M12.6.1 Advantages of FRP Composite Plate Bonding

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.
- **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 headroom 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.

M12.6.2 Disadvantages of FRP Composite Plate Bonding

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 manufacturer’s 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 localized, 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.

M12.6.3 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.

M12.6.4 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 has been indicated above, against the risk of potential problems. Those risks have been minimized 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.

M12.7 Conclusions

This introductory chapter can do no more than summarize 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 utilize 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.