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State-of-the-Art Report on Fiber Reinforced Plastic (FRP) Reinforcement for Concrete Structures

Reported by ACI Committee 440

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The use of FRP as reinforcement for concrete structures has been growing rapidly in recent years. This state-of-the-art report summarizes the current state of knowledge on these materials. In addition to the material properties of the constituents, i.e. resins and fibers, design philosophies for reinforced and prestressed elements are discussed. When the available data warrants, flexure, shear and bond behavior, and serviceability of the members has been examined. Strengthening of existing structures with FRPs and field applications of these materials are also presented.

Keywords: analysis; composite materials; concrete; concrete construction; design; external reinforcement; fibers; fiber reinforced plastic (FRP); mechanical properties; polymer resin; prestressed concrete; reinforcement; reinforced concrete; research; structural element; test methods; testing.

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CHAPTER 1—INTRODUCTION AND HISTORY

1.1—Introduction

Fiber Reinforced Plastic (FRP) products were first used to reinforce concrete structures in the mid 1950s (Rubinsky and Rubinsky 1954; Wines et al. 1966). Today, these FRP products take the form of bars, cables, 2-D and 3-D grids, sheet materials, plates, etc. FRP products may achieve the same or better reinforcement objective of commonly used metallic products such as steel reinforcing bars, prestressing tendons, and bonded plates. Application and product development efforts in FRP composites are widespread to address the many opportunities for reinforcing concrete members (Nichols 1988). Some of these efforts are:

- High volume production techniques to reduce manufacturing costs
- Modified construction techniques to better utilize the strength properties of FRP and reduce construction costs
- Optimization of the combination of fiber and resin matrix to ensure optimum compatibility with portland cement
- Other initiatives which are detailed in the subsequent chapters of this report

The common link among all FRP products described in this report is the use of continuous fibers (glass, aramid, carbon, etc.) embedded in a resin matrix, the glue that allows the fibers to work together as a single element. Resins used are thermoset (polyester, vinyl ester, etc.) or thermoplastic (nylon, polyethylene terephthalate, etc.). FRP composites are differentiated from short fibers used widely today to reinforce nonstructural cementitious products known as fiber reinforced concrete (FRC). The production methods of bringing continuous fibers together with the resin matrix allows the FRP material to be tailored such that optimized reinforcement of the concrete structure is achieved. The pultrusion process is one such manufacturing method widely practiced today. It is used to produce consumer and construction products such as fishing rods, bike flags, shovel handles, structural shapes, etc. The pultrusion process brings together continuous forms of reinforcements and combines them with a resin to produce high-fiber volume, directionally oriented FRP products. This, as well as other manufacturing processes used to produce FRP reinforcement for concrete structures, is explained in more detail later in the report.

The concrete industry's primary interest in FRP reinforcement is in the fact that it does not ordinarily cause durability problems such as those associated with steel reinforcement corrosion. Depending on the constituents of an FRP composite, other deterioration phenomena can occur as explained in the report. Concrete members can benefit from the following features of FRP reinforcement: light weight, high specific strength and modulus, durability, corrosion resistance, chemical and environmental resistance, electromagnetic permeability, and impact resistance.

Numerous FRP products have been and are being developed worldwide. Japan and Europe are more advanced than the U.S. in this technology and claim a larger number of completed field applications because their systematic research and development efforts started earlier and because their construction industry has taken a leading role in development efforts.

1.2—History of the U.S. pultrusion industry

Pultrusion of composites took off immediately after the Second World War. In the U.S., a booming post-war economy created a demand for numerous improved recreational products, the first of which was a solid glass FRP fishing pole. Then came golf course flag staffs and ski poles. As the pultrusion industry gained momentum, other markets developed. The 1960s saw use in the electric utility market due to superior compressive and tensile strengths, along with excellent electrical insulating properties. The following decade saw advances in structural shapes and concrete reinforcements, in addition to continuing growth in recreational, electric utility, and such residential products as ladder channels and rails. Today, the automotive, electronic, medical, and aerospace industries all specify highly advanced pultrusions incorporating the latest in reinforcement fibers encapsulated in the most recent resin formulations.

1.3—Evolution of FRP reinforcement

In the 1960s corrosion problems began to surface with steel reinforced concrete in highway bridges and structures. Road salts in colder climates or marine salt in coastal areas accelerated corrosion of the reinforcing steel. Corrosion products would expand and cause the concrete to fracture. The first solution was a galvanized coating applied to the reinforcing bars. This solution soon lost favor for a variety of reasons, but mainly because of an electrolytic reaction between the steel and the zinc-based coating leading to a loss of corrosion protection.

In the late 1960's several companies developed an electrostatic-spray fusion-bonded (powdered resin) coating for steel oil and gas pipelines. In the early 1970s the Federal Highway Administration funded research to evaluate over 50 types of coatings for steel reinforcing bars. This led to the current use of epoxy-coated steel reinforcing bars.

Research on use of resins in concrete started in the late 1960s with a program at the Bureau of Reclamation on polymer-impregnated concrete. Unfortunately, steel reinforcement could not be used with polymer concrete because of incompatible thermal properties. This fact led Marshall-Vega (later renamed Vega Technologies and currently reformed under the name Marshall-Vega Corporation) to manufacture a glass FRP reinforcing bar. The experiment worked and the resultant composite reinforcing bar became a reinforcement-of-choice for polymer concrete.

In spite of earlier research on the use of FRP reinforcement in concrete, commercial application of this product in conventional concrete was not recognized until the late 1970s. At that time, research started in earnest to determine if composites were a significant improvement over epoxy coated steel. During the early 1980s, another pultrusion company, International Grating, Inc., recognized the product potential and entered the FRP reinforcing bar industry.

In the 1980s there was increased use of FRP reinforcing bars in applications with special performance requirements or where reinforcing bars were subjected to severe chemical attack. Perhaps the largest market, then and even today, is for reinforced concrete to support or surround magnetic resonance imaging (MRI) medical equipment. For these structures, the conventional steel reinforcement cannot be used. Glass FRP reinforcing bars have continued to be selected by structural designers over nonmagnetic (nitronic) stainless steel. Composite reinforcing bars have more recently been used, on a selective basis, for construction of some seawalls, industrial roof decks, base pads for electrical and reactor equipment, and concrete floor slabs in aggressive chemical environments.

In 1986, the world's first highway bridge using composite reinforcement was built in Germany. Since then, there have been bridges constructed throughout Europe and, more recently, in North America and Japan. The U.S. and Canadian governments are currently investing significant sums focused on product evaluation and further development. It appears that the largest markets will be in the transportation industry. At the end of 1993, there were nine companies actively marketing commercial FRP reinforcing bars.

1.4—FRP composites

The concrete reinforcing products described in this state-of-the-art report are FRP composites. This class of materials is defined as a polymer matrix, whether thermosetting (e.g., polyester, vinyl ester, epoxy, phenolic) or thermoplastic (e.g., nylon, PET) which is reinforced by fibers (e.g., aramid, carbon, glass). Specific definitions used within the report also include glass-fiber reinforced plastic (GFRP), carbon fiber reinforced plastic (CFRP) and related abbreviations. For a more complete listing of definitions not included in ACI 116R—Cement and Concrete Terminology, see the glossary of terms in Appendix A. A description of FRP composites and their constitutive materials is given in Chapter 2.

The following sections contain a brief description of some of the most successful technologies and products presently available in North America, Japan, and Europe.

1.4.1 *North America*—Nine companies have marketed or are currently marketing FRP reinforcing bars for concrete in North America, including Autocon Composites, Corrosion Proof Products, Creative Pultrusions, International Grating, Marshall Industries Composites, Marshall-Vega Corporation, Polystructures, Polygon, and Pultrall. Current producers offer a pultruded FRP bar made of E-glass (other fiber types also available) with choice of thermoset resin (e.g., isophthalic polyester, vinyl ester). There are a number of other FRP products manufactured for use in concrete construction, for example bars and gripping devices for concrete formwork, products for tilt-up construction, and reinforcement support.

In order to enhance the bond between FRP reinforcing bar and concrete, several companies have explored the use of surface deformations. For example, Marshall-Vega Corporation produced an E-glass FRP reinforcing bar with deformed surface (Pleimann 1991) obtained by wrapping the bar with an additional resin-impregnated strand in a 45-deg helical pattern prior to entering the heated die that polymerizes the resin. The matrix used was a thermosetting vinyl ester resin. Similar reinforcing bars are currently being produced by International Grating under the name KODIAK™ and by Polystructures under the name PSI Fiberbar™.

Polygon Company has produced pultruded bars made of carbon and S-glass fibers and using epoxy and vinyl ester resins for the matrix (Iyer et al. 1991). The bars, 3 mm (0.12 in.) in diameter, are twisted to make a 7-rod strand, 9.5 mm (0.37 in.) in diameter. Prototype applications limited to piles (Florida) and a bridge deck (South Dakota) have been constructed using these FRP strands (see Chapter 8).

International Grating manufactures FRP bars made of E-glass and vinyl ester resin. These reinforcing bars, intended for nonprestressed reinforcement, have diameters varying between 9 and 25 mm (0.35 and 1.0 in.), and can be coated with sand to improve mechanical bond to concrete. The ultimate strength of the bars significantly decreases with increasing diameter. A number of publications dealing with the performance of both the bars and the concrete members reinforced with them is available (Faza 1991; Faza and GangaRao 1991a and 1991b).

In Canada, Pultrall Inc. manufactures an FRP reinforcing bar under the name of Isorod™. This reinforcing bar is made of continuous longitudinal E-glass fibers bound together with a polyester resin using the pultrusion process. The resulting bar has a smooth surface that can be deformed with a helical winding of the same kind of fibers. A thermosetting polyester resin is applied, as well as a coating of sand particles of a specific grain-size distribution. The pitch of the deformations can be adjusted using different winding speeds. A preliminary study carried out during the development of this product (Chaallal et al. 1991; 1992) revealed an optimum choice of constituents (resin and glass fiber), resin pigmentation (color), and deformation pitch. The percentage of glass fibers ranges from 73 to 78 percent by weight, depending on bar diameter. The most common diameters are 9.5, 12.7, 19.1, and 25.4 mm (0.4, 0.5, 0.75 and 1.0 in.). An extensive testing program including thermal expansion, tension at ambient and high temperatures, compression, flexure, shear fatigue on bare bars, and pullout of bars embedded in concrete was conducted (Chaallal and Benmokrane 1993). Results on bond performance and on the flexural behavior of concrete beams reinforced with Isorod™ reinforcing bars were also published (Chaallal and Benmokrane 1993; Benmokrane et al. 1993).

In 1993, a highway bridge in Calgary, Canada (Rizkalla et al. 1994), was constructed with girders prestressed with CFCC™ and Leadline™, two Japanese products (see next section). Also in Canada, Autocon Composites produces NEFMAC™, a grid-type FRP reinforcement, under license from Japan (see next section). To investigate its suitability for bridge decks and barrier walls in the Canadian climate, durability and mechanical properties of NEFMAC™, including creep and fatigue, were evaluated at the National Research Council of Canada (Rahman et al. 1993) through full-scale tests.

1.4.2 *Japan*—Most major general contractors in Japan are participating in the development of FRP reinforcement with or without partners in the manufacturing sector. Reinforcement in the following configurations has been developed: smooth bar (rectilinear fibers), deformed bar (braided, spiral wound, and twilled), twisted-rod strand, tape, mesh, 2-D net, and 3-D web.

In the last ten years, research and development efforts have been reported in a number of technical presentations and publications. Because the majority of these publications is in Japanese, references in this report are only those papers written in English. For reasons of brevity, the discussion is limited to the six types of FRP reinforcement popular in Japan.

CFCC™ is stranded cable produced by Tokyo Rope, a manufacturer of prestressing steel tendons. The cables are made of 7, 19 or 37 twisted carbon bars (Mutsuyoshi et al. 1990a). The nominal diameter of the cables varies between 5 and 40 mm (0.2 and 1.6 in.). The cables are suitable for pretensioning and internal or external post-tensioning (Mutsuyoshi et al. 1990b). Depending on the application, a number of anchorage devices and methods are available (i.e., resin bonded, wedge, and die-cast method). Tokyo Rope formed a partnership with P.S. Concrete Co. to develop the use of CFCC™ in precast concrete structures. In 1988, the two companies participated in the construction of the first Japanese prestressed concrete highway bridge using FRP tendons (Yamashita and Inukai 1990).

Leadline™ is a type of carbon FRP prestressing bar produced by Mitsubishi Chemical, with their Dialead™ (coal tar pitch) fiber materials. Leadline™ is available in 1 to 17 mm (0.04 to 0.67 in.) diameters for smooth round bars and in 5, 8, 12, and 17 mm (0.20, 0.31, 0.47 and 0.67 in.) diameters for deformed (ribbed or indented) surfaces. End anchorages for prestressing are available for 1, 3, and 8 bar tendons. Leadline™ has been used for prestressing (pre and post-tensioning) of bridges and industrial buildings in Japan. Mitsubishi Chemical and Tonen produce a carbon fiber sheet that has been used to retrofit several reinforced concrete chimneys in Japan. Research to study uses of this product to strengthen bridge beams and columns is currently underway at the Federal Highway Administration and the Florida DOT laboratories.

FiBRA™, an aramid FRP bar developed by Mitsui Construction, consists of braided epoxy-impregnated strands. Braiding makes it possible to manufacture efficient large-diameter bars [nominal diameters varying between 3 and 20 mm (0.12 and 0.75 in.)] and provides a deformed surface configuration for mechanical bond with concrete (Tanigaki et al. 1988). A FiBRA™ bar is approximately 60 percent aramid and 40 percent epoxy by volume. Both the composite ultimate strength and the elastic modulus are about 80 percent of the corresponding volume of aramid, with efficiency slightly decreasing as the bar diameter increases. By controlling the bond between braided strands, rigid or flexible bars can be manufactured. The latter is preferable for ease of shipment and workmanship. Before epoxy hardening, silica sand can be adhered to the surface of rigid bars to further improve the mechanical bond with concrete. Field applications include a three-span pedestrian bridge and a post-tensioned flat slab (Tanigaki and Mikami 1990). A residential project using precast-prestressed joists reinforced with FiBRA™ and supporting the first-floor slab was constructed.

Technora™ FRP bar, manufactured by Sumitomo Construction and Teijin (textile industry), is made by pultrusion of straight aramid fibers impregnated with vinyl ester resin (Kakihara et al. 1991). An additional impregnated yarn is spirally wound around the smooth bar before resin curing to improve mechanical bond to concrete. The deformed-surface bar is available in two diameters [6 and 8 mm (0.24 and 0.32 in.)]. Three to 19 single bars can be bundled in one cable for practical applications. Tendon anchorage is obtained by a modified wedge system or bond-type system (Noritake et al. 1990). In the spring of 1991, two full-size bridges (pretensioned and post-tensioned, respectively) were constructed using these tendons.

NEFMAC™ is a 2-D grid-type reinforcement consisting of glass and carbon fibers impregnated with resin (Sugita et al. 1987; Sekijima and Hiraga 1990). It was developed by Shimizu Corporation, one of the largest Japanese general contractors. NEFMAC™ is formed into a flat or curved grid shape by a pin-winding process similar to filament winding. It is available in several combinations of fibers (e.g., glass, carbon, and glass-carbon) and cross sectional areas [5 to 400 mm^2 (0.01 to 0.62 $in.^2$). It has been used in tunnel lining applications, offshore construction and bridge decks. Applications in buildings include lightweight curtain walls (Sugita et al. 1992).

A 3-D fabric made of fiber rovings, woven in three directions, and impregnated with epoxy was developed by Kajima Corporation, another large Japanese general contractor. The production of the 3-D fabric is fully automatic and allows for the creation of different complex shapes, with different fibers and spacings, according to the required performance criteria. This reinforcement was developed for use in buildings in applications such as curtain walls, parapets, partitions, louvers, and permanent formwork (Akihama et al. 1989; Nakagawa et al. 1993). Experimental results and field applications have demonstrated that 3D-FRP reinforced panels have sufficient strength and rigidity to withstand design wind loads and can easily achieve fire resistance for 60 min (Akihama et al. 1988).

1.4.3 *Europe*—Some of the most well known FRP products available in Europe are described below.

Arapree™ was developed as a joint venture between Dutch chemical manufacturer Akzo Nobel and Dutch contractor HBG. It consists of aramid (Twaron™) fibers embedded in an epoxy resin (Gerritse and Schurhoff 1986). The fibers are approximately 50 percent of the composite and are parallel laid. Either rectangular or circular cross sections can be manufactured (Gerritse et al. 1987). The material is preferably used as a bonded tendon in pretensioned applications with initial prestressing force equal to 55 percent of the ultimate value, in order to avoid creep-rupture (Gerritse et al. 1990). For temporary anchoring (pretensioning), polyamide wedges have been developed to carry a prestress force up to the full tendon capacity. Some field applications have been reported (Gerritse 1990) including posts for a highway noise-barrier and a fish ladder at a hydroelectric power plant, both in The Netherlands. Demonstration projects for hollow-core slabs, balcony slabs, and prestressed masonry have also been completed.

Parafil™, a parallel-lay rope, is manufactured in the U.K. by ICI Linear Composites Ltd. (Burgoyne 1988a). These ropes were originally developed for such nonconstruction applications as mooring buoys and offshore platforms, but were found suitable for structural applications when made with stiff fibers such as aramid. Type G Parafil™ (Burgoyne and Chambers 1985) consists of a closely packed parallel core of continuous aramid (Kevlar 49™) fibers contained within a thermoplastic sheath. The sheath maintains the circular profile of the rope and protects the core without adding to its structural properties. Several anchoring mechanisms are possible for this type of rope. However, the preferred one appears to be the internal wedge (or spike) method, which avoids the use of any resin (Burgoyne 1988b). Parafil™ tendons can only be used as unbonded or external prestressing tendons (Burgoyne 1990).

Polystal™ bars are the result of a joint venture started in the late 1970s between two German companies, Strabag Bau-AG (design/contractor) and Bayer AG (chemical). One bar has a diameter of 7.5 mm (0.30 in.) and consists of E-glass fiber and unsaturated polyester resin (Konig and Wolff 1987). A 0.5-mm (0.02-in.) polyamide sheath is applied at the final production stage to prevent alkaline attack and to provide mechanical protection during handling. It is possible to integrate an optical fiber sensor directly into the bar material during production (Miesseler and Wolff 1991) with the purpose of monitoring tendon strain during service. For unbonded, prestressed concrete applications, 19-bar tendons are used (Wolff and Miesseler 1989). The anchorage is obtained by enclosing the tendon in a profiled steel tube and grouting in a synthetic resin mortar. A number of field applications have been reported since 1980 (Miesseler and Wolff 1991), including bridges in Germany and Austria, a brine pit cover (Germany), and the repair of a subway station (France). Among the latest reported projects is a bridge in New Brunswick, Canada.

BRI-TEN™ is a generic FRP composite bar manufactured by British Ropes Ltd. (U.K.). The bar can be made of aramid, carbon or E-glass fibers depending on the intended use. Bars are manufactured from continuous fiber yarns embedded in a thermosetting resin matrix. With a fiber-to-resin ratio of approximately 2:1, smooth bars with diameters varying from 1.7 to 12 mm (0.07 to 0.47 in.) can be made. Experimental studies have been conducted on 45-mm (1.77-in.) nominal diameter strands by assembling 61 individual 5-mm (0.20-in.) diameter bars.

JONC J.T.™ is an FRP cable produced by the French textile manufacturer Cousin Freres S.A. The cable uses either carbon or glass fibers. The cable consists of resin-impregnated parallel fibers encased in a braided sheath (Convain 1988). The resin for the matrix can be polyester or epoxy. This cable is not specifically manufactured for construction applications.

SPIFLEX™ is a pultruded FRP product of Bay Mills (France), which can be made using aramid, carbon, and E-glass (Chabrier 1988). The thermoplastic polymer used as a matrix depends on fiber-type and intended application. Similarly, any cross section shape can be obtained depending on the intended use.

CHAPTER 2—COMPOSITE MATERIALS AND PROCESSES

2.1—Introduction

Composites are a materials system. The term "composite" can be applied to any combination of two or more separate materials having an identifiable interface between them, most often with an interphase region such as a surface treatment used on selected constituents to improve adhesion of that component to the polymer matrix. For this report, composites are defined as a matrix of polymeric material reinforced by fibers or other reinforcement with a discernible aspect ratio of length to thickness.

Although these composites are defined as a polymer matrix that is reinforced with fibers, this definition must be further refined when describing composites for use in structural applications. In the case of structural applications such as FRP composite reinforced concrete, at least one of the constituent materials must be a continuous reinforcement phase supported by a stabilizing matrix material. For the special class of matrix materials with which we will be mostly concerned (i.e., thermosetting polymers), the continuous fibers will usually be stiffer and stronger than the matrix. However, if the fibers are discontinuous in form, the fiber volume fraction should be 10 percent or more in order to provide a significant reinforcement function.

Composite materials in the sense that they will be dealt with in this chapter will be at the "macrostructural" level. This chapter will address the gross structural forms and constituents of composites including the matrix resins, and reinforcing fibers. This chapter also briefly addresses additives and fillers, as well as process considerations and materials-influenced design caveats.

The performance of any composite depends on the materials of which the composite is made, the arrangement of the primary load-bearing portion of the composite (reinforcing fibers), and the interaction between the materials (fibers and matrix).

The major factors affecting the physical performance of the FRP matrix composite are fiber mechanical properties, fiber orientation, length, shape and composition of the fibers, the mechanical properties of the resin matrix, and the adhesion of the bond between the fibers and the matrix.

2.2—The importance of the polymer matrix

Most published composite literature, particularly in the field of composite reinforced concrete, focuses on the reinforcing fibers as the principal load bearing constituent of a given structural composite element. Arguably, reinforcing fibers are the primary structural constituent in composites. However, it is essential to consider and understand the important role that the matrix polymer plays.

The roles of the polymer matrix are to transfer stresses between the reinforcing fibers and the surrounding structure and to protect the fibers from environmental and mechanical damage. This is analogous to the important role of concrete in a reinforced-concrete structure. Interlaminar shear is a critical design consideration for structures under bending loads. In-plane shear is important for torsional loads. The polymer matrix properties influence interlaminar shear, as well as the in-plane shear properties of the composite. The matrix resin also provides lateral support against fiber buckling under compression loading.

For these reasons, emphasis has been placed on the matrix resin throughout this chapter. This philosophy is in no way intended to diminish the primary importance of fibers in determining the mechanical and physical properties of any given composite reinforcement. Rather, the subject has been approached in this fashion to increase the readers' appreciation of the contribution of the polymeric matrix to the overall performance of the composite product and with the goal of encouraging a more balanced direction in future research and development programs.

2.3—Introduction to matrix polymers

A "polymer" is defined as a long-chain molecule having one or more repeating units of atoms joined together by strong covalent bonds. A polymeric material (i.e., a plastic) is a collection of a large number of polymer molecules of similar chemical structure. If, in a solid phase, the molecules are in random order, the plastic is said to be amorphous. If the molecules are in combinations of random and ordered arrangements, the polymer is said to be semi-crystalline. Moreover, portions of the polymer molecule may be in a state of random excitation. These segments of random excitation increase with temperature, giving rise to the temperature-dependent properties of polymeric solids.

Polymer matrix materials differ from metals in several aspects that can affect their behavior in critical structural applications. The mechanical properties of composites depend strongly on ambient temperature and loading rate. In the Glass Transition Temperature (T_g) range, polymeric materials change from a hard, often brittle solid to a soft, tough solid. The tensile modulus of the matrix polymer can be reduced by as much as five orders of magnitude. The polymer matrix material is also highly viscoelastic. When an external load is applied, it exhibits an instantaneous elastic deformation followed by slow viscous deformation. As the temperature is increased, the polymer changes into a rubber-like solid, capable of large, elastic deformations under external loads. If the temperature is increased further, both amorphous and semi-crystalline thermoplastics reach highly viscous liquid states, with the latter showing a sharp transition at the crystalline melting point.

The glass transition temperature of a thermoset is controlled by varying the amount of cross-linking between molecules. For a very highly cross-linked polymer, the transition temperature and softening may not be observed. For a ther-

Table 2.1—Properties of calcium carbonate filled poyester resin [Mallick (1988a)]

Property	Unfilled Iso poyester	Iso poyester filled with 30 phr* $CaCO_3$
Density, g/ml	1.30	1.48
HDT†, C (F)	79 (174)	83 (181)
Flexural strength, MPa (psi)	121 (17,600)	62 (9000)
Flexural modulus, GPa (10^6 psi)	4.34 (0.63)	7.1 (1.03)

* phr = parts per hundred (resin)
† HDT Heat distortion (temperature)

mosetting matrix polymer such as a polyester, vinyl ester or epoxy, no "melting" occurs. In comparison to most common engineering thermoplastics, thermosetting polymers exhibit greatly increased high-temperature and load-bearing performance. Normally, thermosetting polymers char and eventually burn at very high temperatures.

The effect of loading rate on the mechanical properties of a polymer is opposite to that due to temperature. At high loading rates, or in the case of short durations of loading, the polymeric solid behaves in a rigid, brittle manner. At low loading rates, or long durations of loading, the same materials may behave in a ductile manner and exhibit improved toughness values.

2.3.1 *Thermoset versus thermoplastic matrix materials*—Reinforcing fibers are impregnated with polymers by a number of processing methods. Thermosetting polymers are almost always processed in a low viscosity, liquid state. Therefore, it is possible to obtain good fiber wet-out without resorting to high temperature or pressure. To date, thermosetting matrix polymers (polyesters, vinyl esters and epoxies) have been the materials of choice for the great majority of structural composite applications, including composite reinforcing products for concrete.

Thermosetting matrix polymers are low molecular-weight liquids with very low viscosities. The polymer matrix is converted to a solid by using free radicals to effect crosslinking and "curing." A description of the chemical make-up of these materials can be found later in this chapter.

Thermosetting matrix polymers provide good thermal stability and chemical resistance. They also exhibit reduced creep and stress relaxation in comparison to thermoplastic polymers. Thermosetting matrix polymers generally have a short shelf-life after mixing with curing agents (catalysts), low strain-to-failure, and low impact strength.

Thermoplastic matrix polymers, on the other hand, have high impact strength as well as high fracture resistance. Many thermoplastics have a higher strain-to-failure than thermoset polymers. There are other potential advantages which can be realized in a production environment including:

1) Unlimited storage life when protected from moisture pickup or dried before use
2) Shorter molding cycles
3) Secondary formability
4) Ease of handling and damage tolerance

Despite such potential advantages, the progress of commercial structural uses of thermoplastic matrix polymers has been slow. A major obstacle is that thermoplastic matrix polymers are much more viscous and are difficult to combine with continuous fibers in a viable production operation. Recently, however, a number of new promising process options, especially for filament winding and pultrusion have been developed.

In the case of common commercial composite products, the polymer matrix is normally the major ingredient of the composite. However, this is not the case for structural applications such as composite reinforcing bars and tendons for concrete. In unfilled, fiber-reinforced structural composites, the polymer matrix will range between 25 percent and 50 percent (by weight), with the lower end of the range being more characteristic of structural applications.

Fillers can be added to thermosetting or thermoplastic polymers to reduce resin cost, control shrinkage, improve mechanical properties, and impart a degree of fire retardancy. In structural applications, fillers are used selectively to improve load transfer and also to reduce cracking in unreinforced areas. Clay, calcium carbonate, and glass milled fibers are frequently used depending upon the requirements of the application. Table 2.1 illustrates the effects of particulate fillers on mechanical properties.

Filler materials are available in a variety of forms and are normally treated with organo-functional silanes to improve performance and reduce resin saturation. Although minor in terms of the composition of the matrix polymer, a range of important additives, including UV inhibitors, initiators (catalysts), wetting agents, pigments and mold release materials, are frequently used.

Following is a more detailed explanation of the commercial thermosetting matrix polymers used to produce composite concrete reinforcing products including dowel bars, reinforcing rods, tendons and cable stays.

2.4—Polyester resins

Unsaturated polyester (UP) is the polymer resin most commonly used to produce large composite structural parts. The Composites Institute estimates that approximately 85 percent of U.S. composites production is based on unsaturated polyester resins. As mentioned earlier, these resins are typically in the form of low viscosity liquids during processing or until cured. However, partially processed materials containing fibers can also be used under specific conditions of temperature and pressure. This class of materials has its

Table 2.2—Physical properties of neat-cured resin castings [Ashland Chemical, Inc. (1993)]

	7241 Iso polyester	980-35 Vinyl ester	D-1618 Vinyl ester	D-1222 Vinyl ester
Barcol hardness	50	45	45	40
Tensile strength MPa (psi)	78.6 (11,400)	87.6 (12,700)	89.6 (13,000)	79.3 (11,500)
Tensile modulus MPa (10^5 psi)	3309 (4.8)	3309 (4.8)	3171 (4.6)	3241 (4.7)
Tensile elongation at break, percent	2.9	4.2	5.2	3.9
Flexural strength MPa (psi)	125.5 (18,200)	149.6 (21,700)	149.6 (21,700)	113.7 (16,500)
Flexural modulus MPa (10^5 psi)	3447 (5.0)	3379 (4.9)	3379 (4.9)	3654 (5.3)
Heat distortion temperature, C (F)	109 (228)	133 (271)	119 (252)	141 (296)

own terminology, with the most common preproduction forms of partially reacted or chemically-thickened materials being prepreg (pre-impregnation, see Terminology in Appendix A) and sheet molding compound (SMC).

Unsaturated polyesters are produced by the polycondensation of dihydroxyl derivatives and dibasic organic acids or anhydrides, yielding resins that can be compounded with styrol monomers to form highly cross-linked thermosetting resins. The resulting polymer is then dissolved in a reactive vinyl monomer such as styrene. The viscosity of the solutions will depend on the ingredients, but typically range between 200 to 2000 centipoises (cps). Addition of heat and/or a free-radical initiator such as an organic peroxide, causes a chemical reaction that results in nonreversible cross-linking between the unsaturated polyester polymer and the monomer. Room temperature cross-linking can also be accomplished by using peroxides and suitable additives (typically promoters). Cure systems can be tailored to optimize processing.

There are several common commercial types of unsaturated polyester resin:

Orthophthalic polyester (Ortho polyester)—This was the original form of unsaturated polyester. Ortho polyester resins include phthalic anhydride and maleic anhydride, or fumaric acid. Ortho polyesters do not have the mechanical strength, moisture resistance, thermal stability or chemical resistance of the higher-performing isophthalic resin polyesters or vinyl esters described below. For these reasons, it is unlikely that ortho polyesters will be used for demanding structural applications such as composite-reinforced concrete.

Isophthalic polyester (Iso polyester)--These polymer matrix resins include isophthalic acid and maleic anhydride or fumaric acid. Iso polyesters demonstrate superior thermal resistance, improved mechanical properties, greater moisture resistance, and improved chemical resistance compared to ortho polyesters. Iso polyester resins are more costly than ortho polyester resins, but are highly processable in conventional oriented-fiber fabricating processes such as pultrusion.

Vinyl esters (VE)—Vinyl ester resins are produced by reacting a monofunctional unsaturated acid, (i.e., methacrylic or acrylic acid) with a bisphenol di-epoxide. The polymer has unsaturation sites only at the terminal positions, and is mixed with an unsaturated monomer such as styrene. Vinyl esters process and cure essentially like polyesters and are used in many of the same applications. Although vinyl esters are higher in cost than ortho or iso polyesters, they provide increased mechanical and chemical performance. Vinyl esters are also known for their toughness, flexibility and improved retention of properties in aggressive environments including high pH alkali environments associated with concrete. For these reasons, many researchers believe that vinyl esters should be considered for composite-reinforced concrete applications.

Bisphenol A fumarates (BPA)—Bisphenol A fumarates offer high rigidity, improved thermal and chemical performance compared to ortho or iso polyesters.

Chlorendics—These resins are based on a blend of chlorendic (HET) acid and fumaric acid. They have excellent chemical resistance and provide a degree of fire retardancy due to the presence of chlorine. There are also brominated polyesters having similar properties and performance advantages.

The following table shows the mechanical/physical properties of iso polyester and vinyl esters in the form of neat (unreinforced) resin castings. These resins can be formulated to provide a range of mechanical/physical properties. The data in Table 2.2 are offered to help researchers and designers to better appreciate the performance flexibility inherent in polymer matrix composites.

Table 2.3 shows a comparison of several common thermosetting resins with similar glass fiber reinforcement at 40 percent by weight of the composite. Note the differences between these resins in key engineering properties even at this low level of identical reinforcement.

2.5—Epoxy resins

Epoxy resins are used in advanced applications including aircraft, aerospace, and defense, as well as many of the first-generation composite reinforcing concrete products currently available in the market. These materials have certain attributes that can be useful in specific circumstances. Epoxy resins are available in a range of viscosities, and will work with a number of curing agents or hardeners. The nature of epoxy allows it to be manipulated into a partially-cured or advanced cure state commonly known as a “prepreg.” If the prepreg also contains the reinforcing fibers, the resulting tacky lamina (see Terminology in Appendix A) can be positioned on a mold (or wound if it is in the form of a tape) at room temperature. Epoxy resins are more expensive than commercial polyesters and vinyl esters.

Table 2.3—Mechanical properties of reinforced resins [from Dudgeon (1987)]

Material	Glass content, percent	Barcol hardness	Tensile strength, MPa (ksi)	Tensile modulus, MPa (10^6 psi)	Elongation, percent	Flexural strength, MPa (ksi)	Flexural modulus, MPa (10^6 psi)	Compressive strength, MPa (ksi)
Orthophthalic	40	—	150 (22)	5.5 (0.8)	1.7	220 (32)	6.9 (1.0)	—
Isophthalic	40	45	190 (28)	11.7 (1.7)	2.0	240 (35)	7.6 (1.1)	210 (30)
BP A-fumerate	40	40	120 (18)	11.0 (1.6)	1.2	160 (23)	9.0 (1.3)	180 (26)
Chlorendic	40	40	140 (20)	9.7 (1.4)	1.4	190 (28)	9.7 (1.4)	120 (18)
Vinyl ester	40	—	160 (23)	11.0 (1.6)	—	220 (32)	9.0 (1.3)	120 (30)

Because many of the first generation commercial composite products for reinforcing concrete are based on epoxy resins, these resins are treated throughout this chapter in slightly greater detail than the preceding polyesters and specialty premium corrosion resins. However, it is believed that second-generation composite reinforcing products for concrete will likely be based on new specialty polyesters with higher retention of tensile elongation properties and improved alkali resistance.

Although some epoxies harden at temperatures as low as 80 F (30 C), all epoxies require some degree of heated postcure to achieve satisfactory high temperature performance. Several suppliers now offer specially formulated epoxies which, when heated, have viscosities low enough to be compatible with the process parameters of a new generation of resin-infusion processes (see Terminology in Appendix A). Large parts fabricated with epoxy resin exhibit good fidelity to the mold shape and dimensions of the molded part. Epoxy resins can be formulated to achieve very high mechanical properties. There is no styrene or other monomer released during molding. However, certain hardeners (particularly amines), as well as the epoxy resins themselves, can be skin sensitizing, so appropriate personal protective procedures must always be followed. Some epoxies are also more sensitive to moisture and alkali. This behavior must be taken into account in determining long term durability and suitability for any given application.

The raw materials for most epoxy resins are low-molecular-weight organic liquid resins containing epoxide groups. The epoxide group comprises rings of one oxygen atom and two carbon atoms. The most common starting material used to produce epoxy resin is diglycidyl ether of bisphenol-A (DGEBA), which contains two epoxide groups, one at each end of the molecule. Other materials that can be mixed with the starting liquid include dilutents to reduce viscosity and flexibilizers to improve impact strength of the cured epoxy resin.

Cross-linking of epoxies is initiated by use of a hardener or reactive curing agent. There are a number of frequently used curing agents available. One common commercial curing agent is diethylenetriamine (DETA). Hydrogen atoms in the amine groups of the DETA molecule react with the epoxide groups of DGEBA molecules. As this reaction continues, DGEBA molecules cross-link with each other and a three dimensional network is formed, creating the solid cured matrix of epoxy resins.

Curing time and increased temperature required to complete cross-linking (polymerization) depend on the type and amount of hardener used. Some hardeners will work at room temperature. However, most hardeners require elevated temperatures. Additives called accelerators are sometimes added to the liquid epoxy resin to speed up reactions and decrease curing cycle times.

The continuous use temperature limit for DGEBA epoxy is 300 F (150 C). Higher heat resistance can be obtained with epoxies based on novalacs and cycloaliphatics. The latter will have continuous use temperature capability of up to 489 F (250 C). The heat resistance of an epoxy is improved if it contains more aromatic rings in its basic molecular chain.

If the curing reaction of epoxy resins is slowed by external means, (i.e., by lowering the reaction temperature) before all the molecules are cross-linked, the resin would be in what is called a B-staged form. In this form, the resin has formed cross-links at widely spaced positions in the reactive mass, but is essentially uncured. Hardness, tackiness, and the solvent reactivity of these B-staged resins depends on the degree of curing. Curing can be completed at a later time, usually by application of external heat. In this way, a prepreg, which in the case of an epoxy matrix polymer is a B-staged epoxy resin containing structural fibers or suitable fiber array, can be handled as a tacky two-dimensional combined reinforcement and placed on the mold for manual or vacuum/pressure compaction followed by the application of external heat to complete the cure (cross-linking).

Hardeners for epoxies—Epoxy resins can be cured at different temperatures ranging from room temperature to elevated temperatures as high as 347 F (175 C). Post curing is usually done.

Epoxy polymer matrix resins are approximately twice as expensive as polyester matrix materials. Compared to polyester resins, epoxy resins provide the following general performance characteristics:

- A range of mechanical and physical properties can be obtained due to the diversity of input materials
- No volatile monomers are emitted during curing and processing
- Low shrinkage during cure
- Excellent resistance to chemicals and solvents
- Good adhesion to a number of fillers, fibers, and substrates

Fig. 2.2 shows the effects of various epoxy matrix formulations on the stress-strain response of the matrix.

There are some drawbacks associated with the use of epoxy matrix polymers:

- Matrix cost is generally higher than for iso polyester or vinyl ester resins

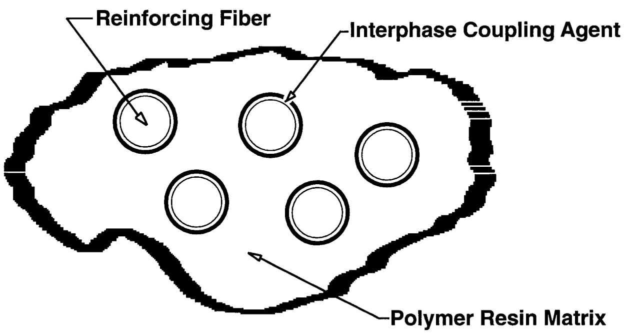

Fig. 2.1—Composite structure at the micro-mechanical level [Composites Institute/SPI (1994)]

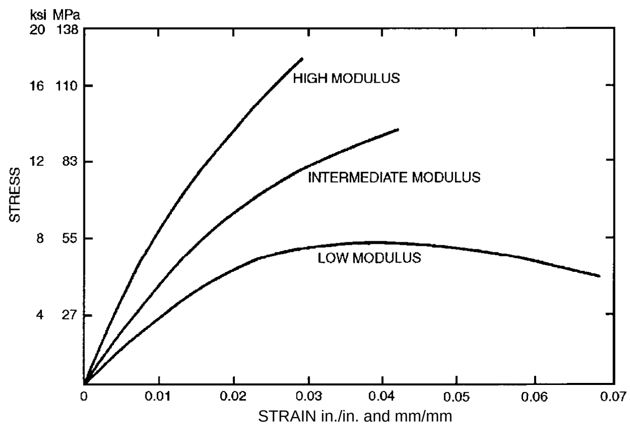

Fig. 2.2—Stress-strain diagram for three epoxy materials [Schwarz (1992a)]

- Epoxies must be carefully processed to maintain moisture resistance
- Cure time can be lengthy
- Some hardeners require special precautions in handling, and resin and some hardeners can cause skin sensitivity reactions in production operations

2.6—Processing considerations associated with polymer matrix resins

The process of conversion of composite constituents to final articles is inevitably a compromise between material physical properties and their manipulation using a variety of fabricating methods. This part will further explore this concept and comment on some of the limiting shape and/or functional characteristics that can arise as a consequence of these choices.

Processability and final part quality of a composite material system depends in large degree on polymer matrix characteristics such as viscosity, melting point, and curing conditions required for the matrix resin. Physical properties of the resin matrix must also be considered when selecting the fabricating process that will be used to combine the fibers and shape the composite into a finished three-dimensional element. As previously mentioned, it is difficult to impregnate or wet-out fibers with very high viscosity matrix polymers (including most thermoplastics), some epoxies and chemically thickened composite materials systems.

In some cases, the viscosity of the matrix resin can be lowered by selected heating, as in the case of thermoplastics and certain epoxies. SMC materials are compounded with fibers at a lower matrix viscosity. The matrix viscosity is increased in a controlled manner using chemical thickening reactions to reach a molding viscosity of several million cps within a desired time window. Processing technologies such as viscosity and thickening control have significant implications for auxiliary processing equipment, tooling, and potential constraints on the shape and size of fabricated parts.

2.7—Structural considerations in processing polymer matrix resins

In general, the concept is simple. The matrix resin must have significant levels of fibers within it at all important load-bearing locations. In the absence of sufficient fiber reinforcement, the resin matrix may shrink excessively, can crack, or may not carry the load imposed upon it. Fillers, specifically those with a high aspect ratio, can be added to the polymer matrix resin to obtain some measure of reinforcement. However, it is difficult to selectively place fillers. Therefore, use of fillers can reduce the volume fraction available for the load-bearing fibers. This forces compromises on the designer and processor.

Another controlling factor is the matrix polymer viscosity. Reinforcing fibers must be fully wetted by the polymer matrix to insure effective coupling and load transfer. Thermoset polymers of major commercial utility either have suitably low viscosity, or this can be easily managed with the processing methods utilized. Processing methods for selected thermoplastic polymers having inherently higher viscosity are just now being developed to a state of prototype practicality.

2.8—Reinforcing fibers for structural composites

Principal fibers in commercial use for production of civil engineering applications, including composite-reinforced concrete, are glass, carbon, and aramid. The most common form of fiber-reinforced composites used in structural applications is called a laminate. Laminates are made by stacking a number of thin layers (laminate) of fibers and matrix and consolidating them into the desired thickness. Fiber orientation in each layer as well as the stacking sequence of the various layers can be controlled to generate a range of physical and mechanical properties.

A composite can be any combination of two or more materials so long as there are distinct, recognizable regions of each material. The materials are intermingled. There is an interface between the materials, and often an interphase region such as the surface treatment used on fibers to improve matrix adhesion and other performance parameters via the coupling agent.

Performance of the composite depends upon the materials of which the composite is constructed, the arrangement of the primary load-bearing reinforcing fiber portion of the composite, and the interaction between these materials. The major factors affecting performance of the fiber matrix com-

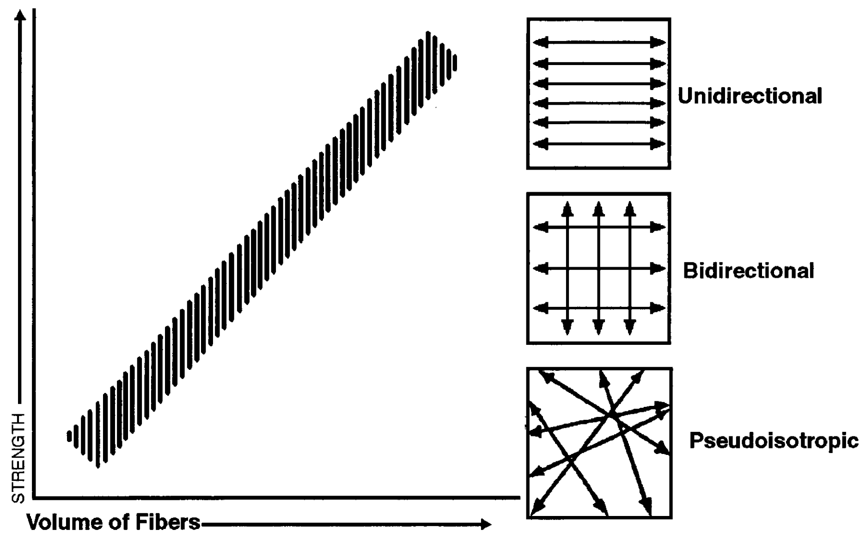

Fig. 2.3—Strength relation to fiber orientation [Schwarz (1992b)]

posite are; fiber orientation, length, shape and composition of the fibers, the mechanical properties of the resin matrix, and the adhesion or bond between the fibers and the matrix.

A unidirectional or one-dimensional fiber arrangement is anisotropic. This fiber orientation results in a maximum strength and modulus in the direction of the fiber axis. A planar arrangement of fibers is two-dimensional and has different strengths at all angles of fiber orientation. A three-dimensional array is isotropic but has substantially reduced strengths over the one-dimensional arrangement. Mechanical properties in any one direction are proportional to the amount of fiber by volume oriented in that direction as shown in Fig. 2.3.

2.8.1 *Fiber considerations*--The properties of a fiber-reinforced composite depend strongly on the direction of measurement in relationship to the direction of the fibers. Tensile strength and modulus of a unidirectionally reinforced laminate are maxima when these properties are measured in the longitudinal direction of the fibers. At other angles, properties are reduced. Similar angular dependance is observed for other physical and mechanical properties.

Metals exhibit yielding and plastic deformation or ductility under load. Most fiber-reinforced composites are elastic in their tensile stress-strain characteristics. The heterogeneous nature of fiber/polymer composite materials provides mechanisms for high energy absorption on a micro-scale comparable to the metallic yielding process. Depending on the type and severity of external loads, a composite laminate may exhibit gradual deterioration of properties.

Many fiber-reinforced composites exhibit high internal damping properties. This leads to better vibrational energy absorption within the material and reduces transmission to adjacent structures. This aspect of composite behavior may be relevant in civil engineering structures (bridges, highways, etc.) that are subject to loads that are more transitory and of shorter duration than sustained excessive loadings.

2.8.2 *Functional relationship of polymer matrix to reinforcing fiber*—The matrix gives form and protection from the external environment to the fibers. Chemical, thermal, and electrical performance can be affected by the choice of matrix resin. But the matrix resin does much more than this. It maintains the position of the fibers. Under loading, the matrix resin deforms and distributes the stress to the higher modulus fiber constituents. The matrix should have an elongation at break greater than that of the fiber. It should not shrink excessively during curing to avoid placing internal strains on the reinforcing fibers.

If designers wish to have materials with anisotropic properties, then they will use appropriate fiber orientation and forms of uni-axial fiber placement. Deviations from this practice may be required to accommodate variable cross-sections and can be made only within narrow limits without resorting to the use of shorter axis fibers or by alternative fiber re-alignment. Both of these design approaches inevitably reduce the load-carrying capability of the molded part and will probably also adversely affect its cost effectiveness. On the other hand, in the case of a complex part, it may be necessary to resort to shorter fibers to reinforce the molding effectively in three dimensions. In this way, quasi-isotropic properties can be achieved in the composite. Fiber orientation also influences anisotropic behavior as shown in Fig. 2.4.

2.8.3 *Effects of fiber length on laminate properties*—Fiber placement can be affected with both continuous and short fibers. Aside from the structural implications noted earlier in this chapter, there may be part or process constraints which impose choice limitations on designers. The alternatives in these cases may require changes in composite part cross section area or shape. Variables in continuous-fiber manufacture, as well as in considerations in part fabrication, make it impossible to obtain equally stressed fibers throughout their length without resorting to extraordinary measures.

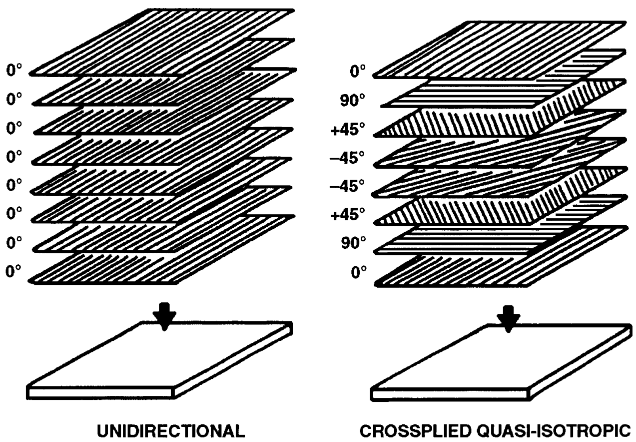

Fig. 2.4—Varying fiber orientation in laminate construction [Schwarz (1992c)]

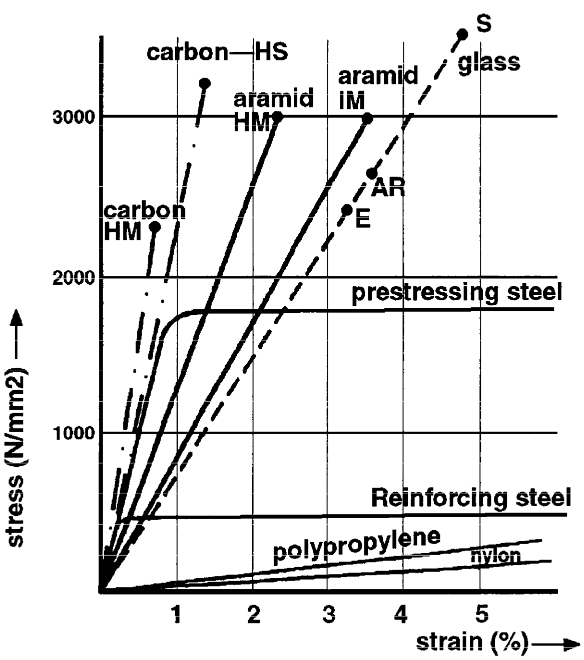

Fig. 2.5—Tensile stress-strain behavior of various reinforcing fibers (Gerritse and Schurhoff)

2.8.4 *Bonding interphase*—Fiber composites are able to withstand higher stresses than can their constituent materials because the matrix and fibers interact to redistribute the stresses of external loads. How well the stresses are distributed internally within the composite structure depends on the nature and efficiency of the bonding. Both chemical and mechanical processes are thought to be operational in any given structural situation. Coupling agents are used to improve the chemical bond between reinforcement and matrix since the fiber-matrix interface is frequently in a state of shear when the composite is under load.

2.8.5 *Design considerations*—Although classical stress analysis and finite element analysis techniques are used, the design of fiber-reinforced composite parts and structures is not a “cook book” exercise. These materials are generally more expensive on a per-pound basis, but are frequently quite cost competitive on a specific-strength basis (i.e., dollars per unit of load carried, etc.). With the exception of the higher-cost carbon fibers, the modulus of fiber-reinforced composites is significantly lower than conventional materials. Therefore, innovative design in respect to shape, fiber choice, fiber placement, or hybridization with other fibers must be utilized by designers to take this factor into account.

The following considerations are representative of the choices which are commonly made:

- Composites are anisotropic and can be oriented in the direction(s) of the load(s) required
- There is a high degree of design freedom. Variations in thickness and compound part geometry can be molded into the part
- Compared to traditional designing, with composites there is usually plenty of tensile (fiber strength) but not comparable stiffness unless carbon fibers are involved. In the case of carbon fiber usage, designers may have to be concerned about impact and brittleness

Table 2.5 may help put these considerations in perspective.

Additional design considerations which should be considered include:

- Designing to provide the maximum stiffness with the minimum materials

Table 2.4—Comparison of properties between reinforced epoxy and selected metals [Mayo (1987)]

Material	Density (gr/cm^3)	Unidirectional strength: Tensile, MPa (ksi)	Unidirectional strength: Compressive, MPa (ksi)	Unidirectional tensile strength: GPa	Unidirectional tensile strength: (10^3 ksi)
Carbon AS-4	1.55	1482 (215)	1227 (178)	145	(21.0)
Carbon HMS	1.63	1276 (185)	1020 (148)	207	(30.0)
S-Glass™	1.99	1751 (254)	496 (72)	59	(8.6)
E-Glass	1.99	1103 (169)	490 (71)	52	(7.6)
Aramid	1.38	1310 (190)	290 (42)	83	(12.0)
Aluminum (7075-T6)	2.76	572 MPa (83 ksi)		69	(10.0)
Titanium (6A1-4V)	4.42	1103 MPa (160 ksi)		114	(16.5)
Steel (4130)	8.0	1241-1379 MPa (180-220 ksi)		207	(30.0)

Table 2.5—Comparative thickness and weight for equal strength materials [from Parklyn (1971)]

	Equal tensile strength		Equal tensile thickness		Equal bending stiffness	
Materials	Thickness	Weight	Thickness	Weight	Thickness	Weight
Mild steel	1.0	1.0	1.0	1.0	1.0	1.0
Aluminum	1.8	0.3	3.0	1.1	1.5	0.5
GFRP[1]	2.4	0.07	25	5.0	3.0	0.6
GFRP[2]	0.3	0.1	6.8	1.5	1.9	0.5

[1] Based on random fiber orientation.
[2] Based on unidirectional fiber orientation.

- Taking advantage of anisotropic nature of material and oriented fibers, but making sure that process of manufacture is compatible with selections
- Optimizing the maximum strain limitations of the laminate. The elongation of the resin is an important factor in choosing the matrix resin for a large structural part. However, the effect of stress crazing and possible stress corrosion in chemical or environmentally stressful conditions may reduce the long term performance and a more conservative design may be required. This will allow for effects of creep, cracking, aging, deleterious solutions, etc.
- Understanding creep and fatigue properties of the laminate under constant and intermittent loads
- Understanding that, in order to develop the acceptable properties, the matrix should be able to accept a higher strain than the reinforcement
- Making sure that the energy stored at failure, which is the area under the stress/ strain curve, is as large as possible, since this indicates a "tough" composite

Earlier in this chapter, the stress-strain relationship for loaded fibers was discussed. Each of the fibers considered suitable for structural engineering uses have specific elongation and stress-strain properties. Fig. 2.6 makes the range of these properties quite graphic.

2.9—Glass fibers

Glass has been the predominant fiber for many civil engineering applications because of an economical balance of cost and specific strength properties. Glass fibers are commercially available in E-Glass formulation (for electrical grade), the most widely used general-purpose form of composite reinforcement, high strength S-2® glass and ECR glass (a modified E Glass which provides improved acid resistance). Other glass fiber compositions include AR, R and Te. Although considerably more expensive than glass, other fibers including carbon and aramid, are used for their strength or modulus properties or in special situations as hybrids with glass. Properties of common high-performance reinforcing fibers are shown in Table 2.6.

2.9.1 *Chemical composition of glass fiber*—Glass fibers are made with different compositions as noted in Table 2.7, utilizing glass chemistry to achieve the chemical and physical properties required.

E-Glass—A family of calcium-alumina-silicate glasses which has the following certified chemical compositions and which is used for general-purpose molding and virtually all electrical applications. E-glass comprises approximately 80 to 90 percent of the glass fiber commercial production. The nomenclature "ECR-glass" is used for boron-free modified E-glass compositions. This formulation offers improved re-

Fig. 2.6—Glass fiber rovings [Owens-Corning Fiberglass Corporation (1995)]

Table 2.6—Comparison of inherent properties of fibers (impregnated strand per ASTM D 2343) [Owens-Corning Corp. (1993)]

	Specific gravity	Tensile strength		Tensile modulus	
		MPa	10^3 psi	GPa	10^6 psi
E-glass	2.58	2689	390	72.4	10.5
S-2-glass®	2.48	4280	620	86.0	13.0
ECR-Glass*	2.62	3625	525	72.5	10.5
K-49 Aramid	1.44	3620	525	131.0	19.0
AS4 Carbon	1.80	3790	550	234.0	34.0

* Mechanical properties—single filament at 72 F per ASTM D 2101

Table 2.7—Compositional ranges for commercial glass fibers (units = perccent by weight)

	E-glass range	S-glass range	C-glass range
Silicon dioxide	52-56	65	64-68
Aluminum oxide	12-16	25	3-5
Boric oxide	5-10	—	4-6
Sodium oxide and potassium oxide	0-2	—	7-10
Magnesium oxide	0-5	10	2-4
Calcium oxide	16-25	—	11-25
Barium oxide	—	—	0-1
Zinc oxide	—	—	—
Titanium oxide	0-1.5	—	—
Zirconium oxide	—	—	—
Iron oxide	0-0.8	—	0-0.8
Iron	0-1	—	—

sistance to corrosion by most acids.

S-Glass—Is a proprietary magnesium alumino-silicate formulation that achieves high strength, as well as higher temperature performance. S-Glass and S-2 Glass have the same composition, but use different surface treatments. S-Glass is the most expensive form of glass fiber reinforcement and is produced under specific quality control and sampling procedures to meet military specifications.

C-Glass—Has a soda-lime-borosilicate composition and is used for its chemical stability in corrosive environments. It is often used in composites that contact or contain acidic materials for corrosion-resistant service in the chemical processing industry.

2.9.2 *Forms of glass fiber reinforcements*—Glass fiber-reinforced composites contain fibers having lengths far greater than their cross sectional dimensions (aspect ratios > 10:1). The largest commercially produced glass fiber diameter is a “T” fiber filament having a nominal diameter of 22.86 to 24.12 microns. A number of fiber forms are available.

Rovings—This is the basic form of commercial continuous fiber. Rovings are a grouping of a number of strands, or in the case of so-called “direct pull” rovings, the entire roving is formed at one time. This results in a more uniform product and eliminates catenary associated with roving groups of strands under unequal tension. Fig. 2.6 shows a photo of continuous roving.

Woven roving—The same roving product mentioned above is also used as input to woven roving reinforcement. The product is defined by weave type, which can be at 0 and 90 deg; at 0 deg, +45 deg, -45 deg, and other orientations depending on the manufacturing process. These materials are sold in weight per square yard. Common weights are 18 oz/yd^2 [(610.3 gr/m^2) and 24 oz/yd^2 (813.7 gr/m^2)] (see Fig. 2.7).

Mats—These are two-dimensional random arrays of chopped strands. The fiber strands are deposited onto a continuous conveyor and pass through a region where thermosetting resin is dusted on them. This resin is heat set and holds the mat together. The binder resin dissolves in the polyester or vinyl ester matrix thereby allowing the mat to conform to the shape of the mold, (see Fig. 2.8).

Combined products—It is also possible to combine a woven roving with a chopped strand mat. There are several techniques for accomplishing this. One technique bonds the two reinforcements together with a thermosetting resin similar to that in the chopped strand approach. Another approach starts with the woven roving but has the chopped strand fibers deposited onto the surface of the woven roving, which is followed immediately by a stitching process to secure the chopped fibers. There are several variations on this theme.

Cloth—Cloth reinforcement is made in several weights as measured in ounces-per-square-yard. It is made from continuous strand filaments that are twisted and plied and then woven in conventional textile processes (see Fig. 2.9).

All composite reinforcing fibers, including glass, will be anisotropic with respect to their length. There are fiber placement techniques and textile-type operations that can further arrange fibers to approach a significant degree of quasi-iso-

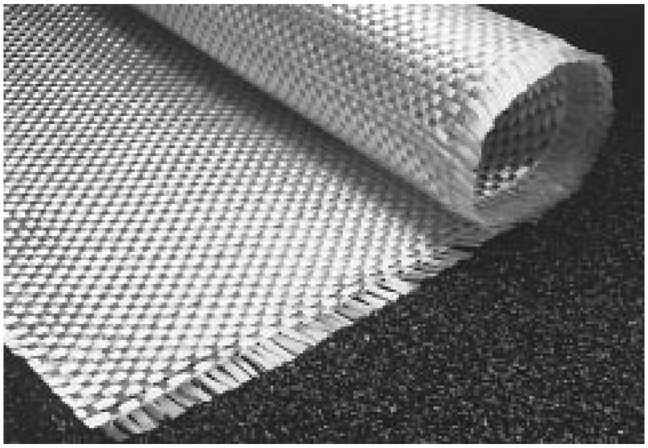

Fig. 2.7—Glass fiber woven rovings [Owens-Corning Corporation (1995)]

Fig. 2.8—Glass fiber chopped strand mat [Owens-Corning Fiberglass Corporation (1995)]

tropic composite performance. Glass fibers and virtually all other composite fibers are also available in a range of fabric-like forms including braided (see Terminology in Appendix A), needle punched, stitched, knitted, bonded, multi-axial, and multiple-ply materials.

2.9.3 *Other glass fiber considerations*—Glass fibers are very surface-active and are hydrophilic. They can be easily damaged in handling. A protective film former is applied immediately as the first step after the fiber-forming process. Sizing solutions containing the film former also contain an adhesion promoter. Adhesion promoters are usually organo-functional silanes, which function as coupling agents.

The film former also provides processability and moisture protection. The adhesion promoter acts to improve the coupling between the fiber and the polymer resin matrix. Fiber suppliers select their adhesion promoters and film formers depending on the matrix resins and manufacturing/processing parameters of the intended product.

2.9.4 *Behavior of glass fibers under load*—Glass fibers are elastic until failure and exhibit negligible creep under controlled dry conditions. Generally, it is agreed that the modulus of elasticity of mono-filament E-glass is approximately 73 GPa. The ultimate fracture strain is in the range of 2.5 to 3.5 percent. The stress-strain characteristics of strands have been thoroughly investigated. The general pattern of the stress-strain relationship for glass fibers was illustrated earlier in Fig. 2.4. The fracture of the actual strand is a cumulative process in which the weakest fiber fails first and the load is then transferred to the remaining stronger fibers which fail in succession.

Glass fibers are much stronger than a comparable glass formulation in bulk form such as window glass, or bottle glass. The strength of glass fibers is well-retained if the fibers are protected from moisture and air-borne or contact contamination.

When glass fibers are held under a constant load at stresses below the instantaneous static strength, they will fail at some point as long as the stress is maintained above a minimum value. This is called “creep rupture.” Atmospheric conditions play a role, with water vapor being most deleterious. It has been theorized that the surface of glass contains submicroscopic voids that act as stress concentrations. Moist air can contain weakly acidic carbon dioxide. The corrosive effect of such exposure can affect the stress in the void regions for glass fiber filaments until failure occurs. In addition, exposure to high pH environments may cause aging or a rupture associated with time.

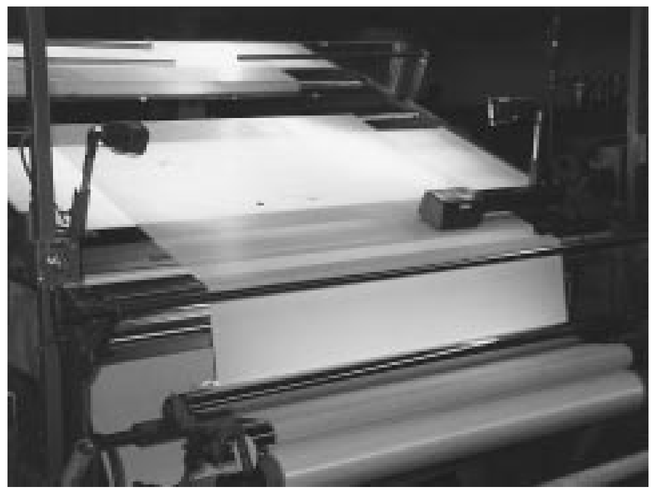

Fig. 2.9—Glass fiber cloth during weaving and inspection [Clark-Schwebel, Inc. (1995)]

These potential problems were recognized in the early years of glass fiber manufacture and have been the object of continuing development of protective treatments. Such treatments are universally applied at the fiber-forming stage of manufacture. A number of special organo-silane functional treatments have been developed for this purpose. Both multi-functional and environmental-specific chemistries have been developed for the classes of matrix materials in current use. Depending upon the resin matrix used, the result of these developments has been to limit the loss of strength to 5 to 10 percent after a 4-hr water boil test.

2.10—Carbon fibers

There are three sources for commercial carbon fibers:

Table 2.8—Typical properties of commercial composite reinforcing fibers [constructed from Mallick (1988b) and Akzo-Nobel (1994)]

Fiber	Typical diameter (microns)	Specific gravity	Tensile modulus GPa (10^6 psi)	Tensile strength GPa (10^3 psi)	Strain to failure, percent	Coefficient of thermal expansion 10^{-6}/C	Poisson's ratio
Glass E-glass	10	2.54	72.4 (10.5)	3.45 (500.0)	4.8	5.0	0.2
S-glass	10	2.49	86.9 (12.6)	4.30 (625.0)	5.0	2.9	0.22
Carbon PAN-Carbon T-300[a]	7	1.76	231 (33.5)	3.65 (530)	1.4	-0.1 to -0.5 (longitudinal), 7-12 (radial)	-0.2
AS[b]	7	1.77	220 (32)	3.1 (450)	1.2	-0.5 to -1.2 (longitudinal), 7-12 (radial)	—
t-40[a]	6	1.81	276 (40)	5.65 (820)	2.0	—	—
HSB[b]	7	1.85	344.5 (50)	2.34 (340)	0.58	—	—
Fortafil 3[TM C]	7	1.80	227 (33)	3.80 (550)	1.7	-0.1	—
Fortafil 5[TM C]	7	1.80	345 (50)	2.76 (400)	0.8	—	—
PITCH-Carbon P-555[a]	10	2.0	380 (55)	1.90 (275)	0.5	-0.9 (longitudinal)	—
P-100[a]	10	2.16	758 (110)	2.41 (350)	0.32	-1.6 (longitudinal)	—
ARAMID Kevlar[TM] 49[d]	11.9	1.45	131 (19)	3.62 (525)	2.8	-2.0 (longitudinal) +59 (radial)	0.35
Twaron[TM] 1055[e*]	12.0	1.45	127 (18)	3.6 (533)	2.5	-2.0 (longitudinal) +59 (radial)	0.35

[a] Amoco
[b] Hercules
[c] Akzo-Nobel/Fortafil fibers
[d] DuPont de Nemours and Co.
[e] Akzo-Nobel Fibers
[*] Minimum lot average values.

Table 2.9—Properties of ARAMID yarn and reinforcing fibers [constructed from DuPont (1994) and Akzo-Nobel (1994)]

Property	Kevlar 49		Twaron 1055*	
Yarn Tensile strength MPa (ksi)	2896	(420.0)	2774	(398.0)
Tenacity dN/tex (g/den)	20.4	(23)	19.0	(21.4)
Modulus GPa (ksi)	117.2	(17,000)	103.4	(15,000)
Elongation at break, percent	2.5	(2.5)	2.5	(2.5)
Density g/cm^3 (lb/in.^{3})	1.44	(0.052)	1.45	(0.052)
Reinforcing fibers Tensile strength MPa (ksi)	3620	(525.0)	3599	(522.0)
Modulus GPa (ksi)	124.1	(18,000)	127.0	(18,420)
Elongation at break, percent	2.9	(2.9)	2.5	(2.5)
Density g/cm^3 (lb/in.^{3})	1.44	(0.052)	1.45	(0.052)

* Minimum lot average values.

pitch, a by-product of petroleum distillation; PAN (polyacrylonitrile), and rayon. The properties of carbon fiber are controlled by molecular structure and degree of freedom from defects. The formation of carbon fibers requires processing temperatures above 1830 F (1000 C). At this temperature, most synthetic fibers will melt and vaporize. Acrylic, however, does not and its molecular structure is retained during high-temperature carbonization.

There are two types of carbon fiber: the high modulus Type I and the high strength Type II. The difference in properties between Types I and II is a result of the differences in fiber microstructure. These properties are derived from the arrangement of the graphene (hexagonal) layer networks present in graphite. If these layers are present in three-dimensional stacks, the material is defined as graphite. If the bonding between layers is weak and two-dimensional layers occur, the resulting material is defined as carbon. Carbon fibers have two-dimensional ordering.

High modulus carbon fibers of 200GPa (30 x 10^6 psi) require that stiff graphene layers be aligned approximately parallel to the fiber axis.

Rayon and isotropic pitch precursors are used to produce low modulus carbon fibers (50 GPa or 7 x 10^6 psi). Both PAN and liquid crystalline pitch precursors are made into higher modulus carbon fibers by carbonizing above 1400 F (800 C). Fiber modulus increases with heat treatment from 1830 F to 5430 F (1000 C to 3000 C). The results vary with the precursor selected. Fiber strength appears to maximize at a lower temperature 2730 F (1500 C) for PAN and some pitch precursor fibers, but increases for most mesophase (anisotropic) pitch precursor fibers.

The axial-preferred orientation of graphene layers in carbon fibers determines the modulus of the fiber. Both axial and radial textures and flaws affect the fiber strength. Orientation of graphene layers at the fiber surface affects wetting and strength of the interfacial bond to the matrix.

Carbon fibers are not easily wet by resins; particularly the higher modulus fibers. Surface treatments that increase the number of active chemical groups (and sometimes roughen the fiber surface) have been developed for some resin matrix materials. Carbon fibers are frequently shipped with an epoxy size treatment applied prevent fiber abrasion, improve handling, and provide an epoxy resin matrix compatible interface. Fiber and matrix interfacial bond strength approaches the strength of the resin matrix for lower modulus carbon fibers. However, higher modulus PAN-based fibers show substantially lower interfacial bond strengths. Failure in high modulus fiber occurs in its surface layer in much the same way as with aramids.

2.10.1 *Commercial forms of carbon fibers*—Carbon fibers are available as “tows” or bundles of parallel fibers. The range of individual filaments in the tow is normally from 1000 to 200,000 fibers. Carbon fiber is also available as a prepreg, as well as in the form of unidirectional tow sheets.

Typical properties of commercial carbon fibers are shown in Table 2.8.

2.11—Aramid fibers

There are several organic fibers available that can be used for structural applications. However, cost, and in some cases service temperature or durability factors, restrict their use to specific applications. The most popular of the organic fibers is aramid. The fiber is poly-para-phenyleneterephthalamide, known as PPD-T. Aramid fibers are produced commercially by DuPont (Kevlar™) and Akzo Nobel (Twaron™).

These fibers belong to the class of liquid crystal polymers. These polymers are very rigid and rod-like. The aromatic ring structure contributes high thermal stability, while the para configuration leads to stiff, rigid molecules that contribute high strength and high modulus. In solution they can aggregate to form ordered domains in parallel arrays. More conventional flexible polymers in solutions bend and entangle, forming random coils.

When PPD-T solutions are extruded through a spinneret and drawn through an air gap during manufacture, the liquid crystal domains can align in the direction of fiber flow. The fiber structure is anisotropic, and presents higher strength and modulus in the longitudinal direction than in the axial transverse direction. The fiber is also fibrillar (it is thought that tensile failure initiates at fibril ends and propagates via shear failure between the fibrils).

2.11.1 *Material properties of aramid*—Representative properties of para-aramid (p-aramid) fibers are given below. Kevlar 49 and Twaron 1055 are the major forms used today because of higher modulus. Kevlar 29 and Twaron 2000 are used for ballistic armor and applications requiring increased toughness. Ultra-high modulus Kevlar 149 is also available. Aramid fibers are available in tows, yarns, rovings, and various woven cloth products. These can be further processed to intermediate stages, such as prepregs. Detailed properties of aramid fibers are shown in Table 2.9.

- Tensile modulus is a function of molecular orientation
- Tensile strength: Para-aramid fiber is 50 percent stronger than E glass. High modulus p-aramid yarns show a linear decrease of both tensile strength and modulus when tested at elevated temperature. More than 80 percent of strength is retained after temperature conditioning
- At room temperature the effect of moisture on tensile properties is < 5 percent
- Creep and fatigue: Para-aramid is resistant to fatigue and creep rupture
- Creep rate is low and similar to that of fiberglass. It is less susceptible to creep rupture
- Compressive properties: Para-aramid exhibits nonlinear, ductile behavior under compression. At a compression strain of 0.3 to 0.5 percent, a yield is observed. This corresponds to the formation of structural defects known as kink bands, which are related to compressive buckling of p-aramid molecules. This compression behavior limits the use of p-aramid fibers in applications that are subject to high strain compressive or flexural loads
- Toughness: Para-aramid fiber’s toughness is related directly to conventional tensile toughness, or area under the stress-strain curve. The p-aramid fibrillar structure and compressive behavior contribute to composites that are less notch sensitive
- Thermal properties: The p-aramid structure results in a high degree of thermal stability. Fibers will decompose in air at 800 F (425 C). They have utility over the temperature range of -200 C to 200 C, but are not used long-term at temperatures above 300 F (150 C) because of oxidation. The fibers have a slightly negative longitudinal coefficient of thermal expansion of -2 x 10 -6/K
- Electrical properties: Para-aramid is an electrical insulator. Its dielectric constant is 4.0 measured at 106 Hz
- Environmental behavior: Para-aramid fiber can be degraded by strong acids and bases. It is resistant to most other solvents and chemicals. UV degradation can also occur. In polymeric composites, strength loss has not been observed

One caution concerns the compressive behavior noted above, which results in local crumpling and fibrillation of in-

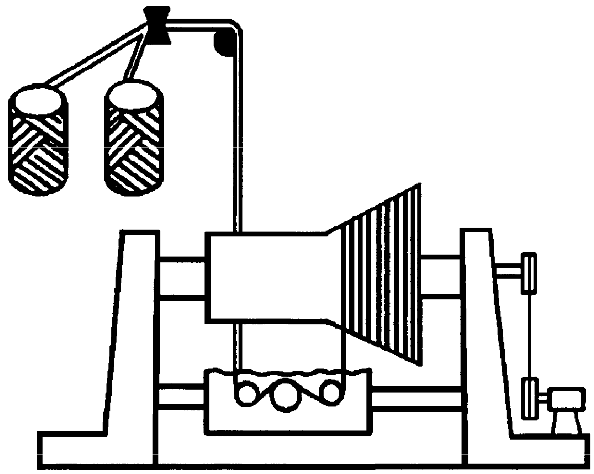

Fig. 2.10—Filament winding process [Mettes (1969e)]

dividual fibers, thus leading to low strength under conditions of compression and bending. For this reason aramids are unsuitable, unless hybridized with glass or carbon fiber, for use in FRP shell structures which have to carry high compressive or bending loads. Such hybridized fiber structures lead to a high vibration damping factor which may offer advantages in dynamically loaded FRP structures.

2.12—Other organic fibers

Ultra-high-molecular-weigh t-polyethyl ene fibers—One fiber of this type manufactured and marketed by Allied Signal Corp. in the United States is called Spectra™. It was originally developed in the Netherlands by DSM (Dutch State Mines).

Table 2.10 shows the properties of Spectra ultra-high-molecular-weight polyethylene fibers. The major applications for Spectra have been in rope, special canvas and woven goods, and ballistic armor. Its lightness combined with strength and low tensile elongation make it attractive for these uses. Drawbacks include fiber breakdown at temperatures above 266 F (130 C). None of the current resin matrix materials bond well to this fiber. Plasma treatment has been used to etch the surface of the fibers for a mechanical bond to the resin matrix, but this is expensive, and is not readily available in commercial production.

2.13—Hybrid reinforcements

It should be apparent that properties of the fibers differ significantly. The so-called high-performance fibers also carry high performance price tags.

These materials can be combined in lamina, and in uniaxial arrangements as hybrids to give appropriate properties at an acceptable cost. The infrastructure applications are natural opportunities for evaluation and utilization of such combinations. Table 2.11 illustrates the results that can be obtained.

Both polymer matrix resin and reinforcement exercise an interactive effect on the fabrication used to join composite materials, forming the finished part.

2.14—Processes for structural moldings

There are several methods of achieving reliable fiber placement. These methods can be considered process-specific (i.e., the nature of the forming process and/or its contingent tooling largely controls the fabricated result). In this category are the common commercial fabricating processes.

Filament winding—This process takes continuous fibers

Table 2.10—Properties of spectra™ fibers [from Pigliacampi (1987)]

	Spectra 900	Spectra 1000
Density gr/cm^3 (lb/in.^{3})	0.97 (0.035)	0.97 (0.035)
Filament diameter m (in.)	38 (1500)	27 (1060)
Tensile modulus GPa (10^6 psi)	117 (17)	172 (25)
Tensile strength GPa (10^6 psi)	2.6 (0.380)	2.9-3.3 (0.430-0.480)
Tensile elongation, percent	3.5	2.7
Available yarn count (number of filaments)	60-120	60-120

Table 2.11—Properties of carbon-glass-polyester hybrid composites* [from Schwarz (1992e)]

Carbon/glass ratio	Tensile strength, MPa (ksi)	Modulus of elasticity (tension), GPa (10^6 psi)	Flexural strength, MPa (ksi)	Flexural modulus, GPa (10^6 psi)	Interlaminar shear strength, MPa (ksi)	Density, gr/cm^3 (lbs/in.^{3})
0:100	604.7 (87.7)	40.1 (5.81)	944.6 (137)	35.4 (5.14)	65.5 (9.5)	1.91 (0.069)
25:75	641.2 (93.0)	63.9 (9.27)	1061.8 (154)	63.4 (9.2)	74.5 (10.8)	1.85 (0.067)
50:50	689.5 (100)	89.6 (13.0)	1220.4 (177)	78.6 (11.4)	75.8 (11.0)	1.80 (0.065)
75:25	806.7 (117)	123.4 (17.9)	1261.7 (183)	1261.7 (16.3)	82.7 (12.0)	1.66 (0.060)

* Fiber contents are by volume. Resin is 48 percent Thermoset Polyester, plus 52 percent continuous unidirectional oriented fiber by volume, equivalent to 30 percent resin and 70 percent fiber by weight. Properties apply to longitudinal fiber direction only.
1 ksi = 6.895 MPa; 1 lb/in.^{3} = 0.0361 g/cm^3.

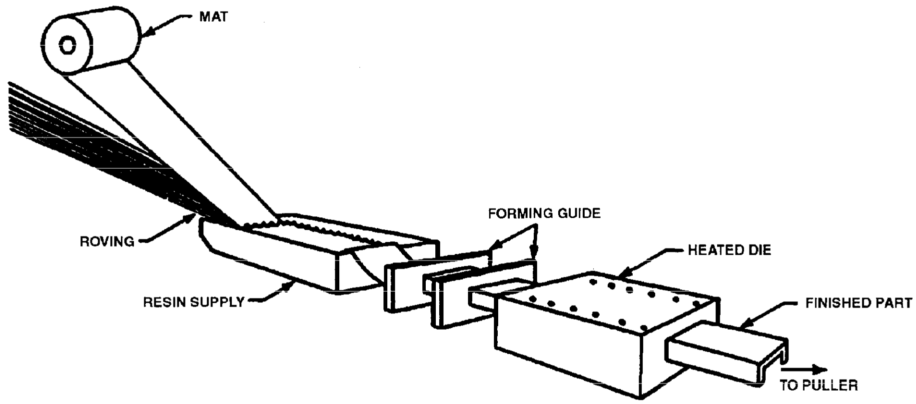

Fig. 2.11—Pultrusion process [Creative Pultrusions, Inc. (1994)]

in the form of parallel strands (rovings), impregnates them with matrix resin and winds them on a rotating cylinder. The resin-impregnated rovings are made to traverse back and forth along the length of the cylinder. A controlled thickness, wind angle, and fiber volume fraction laminate is thereby created. The material is cured on the cylinder and then removed (see Fig. 2.10).

Pipe, torsion tubes, rocket cases, pressure bottles, storage tanks, airplane fuselages, and the like are made by this process. The moving relationship between the rotating surface and the roving/matrix is usually controlled by computer. There can be additional add-on fiber/matrix placement systems to add chopped short-length fibers and/or particulate materials to increase thickness at low cost. Polyester, vinyl ester, and epoxy matrix materials are used.

Pultrusion—This process makes a constant cross-section part of unlimited length which is constrained only by building and shipping limitations. The pultrusion process uses continuous fibers from a series of creel positions (see Terminology in Appendix A). All the fiber rovings necessary for the cross-section of the part are drawn to a wet-out bath that contains the resin matrix, catalyst (or hardener), and other additives. The rovings are impregnated in the bath. Excess liquid resin is removed and returned to the bath, while the wet-out roving enters the pultrusion die. These dies are generally 36 in. to 48 in. (0.9-1.3 m) long and are heated electrically or by hot oil. In some cases, a radio-frequency (RF) preheating cabinet is employed to increase the ease of curing thick sections. Throughput rate is generally about 0.9 m (36 linear in.) per min. Complex and thick sections may take more time to affect complete cure while very thin sections may take less time. Polyester resin and vinyl esters are the major matrix materials used in the pultrusion process (Fig. 2.11).

Examples of products produced by pultrusion include oil well sucker rods; tendons for prestressing and post-tensioning concrete; concrete formties; structural shapes for mechanical fabrication used in offshore drilling rigs, and chemical processing plants; grating; third rail covers; automobile drive shafts; ground anchors and tie backs; sheet piling, and window frame sections.

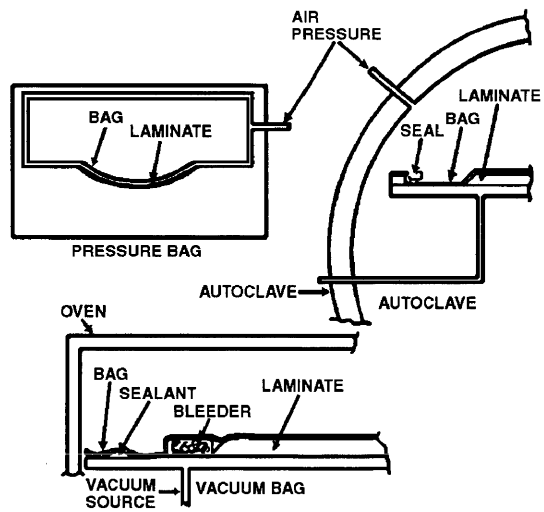

Fig. 2.12—Vacuum compaction processing [Schwarz (1992f)]

Vacuum compaction processes—This is a family of processes in which the weight of the atmosphere can work against a materials system that has been sufficiently evacuated of entrapped air to allow compression and compaction of the uncured laminate to take place. In some forms of the process, a pre-impregnated arrangement of fibers is placed on a mold in one or more lamina thicknesses. A covering sheet of stretchable film is placed over the lamina array and secured to the mold surface. A vacuum is drawn from within the covered area by a hose leading to a vacuum pump. As the air is evacuated, the stretchable sheet is pressed against the fiber/prepreg array to compact the lamina. The entrapped air is thereby removed from between the laminae plies. If the resin matrix is heated by one of a number of methods, (infrared lamps, heated mold, steam autoclave, etc.), the resin viscosity drops and additional resin densification can take place before the increase in resin viscosity associated with curing (Fig. 2.12).

Other processes use vacuum to compact a dry fiber array on the mold. This allows the resin to flow into the evacuated mechanical spaces between and among the fibers. This is easier said than accomplished. There are several modifications of this methodology that can allow the resin to flow through the compacted fiber arrays. Most of these methods utilize auxiliary resin distribution schemes and positive spacing methods to keep the stretch film from clamping off the flow of resin prematurely. Resin cure is described above. There are currently demonstration processes of this type which appear to be suitable for making very large moldings in this manner. Note that this process does not require a molding press, only a single-sided tool.

Matched mold processes—This system includes a range of process materials. However, several characteristics are shared:

- The molds define the shape and thickness of the part, so they must have a means of being reproducibly repositioned for each part. In most cases this implies a press of some sort.
- The practical limit on the size of the press, plane area and openings. Pressing forces depending on the material system in the range of 30 to 900 psi (0.21- 6.21 MPa) will be required. The lower number is associated with Resin Transfer Molding (RTM), and the higher number is common for Sheet Molding Compound. Also, these systems generally use short fibers, in three dimensional arrays, and properties will be quasi-isotropic, and much lower than the anisotropic arrays of continuous long fibers.

2.15—Summary

In this chapter, the major materials used in composite systems were identified and discussed. The interactions between the form and physical nature of these materials and the molding processes, a relationship somewhat unique to structural composites, were discussed. This interaction should be kept in mind to continually remind the structural practitioner of the potential efficiency and cost trade-offs available with composites. When one chooses composite materials without sufficient regard for the inter-relationship of materials, form of materials, and processing, the result may be overly expensive, structurally ineffective, or difficult to fabricate.

CHAPTER 3—MECHANICAL PROPERTIES AND TEST METHODS

3.1—Physical and mechanical properties

In discussions related to the properties of FRP bars or tendons, the following points must be kept in mind. First, an FRP bar is anisotropic, with the longitudinal axis being the strong axis. Second, unlike steel, mechanical properties of FRP composites vary significantly from one product to another. Factors such as volume and type of fiber and resin, fiber orientation, dimensional effects, and quality control during manufacture, play a major role in establishing product characteristics. Furthermore, the mechanical properties of FRP composites, like all structural materials, are affected by such factors as loading history and duration, temperature, and moisture.

While standard tests have been established to determine the properties of traditional construction materials, such as steel and concrete, the same cannot be said for FRP materials. This is particularly true for civil engineering applications, where the use of FRP composites is in its stage of infancy. It is therefore required that exact loading conditions be determined in advance and that material characteristics corresponding to those conditions be obtained in consultation with the manufacturer.

3.1.1 *Specific gravity*—FRP bars and tendons have a specific gravity ranging from 1.5 to 2.0 as they are nearly four times lighter than steel. The reduced weight leads to lower transportation and storage costs and decreased handling and installation time on the job site as compared to steel reinforcing bars. This is an advantage that should be included in any cost analysis for product selection.

3.1.2 *Thermal expansion*—Reinforced concrete itself is a composite material, where the reinforcement acts as the strengthening medium and the concrete as the matrix. It is therefore imperative that behavior under thermal stresses for the two materials be similar so that the differential deformations of concrete and the reinforcement are minimized. Depending on mix proportions, the linear coefficient of thermal

Table 3.1—Comparison of mechanical properties (longitudinal direction)

	Steel reinforcing bar	Steel tendon	GFRP bar	GFRP tendon	CFRP tendon	AFRP tendon
Tensile strength, MPa (ksi)	483-690 70-100	1379-1862 200-270	517-1207 75-175	1379-1724 200-250	165-2410 240-350	1200-2068 170-300
Yield strength, MPa (ksi)	276-414 40-60	1034-1396 150-203	Not applicable			
Tensile elstic modulus, GPa (ksi)	200 29,000	186-200 27,000-29,000	41-55 6000-8000	48-62 7000-9000	152-165 22,000-24,000	50-74 70,000-11,000
Ultimate elongation, mm/mm	> 0.10	>0/04	0.035-0.05	0.03-0.045	0.01-0.015	0.02-0.026
Compressive strength, MPa (ksi)	276-414 40-60	N/A	310-482 45-70	N/A	N/A	N/A
Coefficient of thermal expansion (10^{-6}/C) (10^{-6}/F)	11.7 6.5	11.7 6.5	9.9 5.5	9.9 5.5	0.0 0.0	-1.0 -0.5
Specific gravity	7.9	7.9	1.5-2.0	2.4	1.5-1.6	1.25

Note: All properties refer to unidirectional reinforced coupons. Properties vary with the fiber volume (45-70 percent), coupon diameter, and grip system.
N/A = Not available.

expansion for concrete varies from 6 to 11 x 10^{-6} per C (4 to 6 x 10^{-6} per F) (Mindess et al. 1981). Listed in Table 3.1 are the coefficients of thermal expansion for typical FRP products.

3.1.3 *Tensile strength*—FRP bars and tendons reach their ultimate tensile strength without exhibiting any material yielding. A comparison of the properties of FRP and steel reinforcing bars and tendons is shown in Table 3.1. The mechanical properties of FRP reported here are measured in the longitudinal (i.e. strong) direction. Values reported for FRP materials cover some of the more commonly available products.

Unlike steel, the tensile strength of FRP bars is a function of bar diameter. Due to shear lag, fibers located near the center of the bar cross section are not subjected to as much stress as those fibers that are near the outer surface of the bar (Faza 1991). This phenomenon results in reduced strength and efficiency in larger diameter bars. For example, for GFRP reinforcement produced by one U.S. manufacturer, the tensile strength ranges from nearly 480 MPa (70 ksi) for 28.7 mm (No. 9) bars to 890 MPa (130 ksi) for 9.5 mm (No. 3) bars (Ehsani et al. 1993).

Some FRP tendons were made by stranding seven GFRP (S-2 Glass) or CFRP pultruded bars of diameter ranging from 3 to 4 mm (0.125 to 0.157 in.). The ultimate strength of these tendons was comparable to that of a steel prestressing strand. For GFRP tendons, ultimate strength varied from 1380 to 1724 MPa (200 to 250 ksi); while for CFRP tendons, it varied from 1862 to 2070 MPa (270 to 300 ksi) (Iyer and Anigol 1991).

3.1.4 *Tensile elastic modulus*—As noted in Table 3.1, the longitudinal modulus of elasticity of GFRP bars is approximately 25 percent that of steel. The modulus for CFRP tendons, which usually employ stiffer fibers, is higher than that of GFRP reinforcing bars.

3.1.5 *Compressive strength*—FRP bars are weaker in compression than in tension. This is the result of difficulties in accurately testing unidirectional composites in compression, and is related to gripping and aligning procedures, and also to stability effects of fibers. However, the compressive strength of FRP composites is not a primary concern for most applications. The compressive strength also depends on whether the reinforcing bar is smooth or ribbed. Compressive strength in the range of 317 to 470 MPa (46 to 68 ksi) has been reported for GFRP reinforcing bars having a tensile strength in the range of 552 to 896 MPa (80 to 130 ksi) (Wu 1990). Higher compressive strengths are expected for bars with higher tensile strength.

3.1.6 *Compressive elastic modulus*—Unlike tensile stiffness, compressive stiffness varies with FRP reinforcing bar size, type, quality control in manufacturing, and length-to-diameter ratio of the specimens. The compressive stiffness of FRP reinforcing bars is smaller than the tensile modulus of elasticity. Based on tests of samples containing 55 to 60 percent volume fraction of continuous E-glass fibers in a matrix of vinyl ester or isophthalic resin, a modulus of 34 to 48 GPa (5000 to 7000 ksi) has been reported (Wu 1990). Another manufacturer reports the compressive modulus at 34 GPa (5000 ksi) which is approximately 77 percent of the tensile modulus for the same product (Bedard 1992).

3.1.7 *Shear strength*—In general, shear strength of composites is very low. FRP bars, for example, can be cut very easily in the direction perpendicular to the longitudinal axis with ordinary saws. This shortcoming can be overcome in most cases by orienting the FRP bars such that they will resist the applied loads through axial tension. Shear tests using a full-scale Isoipescu test procedure have been developed (Porter et al. 1993). This shear test procedure has been applied successfully to obtain shear properties for FRP dowel bars on over 200 specimens.

3.1.8 *Creep and creep rupture*—Fibers such as carbon and glass have excellent resistance to creep, while the same is not true for most resins. Therefore, the orientation and volume of fibers have a significant influence on the creep performance of reinforcing bars and tendons. One study reports that for a high-quality GFRP reinforcing bar, the additional strain caused by creep was estimated to be only 3 percent of the initial elastic strain (Iyer and Anigol 1991).

Under loading and adverse environmental conditions, FRP reinforcing bars and tendons subjected to the action of a constant load may suddenly fail after a time, referred to as the endurance time. This phenomenon, known as creep rupture, exists for all structural materials including steel. For steel prestressing strands, however, this is not of concern. Steel can endure the typical tensile loads, which are about 75 percent of the ultimate strength, indefinitely without any loss of strength or fracture. As the ratio of the sustained tensile stress to the short-term strength of the FRP increases, endurance time decreases. Creep tests were conducted in Germany on GFRP composites with various cross sections. These studies indicate that creep rupture does no occur if sustained stress is limited to 60 percent of the short-term strength (Budelmann and Rostasy 1993).

The above limit on stress may be of little concern for most reinforced concrete structures since the sustained stress in the reinforcement is usually below 60 percent. It does, however, require special attention in applications of FRP composites as prestressing tendons. It must be noted that other factors, such as moisture, also impair creep performance and may result in shorter endurance time.

Short-term (48 hr) and long-term (1 year) sustained load corresponding to 50 percent of the ultimate strength was applied to GFRP and CFRP tendons at room temperature. The specimens showed very little creep. Tensile modulus and ultimate strength after the test did not change significantly (Anigol 1991, and Khubchandani 1991).

3.1.9 *Fatigue*—FRP bars exhibit good fatigue resistance. Most research in this regard has been on high-modulus fibers (e.g., aramid and carbon), which were subjected to large cycles of tension-tension loading in aerospace applications. In tests where the loading was repeated for 10 million cycles, it was concluded that carbon-epoxy composites have better fatigue strength than steel, while the fatigue strength of glass composites is lower than steel at a low stress ratio (Schwarz 1992). Other research (Porter et al. 1993) showed good fatigue resistance of GFRP dowel bars in shear subjected to 10 million cycles. In another investigation, GFRP bars constructed for prestressing applications were subjected to repeated cyclic loading with a maximum stress of 496 MPa (72 ksi) and a stress range of 345 MPa (50 ksi). The bars could stand more than 4 million cycles of loading before failure initiated at the anchorage zone (Franke 1981).

CFRP tendons exhibited good fatigue resistance as shown in the tension-tension fatigue test for 2 million cycles. The mean stress was 60 percent of the ultimate strength with minimum and maximum stress levels of 55 and 64 percent of the ultimate strength. The modulus of elasticity of the tendons did not change after the fatigue test (Gorty 1994).

3.2—Factors affecting mechanical properties

Mechanical properties of composites are dependent on many factors including load duration and history, temperature, and moisture. These factors are interdependent and, consequently, it is difficult to determine the effect of each one in isolation while the others are held constant.

3.2.1 *Moisture*—Excessive absorption of water in composites could result in significant loss of strength and stiffness. Water absorption produces changes in resin properties and could cause swelling and warping in composites. It is therefore imperative that mechanical properties required of the composites be determined under the same environmental conditions where the material is to be used. There are, however, resins which are formulated to be moisture-resistant and may be used when a structure is expected to be wet at all times. In cold regions, the effect of freeze-thaw cycles must also be considered.

3.2.2 *Fire and temperature*—Many composites have good to excellent properties at elevated temperatures. Most composites do not burn easily. The effect of high temperature is more severe on resin than on fiber. Resins contain large amounts of carbon and hydrogen, which are flammable, and research is continuing on the development of more fire-resistant resins (Schwarz 1992). Tests conducted in Germany have shown that E-glass FRP bars could sustain 85 percent of their room-temperature strength, after half an hour of exposure to 300 C (570 F) temperature while stressed to 50 percent of their tensile strength (Franke 1981). While this performance is better than that of prestressing steel, the strength loss increases at higher temperatures and approaches that of steel.

The problem of fire for concrete members reinforced with FRP composites is different from that of composite materials subjected to direct fire. In this case, the concrete serves as a barrier to protect the FRP from direct contact with flames. However, as the temperature in the interior of the member increases, the mechanical properties of the FRP may change significantly. It is therefore recommended that the user obtain information on the performance of a particular FRP reinforcement and resin system at elevated temperatures when potential for fire is high.

3.2.3 *Ultraviolet rays*—Composites can be damaged by the ultraviolet rays present in sunlight. These rays cause chemical reactions in a polymer matrix, which can lead to degradation of properties. Although the problem can be solved with the introduction of appropriate additives to the resin, this type of damage is not of concern when FRP elements are used as internal reinforcement for concrete structures, and therefore not subjected to direct sunlight.

3.2.4 *Corrosion*—Steel reinforcement corrodes and the increase in material volume produces cracks and spalling in concrete to accelerate further deterioration. A major advantage of composite materials is that they do not corrode. It must be noted, however, that composites can be damaged as a result of exposure to certain aggressive environments. While GFRP bars have high resistance to acids, they can deteriorate in an alkaline environment. In a recently completed study for prestressed concrete applications, a particular type of glass-epoxy FRP strand embedded in concrete was subjected to salt water tidal simulation, which resulted in water gain and loss of strength (Sen et al. 1993). Although these results cannot be generalized, they highlight the importance of the selection of the correct fiber-resin system for a particular application. FRP tendons made of carbon fibers are resistant to most chemicals (Rostasy et al. 1992).

3.2.5 *Accelerated aging*—Short-term need for long-term weathering data has necessitated the creation of such analytical techniques as accelerated aging to predict the durability of composite structures subjected to harsh environments over time. Research done at Pilkington Bros. (Proctor et al. 1982) shows that long-term aging predictions, made over a very short period of time and at higher temperatures correlate well with real weather aging. Based on these findings, researchers (Porter et al. 1992) developed two equations for accelerated aging of FRP composites. The first equation gave an acceleration factor based on the mean annual temperature of a particular climate. The second equation showed a relationship between bath temperature and number of acquired accelerated aging days per day in the bath (Lorenz 1993, Porter et al. 1992). By using these two equations, dowel bars composed of E-glass fibers encapsulated in a vinyl ester resin were aged at an elevated temperature of 60 C (140 F) for nine weeks. Specimens were aged in water, lime, and salt bath solutions. An accelerated aging period of 63.3 days at an elevated temperature of 60 C (140 F) in the solutions was utilized without appreciable degradation for a lime bath. This accelerated aging was equivalent to approximately 50 years.

3.3—Gripping mechanisms

The design and development of a suitable gripping mechanism for FRP bars in tension tests and in pre and post-tensioned concrete applications have presented major difficulties to researchers and practitioners. Due to the low strength of FRP reinforcing bars and tendons in the transverse direction, the forces introduced by the grips can result in localized failure of the FRP within the grip zone. Clearly, the use of longer grips to reduce the stresses in the grip zone is impractical in most cases.

One type of re-usable grips (GangaRao and Faza 1992) consists of two steel plates 178 by 76 by 19 mm (7.0 by 3.0 by 0.75 in.) with a semi-circular groove is cut out of each plate. The groove diameter is 3 mm (0.12 in.) larger than the diameter of the bar to be tested. Fine wet sand on top of an epoxy-sand coating is used to fill the groove. Two plates are carefully brought together at each end of the bar to be tested. The grips are then placed inside the jaws of a universal testing machine. Although these grips may allow a slight slippage of the bar, this limitation is not a major concern when the bar is being tested to failure. It has been reported (Chen et al. 1992) that a set of such grips was successfully used for tensioning FRP reinforcing bars. In this application, six high-strength bolts were used to clamp the two plates together.

A method for stressing FRP cables using steel chucks 15 mm (0.6 in.) in diameter was developed (Iyer and Anigol 1991). Two steel chucks are used at each end to develop the full strength of the cable.

Researchers (Porter et al. 1992) have developed a gripping method where FRP bars were bonded with epoxy into a copper pipe. Tensile testing studies using these grips have produced a procedure for gripping FRP specimens without crushing the bar. More than 200 tensile specimens were successfully tested using a long length between grips. Consistent tensile values were produced that reasonably match the theoretical specimen tension strengths. Research is underway to investigate the use of regular steel grips threaded internally and filled with the same epoxy.

3.4—Theoretical modeling of GFRP bars

Theoretical modeling of the mechanical properties of an FRP reinforcing bar, subjected to a variety of static loads, has been attempted through micromechanical modeling, macromechanical modeling, and three-dimensional finite element modeling (Wu 1990).

The objective of micromechanical modeling was to predict material properties as a function of the properties of the constituent materials. A unidirectional FRP bar was analyzed as a transversely isotropic material. In this model, individual fibers were assumed to be isotropic.

In the macromechanical model, FRP reinforcing bars were treated as homogeneous but anisotropic bars of circular cross-section. The theory of elasticity solution for circularly laminated bars was used (Wu 1990). The reinforcing bar was assumed to be axisymmetric, with a number of thin layers of transversely isotropic material comprising the cylinder wall. A monoclinic material description was used since each layer could have arbitrary fiber orientation.

A three-dimensional finite element analysis using isoparametric elements and constitutive equations of monoclinic materials was also employed (Wu 1990). Simulation of actual tensile test conditions of FRP bars were performed assuming a linear distribution of shear transfer between the gripping mechanism and the bar. First ply failure along with the maximum stress failure criteria were employed in this model. The ultimate tensile strength predicted by the analysis was 25 percent higher than the experimental value. To overcome the limitations of both finite element model and elasticity solution, a mathematical model using the strength of materials approach, including the shear lag between the fibers, was developed. The maximum failure strain of the fibers was considered as the only governing criterion for failure. The model used a circular cross section to compute tensile or bending strength. The major assumption in developing this model was that strain distribution across the section is parabolic and axisymmetric. The parabolic strain distribution was assumed to result from the radial stresses induced by the gripping mechanism. The model predicted tensile forces in the core fibers lower than those forces at the surface of the bar.

3.5—Test methods

3.5.1 *Introduction*—Test methods are important to evaluate the properties of resin, fiber, FRP composite, and structural components. This section deals with test methods related to FRP composites for civil engineering applications. The resin groups included are: polyester, vinyl ester, epoxy, and phenolic. The fibers included are: E-glass, S-2 glass, aramid, and carbon. FRP composites made of a combination of the above resins and fibers with different proportions are used for reinforcement of concrete members as bars, cables, and plates. Only continuous fiber reinforcements are included in this report. ASTM standards divide the test methods relative to FRP composites into two sections; one dealing with glass FRP composites, and one dealing with high-modulus FRP composites using fiber types such as carbon.

3.5.2 *Test methods*

3.5.2.1 *Glass composite bars (GFRP)*

Tension test—Pultruded bars made with continuous glass fiber and ranging in diameter from 3.2 to 25.4 mm (0.12 to 1.00 in.) can be tested for tensile strength using ASTM D 3916. Aluminum grips with sandblasted circular surfaces are used. This test determines the ultimate strength, elastic modulus, percentage elongation, ultimate strain, and Poisson's ratio.

Flexural strength test—Flexural strength tests on pultruded GFRP bars can be conducted using ASTM D 4476. This test provides modulus of rupture and modulus of elasticity in bending.

Horizontal shear strength test—Horizontal shear strength of pultruded GFRP bars can be determined using ASTM D 4475 which is a short beam test method.

Creep and relaxation test—Aluminum grips can be used to hold a specimen between special steel jigs as shown in ASTM D 3916. This jig provides a self-straining frame condition to apply a constant load. The specimen extension can be measured by a dial gage or strain gage to determine the increase in strain under sustained load with time.

Nondestructive testing—Acoustic emission (AE) technique was used to monitor the behavior of GFRP bars subjected to direct tension (Chen et al. 1992a, 1993). AE signals emitted by breakage of matrix and fibers were monitored using two AE sensors (Chen et al. 1993).

3.5.2.2 *Carbon composite bars (CFRP)*

Tension test—Test methods and fixtures used for glass FRP bars could be used for carbon FRP composites, but may not be entirely suitable as higher stress levels are needed to attain tensile failure. Testing methods with flat jaws may be used for determining the tensile strength, elastic modulus, and ultimate strain.

Flexural and horizontal shear—Test methods for high modulus FRP composites are not listed in ASTM, but the methods recommended for glass FRP bars can be used for evaluating carbon pultruded bars.

3.5.2.3 *Composite plates*—Glass and high-modulus (carbon) laminated plates can be tested for tension, compression, flexure, tension-tension fatigue, creep, and relaxation using the ASTM methods as listed: D 3039 (Tension), D 3410 (Compression), D 790 (Flexure), D 3479 (Fatigue), D 2990 (Creep), and D 2991 (Relaxation).

3.5.2.4 *Composite cables*—Composite cables are generally made of several small-diameter pultruded FRP bars. A major problem for determining the tensile properties of a cable is holding the cable without causing failure at the anchorage. Several anchorages are under development and most of them use a polymer resin within a metal tube.

An anchorage system previously described (Iyer and Anigol 1991) was successfully used with a total standard length of cable of 1220 mm (4 ft) and with 250 mm (10 in.) anchorage length on either end. Steel plates having holes to hold the steel chucks were mounted on a universal testing machine. Glass, aramid, and carbon FRP cables could be tested using this anchorage system (Iyer 1991). A short-term sustained-load test with this anchorage system was conducted for a limited time (48 hr) using a servo-controlled testing machine. A long-term sustained-load test was conducted using three cables and a modified creep frame used for concrete testing. Anchorage slip was monitored with dial gages and LVDTs to determine the net creep of the cables (Gorty 1994). Tension-tension fatigue tests were also conducted with stress varying sinusoidally between 45 and 60 percent of the ultimate strength, at a frequency of 8 Hz, and for a total of 1 and 2 million cycles. The elastic modulus before and after cyclic loading could be determined to evaluate performance of the cable under cyclic loading (Gorty 1994).

Tube anchorages with threaded ends and nuts were found to be successful. One advantage of this method is that it can be adapted to any bar or cable type and diameter (Iyer et al. 1994).

3.5.3 *Conclusion*—Test methods are needed to determine properties of FRP products. Test results are used for quality control during production and for field use. Hence, test methods must be reproducible and reliable. Variation of test procedure and specimen geometry should be addressed to develop meaningful comparisons. Statistical methods of approval are needed to establish the properties of bars, plates, and cables. Other tests that take into consideration environmental changes such as temperature and moisture should be included in the evaluation of FRP products.

CHAPTER 4—DESIGN GUIDELINES

This chapter provides guidance for the design of FRP reinforced members. Specific design equations are avoided due to the lack of comprehensive test data. Where appropriate, references are made to research recommendations given in Chapter 5. This separation is intentional since research for one specific FRP material, that is, glass, may not be applicable to alternative materials, for example, carbon and aramid.

4.1—Fundamental design philosophy

The development of proposed behavioral equations in Chapter 5 and the constructed examples cited in Chapter 8 suggest that the design of concrete structures using FRP reinforcement is well advanced. In fact, with the exception of the comprehensive testing on GFRP reinforcing bars, (GangaRao and Faza, 1991) and the Parafil studies in England (Kingston 1988 and Burgoyne 1988), designs have been completed using basic engineering principles rather than formalized design equations.

For flexural analysis, the fundamental principles include equilibrium on the cross section, compatibility of strains, typically the use of plane sections remaining plane, and constitutive behavior. For the concrete, the constitutive behavior model uses the Whitney rectangular stress block to approximate the concrete stress distribution at strength conditions. For the FRP reinforcement, the linear stress versus strain relationship to failure must be used. These models work very well for members where the FRP reinforcement is in tension. More work is needed for the use of FRP in compression zones due to possible buckling of the individual fibers within the reinforcing bar.

The philosophy of strengthening reinforced concrete members with external FRP plates basically uses the same assumptions. With bonded plates, much more attention must be placed on the interlaminar shear between the plate and the concrete and at the end termination of the plates.

There is so little research available on the use of FRP shear reinforcement that design recommendations have not been suggested. The literature would suggest that the lower modulus of elasticity of the FRP shear reinforcement allows the shear cracks to open wider than comparable steel reinforcement. A reduction in shear capacity would be expected since “concrete contribution” is reduced.

The use of FRP materials as a reinforcement for concrete beams requires the development of design procedures that ensure adequate safety from catastrophic failure. With steel reinforcing, a confident level of safety is provided by specifying that a section's flexural strength be at least 25 percent less than its balanced flexural strength ($\rho_{actual} < 0.75\rho_{bal}$). This ensures the steel will yield before the concrete crushes, therein, guaranteeing a ductile failure. The result is the ability of the failed beam to absorb large amounts of energy through plastic straining in the reinforcing steel. FRP materials respond linearly and elastically to failure at which point brittle rupture occurs. As a result, failure, whether the result of shear, flexural compression or flexural tension, is unavoidably sudden and brittle. Building codes and design specifications will eventually recognize the advantages and disadvantages of FRP materials when defining analytical procedures on which engineers will rely for design. This may require lower flexural capacity reduction factors to be more compatible with the specific performance limitations of FRP materials.

4.2—Ductility

A formal definition of ductility is the ratio of the total deformation or strain at failure to the deformation or strain at yielding. FRP reinforcements have a linear stress versus strain relationship to failure. Therefore, by the above definition, the behavior of FRP reinforced members cannot be considered ductile.

The 1995 edition of ACI 318 contains an appendix with alternative provisions for the establishment of capacity reduction factors. Part of ACI 318-95 defines the maximum reinforcement ratio for tension controlled sections by the ratio that produces a net tensile strain of not less than 0.005 at nominal strength. The net tensile strain is measured at the level of the extreme tension reinforcement at nominal strength due to factored loads, exclusive of effective prestress strain (Mast, 1992). This provision was enacted to allow for members with various reinforcing materials including high strength steel reinforcement and steel prestressing strands, which have markedly different yield strains than ordinary reinforcement. Using the above definition, ductility of FRP reinforced member may be replaced by the concept of tension controlled section which is defined as one having a maximum net tensile strain of 0.005 or more.

If a pseudo-ductile model is used, the designer must realize that the member recovery will be essentially elastic. Minor damage to the concrete will occur at large deformations, but no "yielding" of the reinforcement will occur. In seismic zones, there will be little or no energy dissipation resulting from the large deformations.

4.3—Constitutive behavior and material properties

Chapter 3 provides some guidance for the material properties for FRP reinforcement. Since variation in fiber content and manufacturing quality control will affect both the strength and the elastic modulus, a designer should verify the properties of the actual material being used. The ultimate tensile strength of the FRP reinforcement must include consideration of the statistical variation of the product. Some researchers suggest that the maximum strength be taken as the average strength minus three standard deviations (Mutsuyoshi 1992). This assumes that statistical records are available and that they are representative of FRP productions.

Use of the Whitney rectangular stress block is satisfactory for determination of the concrete strength behavior, although several researchers have used more complex constitutive rules for the concrete stress versus strain behavior.

The specific material properties lead to a number of design considerations. First, the moduli of elasticity of most FRP reinforcements are lower than that of steel. This means that larger strains are needed to develop comparable tensile stresses in the reinforcement. If comparable amounts of FRP and steel reinforcement are used, the FRP reinforced beam will have larger deflections and crack widths than the steel reinforced section.

FRP reinforcements' strength is time dependent. Like a concrete cylinder, FRPs will fail at a sustained load considerable lower than their short term static strength. At the present time, most designers and researchers are limiting the sustained load in FRP reinforcements to 50-60 percent of the static tensile strength. It was reported that the time-dependent creep strength of Polystal GFRP is about 70 percent of the short-term strength (Miesseler, 1991). However, others reported a linear relationship between sustained stresses and logarithm of time (Gerritse 1991). In light of these results, a lower sustained stress is advisable for GFRP reinforcement in the presence of aggressive environments.

4.4—Design of bonded FRP reinforced members

4.4.1 *Flexural behavior*—The flexural design of reinforced and prestressed concrete members with FRP reinforcement proceeds from basic equilibrium on the cross-section and constitutive behavior of the concrete and the FRP reinforcement. Unlike steel reinforcement, no constant tensile force may be assumed after yield point. The stress in reinforcement continues to increase with increasing strain until the reinforcement ruptures. The only condition of known forces in an FRP reinforced beam is the balanced condition where the concrete fails in compression at the same time that the reinforcement ruptures. This could be defined as the balanced ratio ρ_{br} and is given as (Dolan, 1991):

$$\rho_{br} = 0.85\beta_1 f_c'/f_{pu}\ \varepsilon_{cu}/(\varepsilon_{cu}+\varepsilon_{pu}-\varepsilon_{pi})$$

where

ε_{cu} is the ultimate concrete strain
ε_{pu} is the ultimate strain of the tendon
ε_{pi} is the strain due to the prestressing including losses
f_c' is the compression strength of the concrete
β_1 is a material property to define the location of the neutral axis from the depth of the compression block
f_{pu} is the ultimate tensile stress of the tendon

If the reinforcing ratio ρ is slightly less than ρ_{br}, failure will occur by rupture of the tendon and the concrete will be near its ultimate stress conditions. If $\rho < \rho_{br}$, the flexural member will fail by rupture of the tendon and the concrete stress state must be determined to locate the compression centroid. If $\rho > \rho_{br}$, compression failure of the concrete will occur first. The percentage of reinforcement should be selected to ensure formation of cracks and considerable deformation before failure to provide the "warning behavior" commonly used for concrete structures.

At the present time, there is insufficient data to accurately define a capacity reduction factor ϕ for bonded FRP reinforced beams. For beams with a $\rho < \rho_{br}$, a ϕ factor of 0.85 may be a reasonable assumption since the failure can be made analogous to a shear failure. However, it has been shown that this condition is practically unattainable in nonprestressed flexural members since deflection becomes excessive (Nanni, 1993). For $\rho > \rho_{br}$, a ϕ factor of 0.70 may be more appropriate since failure due to crushing of the concrete in compression. A minimum amount of flexural reinforcement should be used to provide an adequate post-cracking strength to prevent brittle failure at first cracking.

Researchers (Faza 1991, Brown et al 1993) have reported that ACI 318 Code strength equations conservatively predict the flexural strength of FRP reinforced members. If the reinforcement ratio is near ρ_{br}, the ACI equations will be conservative because there is a reserve tensile capacity in the reinforcement. ACI strength equations are not valid for $\rho < \rho_{br}$.

4.4.2 *Flexural cracking*—Excessive cracking is undesirable because it reduces stiffness, enhances the possibility of deterioration, and adversely affects the appearance of the beams. Since FRP reinforcements are not subject to the same corrosion mechanisms as steel, the crack width limitations established by ACI Committee 318 may not be applicable. The crack width will be dependent upon the physical bonding characteristics of the reinforcement and its modulus. Research cited in Chapter 5 provides guidance for glass FRP reinforcement. Guidance for other FRP materials and configurations is not readily available.

4.4.3 *Deflections*—The deflection of FRP reinforced members will be greater than comparable steel reinforced members because of the lower modulus of elasticity of the FRP. This leads to greater strains to achieve comparable stress levels and to lower transformed moment of inertia. Deflection limitations proposed by ACI 318 are independent of the concrete strength and reinforcement. They should remain applicable to FRP reinforced sections.

Three approaches are possible for the prediction of deflection in FRP prestressed members. The first method involves solving for the curvature at several sections along the member and integrating the moment curvature diagram. This is a first principal approach and is applicable to all FRP materials. The other two approaches use the effective moment of inertia.

Chapter 5 describes a modified moment of inertia for glass FRP reinforcement. This approach has the benefit of extensive correlation with test data. (GangaRao and Faza, 1991)

An alternative approach is to use the existing ACI deflection equations (Branson). The cracked moment of inertia uses the transformed FRP section. The ACI equations are modified to use a 4th or 5th power ratio for the transformed sectors (Brown et al, 1993) instead of a 3rd power for the computation of the effective moment of inertia. This effectively softens the section and results in a reasonable deflection prediction.

4.4.4 *Development length*—The development length depends upon the surface of the FRP reinforcement. While guidance is given in Chapter 5 for helically wrapped glass FRP reinforcement, these results may not be universally applicable to all FRP reinforcement. For example, Mitsui's FiBRA™ has a deformed surface due to braiding and Tokyo Rope's CFCC™ (Mutsuyoshi 1990) has a roughened surface due to the final fiber wrapping. Technora's™ rod (Kakihara, 1991, Kimura et al 1989) has an external helical wrap while Mitsubishi's Leadline™ has a depression in the rod surface. These conditions are sufficiently different to suggest that more research is needed prior to the establishment of generic design guidelines.

4.4.5 *Transfer length*—There are currently no comprehensive data on the transfer length of FRP prestressing tendons. Tokyo Rope's CFCC tendon (Mutsuyoshi, 1991) has a surface which is considerably rougher than a 7 wire steel strand. Researchers have reported (Zhao, 1994, Maaman et al 1993, Yonekura et al 1993 and Santoh 1993) splitting at the end of prestressed members which suggests that the transfer length is shorter than that of steel. Hand wound tendons made of several smooth FRP rods have less interlock than the more tightly wound steel strand. These tendons would be expected to have longer transfer lengths. If a design has a critical transfer length requirement, verification of the transfer length by physical testing should be required.

4.5—Unbonded reinforcement

Unbonded reinforcement is typically found in prestressing applications. Parafil™ (Burgoyne 1988) is a commercially available product which uses no resin matrix and is indicative of unbonded FRP reinforcement. Unbonded tendons require a reliable anchorage. The anchorage must develop the ultimate tensile strength of the tendon and be suitable for prestressing applications. The most common anchors use epoxy to contain the tendon. The long term performance of these anchors is dependent upon the resin and few durability tests have been conducted. The mechanical anchor of the Parafil™ tendon avoids the use of epoxies.

If the sustained load on an unbonded tendon is maintained below 60 percent of its ultimate strength, it is very difficult to create a flexural condition that will strain the tendon to its ultimate capacity. The capacity reduction factors for members with FRP unbonded tendons may be similar to that of steel reinforced members except that consideration of the anchorage reliability must be included.

4.5.1 *Flexural strength*—The flexural strength of unbonded tendons is determined by the tensile stress in the tendon. This stress is found by integrating the change in strain along the length of the beam. The change in strain is a function of the depth of the beam, the loading and the eccentricity of the tendon. For members with a span to depth ratio greater than 30, there is virtually no increase in effective prestressing stress. This is because the increase in strain is small and the modulus of elasticity of the FRP tendon is less than that of steel. For members with span to depth ratios less than thirty, the basic integration is required.

Equations given in ACI 318 for steel unbonded tendons are derived from providing a lower bound to the results of available test data. Since the modulus of elasticity of FRP tendons is less than that of steel, use of the ACI 318 equations can be expected to over predict the stress increases in FRP tendon. However, the formula developed by Naaman et al (Naaman et al 1991), although more complex than the ACI formula, should be applicable to unbonded FRP tendons.

4.5.2 *Deflections*—Deflections of an uncracked concrete section reinforced with an unbonded FRP tendons may be computed using the guidelines of ACI 318. Once the section cracks, the member will have a small number of large cracks. The lack of strain compatibility within the section precludes accurate determination of the member deflection.

4.6—Bonded external reinforcement

Strain compatibility between the reinforced concrete section and the bonded plate is the principal method of computing the member's flexural strength. The designer must recognize that three possible failure modes will exist. These are the tensile strength of the plate, the interlaminar shear strength of the adhesive between the concrete and the plate, and the shear failure of the concrete immediately above the adhesive.

The best performing designs include plates which have been bonded to the full length of the member. Additionally, vertical reinforcement is provided to retard peeling at the end of the FRP plate.

4.7—Shear design

The vast majority of the research data is for members that are not shear critical. There are a very small number of tests with FRP shear reinforcement. Experimental results of shear anchorage indicate that the stirrups will fail in the corners due to premature failure at the bend. The few tests that have been completed with FRP stirrups suggest that the shear resistance is less than predicted. This may be due to the large cracks that result from the lower modulus of elasticity of the stirrups. The larger cracks can reduce several components of the concrete contribution to shear resistance. Members with FRP longitudinal reinforcement and steel stirrups did not report unusual shear behavior (Rizkalla et al 1994). Special attention should be devoted to the reduced dowel contribution of FRP reinforcement in presence of shear cracks (Jeong et al 1994).

External shear reinforcement in the form of bonded FRP overwrap has been applied to beams with insufficient shear strength. These tests (Rider 1993) have indicated that this procedure provided sufficient shear resistance to allow full development of the flexural capacity of the beam.

CHAPTER 5—BEHAVIOR OF STRUCTURAL NONPRESTRESSED ELEMENTS

This chapter summarizes diverse research findings regarding the performance of FRP as a main structural reinforcement for nonprestressed concrete flexural members. Equations presented herein explicitly represent research results and products of the investigator as referenced.

5.1—Strength of beams and slabs reinforced with FRP

The wide-spread implementation of FRP as a reinforcement for concrete structural members requires: (1) a comprehensive understanding of how these two materials behave together as a structural system, and (2) analytical techniques that reliably predict the composite behavior. In this regard, three important physical characteristics of FRP materials must be considered: (1) high tensile strength, (2) low modulus of elasticity, and (3) linear-elastic brittle behavior to failure. Substitution of FRP for steel on an equal area basis typically results in significantly higher deflections with wider crack widths and greater flexural strength. As a consequence, deflection limitations will likely be an important parameter in design considerations. This behavior is due to higher tensile strength and lower modulus values of FRP, assuming good force transfer.

Flexural failure of concrete members reinforced with currently available FRP materials can only be brittle. This occurs either as a result of concrete crushing or FRP tensile rupture. This behavior differs from the behavior of concrete beams under-reinforced with steel. In addition, shear capacity is also likely to be significantly reduced as a result of increased crack width and reduced size of compressive stress blocks.

5.1.1 *Flexural strength*—Nawy and Neuwerth (1971) monotonically tested 20 simply supported rectangular beams reinforced with GFRP and steel reinforcing bars. Samples were loaded with two concentrated loads applied at the third span points. All beams were 7 in. (178 mm) deep by 3.5 in. (89 mm) wide by 72 in. (1800 mm) long with an effective depth that varied slightly from 6.25 in. (159 mm) to 6.5 in. (165 mm). The beams were grouped in five series with four beams each. The four beams in each series included: two beams reinforced with FRP reinforcing bars with a bar diameter = 0.118 in (3 mm), one beam reinforced with an equal number of steel bars with a steel bar diameter = 0.125 in. (3.2 mm); and one beam reinforced with FRP bars and with chopped steel wire in the concrete mixture. Stirrups were not provided in any of the beams. The percentage of reinforcement varied from 0.19 to 0.41 percent for FRP reinforced beams and from 0.22 to 0.45 percent for steel reinforced beams. Tensile strength and modulus of elasticity for the FRP were 155 ksi (1.1 GPa) and 7300 ksi (50.3 GPa), respectively. Concrete strength ranged from 4.10 ksi (28.3 MPa) to 5.13 ksi (35.4 MPa). The tests revealed an increase in ultimate moment capacity for steel reinforced beams as the percentage of reinforcement was increased. The reinforcing ratio of FRP beams did not affect moment capacity because the beams failed by compression of the concrete, thus not developing the full capacity of the FRP. The authors suggested that because the modulus of FRP is only slightly higher than that of concrete, limited tensile stress can be transmitted from the concrete to the FRP reinforcement. Thus, most of the tensile load is initially absorbed by the concrete. When the tensile strength of the concrete is exceeded, cracks form and this cracking process continues until the cracks extend over three-fourths of the beam span at a spacing of approximately 4 in. (102 mm) to 6 in. (152 mm). When further load was applied, the concrete crushed.

In a second study, Nawy and Neuwerth (1977) tested 14 simply supported beams, 12 of which were longitudinally reinforced with glass FRP bars. No shear reinforcement was used. All beams were 10 ft (3000 mm) long by 5 in. (127 mm) wide and 12 in. (305 mm) deep with an effective depth of 11.25 in. (286 mm) and loaded with two concentrated loads, at the one-third span points. FRP reinforcement ranged from 0.65 percent (2 FRP bars) to 2.28 percent (7 FRP bars). The FRP reinforcing bars were of 0.25 in. (6.4 mm) diameter and had tensile strength and modulus values of 105 ksi (723 MPa) and 3600 ksi (24.8 GPa), respectively. Concrete strength ranged from 4.30 ksi (29.6 MPa) to 5.80 ksi (40 MPa). Analysis of test results indicated that behavior

Table 5.1—Reinforcing bars

Reinforcement type	Area in.^{2} (mm^2)	Modulus of elasticity x 10^6 psi (GPa)
Aramid	0.059 (37.75)	10.30 (71)
Carbon	(0.068 (43.63)	50.76 (350)
Glass	0.130 (82.38)	10.73 (74)
D13 Steel	0.200 (126.7)	27.70 (191)

of the beams with respect to cracking, ultimate load, and deflection could be predicted with the same degree of accuracy as for steel reinforced concrete beams. The ratio of the observed to calculated moment capacity was close to 1.0, with a mean value of 1.09 and a standard deviation of 0.18. With respect to serviceability, a working load stress level in the FRP of 15 percent of its tensile capacity was discussed for concrete strengths between 4 ksi (27.6 MPa) and 5 ksi (34.5 MPa).

Larralde et al. (1988) examined flexural and shear performance of concrete beams reinforced only with GFRP reinforcing bars and in combination with steel reinforcing bars. The study used test results to determine if the theory used for steel reinforced concrete can be used to predict the performance of concrete beams reinforced with GFRP reinforcing bars. Four beam specimens, 6 in. (152 mm) wide by 6 in. (152 mm) high, with 1 in. (25 mm) of cover, and 5 ft (1500 mm) of length were cast. Beams one and two were simply supported and loaded at a single location; and beams three and four were simply supported and loaded at two locations. Beam one was reinforced with three-#4 steel bars; beam two with two-#4 FRP and one #4 steel reinforcing bar; beam three with three #4 FRP reinforcing bars, and beam four with two-#4 steel and one-#4 FRP reinforcing bars. Concrete strength for beams one and two was 4.24 ksi (29.2 MPa) and for beams three and four 3.73 ksi (25.7 MPa). FRP modulus and tensile strength were 6000 ksi (41.4 GPa) and 150 ksi (1.0 GPa), respectively. Deflections were calculated using the moment of inertia of the cracked transformed section, neglecting the tensile strength of concrete below the neutral axis. Ultimate load capacities were calculated using (a) transformed sections (b) linearly elastic composite sections, (c) limiting concrete compressive strength to f_c', (d) equilibrium**,** and (e) nonlinear stress-strain distribution for concrete. A flexural failure occurred in beam one by yielding in the steel and was followed by concrete crushing. Diagonal tension failures occurred in beams two, three, and four; therefore theoretical flexural strength could not be compared with test results and no conclusion was derived regarding the accuracy of flexural strength prediction for concrete beams reinforced with FRP. The authors recognized that a methodology for shear strength prediction of FRP reinforced concrete needs to be developed independently from steel/concrete equations.

Saadatmanesh and Ehsani (1991a) tested six concrete beams, longitudinally and shear-reinforced with different combinations of GFRP and steel reinforcing bars. The FRP tensile strength and modulus values were 171 ksi (1.2 GPa) and 7700 ksi (53.1 GPa), respectively. All beams had a clear span of 10 ft (3.05 m), were simply supported, were loaded at two points and had a shear span of 51 in. (1300 mm). Sample cross-sections were 8 in. (203 mm) wide by 18 in. (457 mm) high. The study focused on experimentally determining the feasibility of using FRP bars as reinforcement for concrete beams. Steel stirrups provided adequate shear strength to the longitudinal GFRP reinforced beams to result in either a flexural compression or tension failure (tensile rupture of the FRP bar). Based on the large number of uniformly distributed cracks, it was concluded that a good mechanical bond developed between the FRP bars and concrete. Specimens reinforced with FRP stirrups and steel longitudinal reinforcement failed as a result of yielding in the longitudinal bars. This was followed by large plastic deformation until a concrete compression failure occurred. Calculated maximum loads using FRP properties were reasonably close to the experimental measured values.

Satoh et al. (1991) tested four simply supported concrete beams each with a different type of fiber reinforcement. The type of reinforcement, the area of reinforcement, and the respective modulus of elasticity are given in Table 5.1. All samples were 3.28 ft (1000 mm) in length by 7.9 in. (200 mm) wide by 5.9 in. (150 mm) high with an effective depth of 4.72 in. (120 mm), were simply supported and were loader at two points with a shear span of 19.7 in. (500 mm). All beams were reinforced with steel stirrups 0.39 in. (10 mm) in diameter at a 2.8 in. (70 mm). All four samples failed in flexure. The ratio of experimental failure load to predicted flexural strength (using elastic theory) for beams reinforced with AFRP, CFRP, GFRP grids and D13 bars was 0.75, 0.86, 0.98, and 1.04, respectively. Tensile stress in the reinforcement was measured using bonded strain gauges located at midspan. Experimental reinforcement strain results compared well with predicted values calculated using elastic theory. Based on these results, the authors concluded that the failure load for concrete beams reinforced with FRP can be calculated using elastic theory applicable for reinforced concrete members. Theoretical load-deflection behavior was predicted using an effective moment of inertia as developed by Branson (1977). Experimental load-deflection behavior was reported to agree well with theoretical predictions.

Goodspeed et al. (1991) investigated the cyclic response of concrete beams, 6 ft (1800 mm) long, 8 in. (203 mm) wide and 4 in. (101 mm) high, reinforced with a two dimensional FRP grid. Tensile strength and modulus of the FRP were 120 ksi (827 MPa) and 6000 ksi (41.4 GPa), respectively. Concrete strength was between 4.2 ksi (29 MPa) and 4.6 ksi

Table 5.2—Specimen details

Number of bars	Longitudinal type	Shear reinforcement
3	#3 sand coated or deformed	#2 steel or #3 deformed FRP
2	#4 deformed FRP	#2 smooth FRP bars or #3 deformed FRP bars
2	#8 deformed FRP	#2 steel bars of #3 FRP deformed
2	#3 deformed FRP	#2 steel bars of #3 deformed FRP
5 or 3	#3 sand coated FRP #4 sand coated FRP	#3 deformed FRP

(31.7 MPa). Test samples were reinforced at 110 percent of a balanced strain condition. Samples were simply supported and loaded at two locations with a shear-span of 24 in. (610 mm). The following two cyclic load schedules were used:

1) The first series was subjected to 20 cycles of 0 to 50 percent of maximum monotonic capacity
2) The second series was subjected to 10 cycles for each loading case as follows: 0 to 20 percent, 0 to 35 percent, 0 to 20 percent, 0 to 50 percent... 0 to 80 percent of maximum monotonic capacity for a total of 80 cycles

The results of the first series showed an increase in deflection with each cycle. The amount of increased deflection decreased with each cycle asymptotically. Also, the increase in permanent deflection was about half the increase in maximum deflection. Results from the second test series showed an increase in deflection each time the maximum applied load was increased, for example from 35 percent to a 50 percent load case. There appeared to be no increase in deflection when the load was reduced and cycled at 20 percent of maximum. The load-deflection curve drawn with the first load-deflection points from each time a larger load was cycled, appeared to follow the load-deflection curve of a monotonically loaded sample of identical design. Only minor differences in crack pattern between cyclically and monotonically loaded samples was observed indicating crack propagation stabilized after a relatively few number of cycles.

Bank et al. (1991) tested seven full-size concrete bridge deck slabs, six of which were reinforced with pultruded GFRP gratings and one with steel reinforcing bars. The test span length was 8 ft (2400 mm), with a projection of 1 ft (305 mm) on either side of the supports. Slab width was 4 ft (1220 mm) and total depth was 8.5 in. (216 mm). One and one half in. (32 mm) cover was used and concrete strength was 4.93 ksi (34 MPa). The slabs were designed for a live load moment designated by AASHTO (1989) Article 3.24.1 using a nominal HS-25 loading with a live load impact factor of 30 percent bringing the nominal service load to 26 kips (11.8 MN). This load was then used to calculate a service limit state deflection of d_{allow} = 0.192 in. (3.3 mm). One slab of each grating type and the steel reinforced slab were tested monotonically to 26 kips (11.8 MN), then subjected to 10 loading unloading cycles of 0 to 26 kips (0 to 11.8 MN), and then loaded monotonically to failure. The loading unloading cycles for the 3 remaining slabs were as follows: 0 to 26 kips (0 to 11.8 MN) for 10 cycles, 0 to 52 kips (0 to 23.6 MN) for 10 cycles, 0 to 26 kips (0 to 11.8) for 10 cycles, 0 to failure. Behavior of all FRP reinforced slabs was similar. Initial cracking occurred between 10 and 15 kips (4.5 and 6.8 MN) followed by development of flexural cracks. At loads near ultimate, flexural shear cracking was observed. Failure was the result of concrete crushing followed immediately by propagation of a flexural-shear crack in a diagonal path towards the outer support. This crack was intercepted by the top surface of the FRP grating and redirected horizontally along the top surface of the grating to the free end. No failure of the FRP grating was observed. The steel reinforced slab failed by yielding of the reinforcing bars and subsequent crushing of the concrete. Service load midspan deflections for all FRP reinforced slabs were close to the allowable limit of 0.192 in. (4.9 mm). Deflection was found to stabilize after a limited number of cycles. All slabs failed at loads in excess of three times the service load.

Faza and GangaRao (1992a) investigated the flexural performance of simply supported rectangular concrete beams with an effective length of 9 ft (2700 mm), reinforced with GFRP reinforcing bars and subjected to load applied at two locations. Tensile strength of the FRP reinforcing bars ranged from 80 ksi (551 MPa) for #8 bars and 130 ksi (896 MPa) for #3 bars while the concrete strength ranged from 4.2 ksi (29 MPa) to 10 ksi (69.0 MPa). The 27 concrete test beams were 6 in. (152 mm) in width, 12 in. (305 mm) in height and contained different configurations of FRP reinforcements (i.e. reinforcing bar size, type of reinforcing bar), and type of stirrups (steel, FRP smooth, FRP ribbed). Five groups of beams were tested, details of the test beams are given in Table 5.2. From the test results, it was concluded that:

1) In order to take advantage of the high FRP reinforcing bar ultimate strength [i.e. 80 to 130 ksi (551 to 896 MPa)], use of high-strength concrete instead of normal-strength concrete [10 ksi (69 MPa) versus 4 ksi (27.6 MPa)] is essential. The ultimate moment capacity of high-strength concrete beams [f_c' = 10 ksi (69 MPa)] was increased by 90 percent when an equal area of FRP reinforcing bars of ultimate tensile strength of 130 ksi (896 MPa) were used in lieu of mild steel reinforcing bars [60 ksi (414 MPa)]. The ultimate moment capacity of concrete beams reinforced with sand-coated FRP reinforcing bars is about 70 percent higher than that of beams reinforced with steel reinforcing bars for the same area and concrete strength
2) The use of sand-coated FRP reinforcing bars, in addition to high strength 6 to 10 ksi (41 to 69 MPa) concrete, was found to increase the cracking moment of the beams and to reduce the crack widths, in addition to eliminating the sudden propagation of cracks toward the compression zone. This behavior was related to a better force transfer between the sand-coated FRP reinforcing bar and concrete. The crack pattern was very similar to a pattern expected of a beam reinforced with steel reinforcing bars
3) Beams cast with higher strength concrete and rein-

forced with two-#3, three-#3, and two-#4 FRP reinforcing bars failed when the FRP bars reached ultimate tensile strength

4) Beams reinforced with two #8 FRP reinforcing bars failed in shear before reaching the ultimate tensile strength of the bars; using high strength concrete 7.5 ksi (51.7 MPa) increased the moment capacity by 50 percent over beams cast with normal strength concrete 4.2 ksi (29 MPa)

Zia et al. (1992) investigated the flexural and shear behavior of simply supported rectangular concrete beams, approximately 3 in. (76.2 mm) wide by 4.5 in. (114.3 mm) deep and 96 in. (2400 mm) long, loaded at two locations and reinforced with a three-dimensional continuous carbon fiber fabric. The three-dimensional fabric was roughly 1.6 in. (41 mm) wide by 3.6 in. (91 mm) high. Longitudinal reinforcing bars were spaced at 0.8 in. (20 mm) intervals across the fabric width and 1.2 in. (30 mm) intervals through its depth. Transverse bar elements (shear reinforcement) were longitudinally spaced at 1.2 in. (30 mm) intervals. Total gross cross-sectional area of the 12 longitudinal FRP bars was 0.078 in.^{2} (127.8 mm^2). Tensile strength and modulus of the CFRP was 180 ksi (1.24 GPa) and 16 x 10^6 psi (113 GPa), respectively. Three beams were tested in flexure with a shear span of 39 in. (990 mm) and an 18 in. (457 mm) region of constant moment. Concrete strength for these three samples was 2.35 ksi (16.2 MPa), 2.82 ksi (19.4 MPa) and 2.95 ksi (20.3 MPa), respectively. The beams were under-reinforced relative to a balanced design. After initial cracking and increasing load, many closely spaced small vertical cracks developed. At ultimate load, the longitudinal carbon FRP bars ruptured successively from the lowest layer upward.

Bank et al. (1992a) tested nine slabs simply supported and loaded at two locations, having shear-span to effective depth ratios of approximately three (a/d = 3) and reinforced with a variety of molded and pultruded GFRP gratings. Slabs of two different sizes were fabricated. The first six slabs measured 56 in. (1.4 m) long by 12 in. (305 mm) wide by 4 in. (102 mm) thick and the second group of three slabs measured 42 in. (1100 mm) long by 12 in. (305 mm) wide by 4 in. (102 mm) thick, one of which was reinforced with epoxy-coated steel reinforcing bars. Reinforcement was placed in the tension zone with 0.5 in. (13 mm) of cover. Concrete strength ranged from 2.65 ksi (18.3 MPa) to 4.10 ksi (28.3 MPa). In addition to load and deflection data, strain was measured on the FRP grating and on the concrete. Following initial cracking, flexural cracks developed in the constant moment region at regular intervals of about 3 in. (76 mm). With increasing load, diagonal tension shear cracking developed in the shear span. Flexural compression failure occurred in three of the first six slabs, and the remaining slabs failed in shear. The slabs that failed in compression had the lowest concrete strength. In several of the shear failures, the concrete below the reinforcement in the shear-span was completely separated from the slab. In all slabs, the experimental shear force V_{exp} was significantly larger than (V_c = $2.0\sqrt{f_c'}bd$). The effective flexural stiffness EI of the slabs was calculated using deflection data, strain data, and a transformed cracked-section theoretical method. Reasonable agreement between these three methods was achieved, indicating that the effective stiffness of FRP-reinforced concrete slabs can be predicted using theoretical methods with some degree of confidence.

Bank and Xi (1992b) tested the performance of four full-scale concrete slabs 20 ft (6100 mm) by 4 ft (1200 mm) by 8.5 in. (216 mm), doubly reinforced (top and bottom) with 2 in. (51 mm) deep, commercially produced pultruded FRP gratings having longitudinal bar intervals of 3 in. (76 mm) and 2 in. (51 mm) on-center, respectively, and transverse bars located at 6 in. (152 mm) intervals. The cross-sectional profile of the longitudinal bars resembled that of a “T” and had an approximate area of 0.54 in.^{2} (350 mm^2). A fifth sample reinforced with #5 steel bars (Grade 60) located 4.5 in. (114.3 mm) on center (top and bottom) and having the same dimensions as the four FRP slabs was tested for control purposes. All samples were provided with 1 in. (25 mm) concrete cover, top and bottom, and supported as continuous beams over two spans of 8 ft (2400 mm). Two equal loads of magnitude P were placed 3.38 ft (1000 mm) from the center support and applied over 10 in. (254 mm) by 25 in (635 mm) by 2 in. (51 mm) thick steel plates. The FRP tensile strength and modulus values (as reported from manufacturers' data) were 60 ksi (414 MPa) and 5000 ksi (34.5 GPa), respectively. Slabs were loaded as follows: first, under a monotonically increasing load to 26 kips (11.8 MN) [see service load, Bank et al. (1991)], then subjected to 10 loading unloading cycles of 0 to 26 kips (0 to 11.8 MN), and finally loaded monotonically to failure. Slab performance was evaluated with respect to ultimate and serviceability limit state criteria. The behavior of all FRP grating-reinforced slabs was similar. Flexural cracking developed early in both the positive and negative moment regions and were in line with the transverse bar locations. All slabs experienced shear failure in the short shear-span between the middle support and the load point. The ratio of failure to service load for FRP reinforced slabs were 4.26, 3.89, 4.17, and 4.16. For the steel reinforced slab, this ratio was 3.34. No evidence of shear cracking was observed prior to failure. At higher loads, nonlinear compressive strain was recorded in all FRP gratings. This was assumed to be the result of localized compression failure in the gratings. The local radius of curvature in the positive moment region generally satisfied a recent AASHTO draft serviceability specification. However, in the negative moment region this criterion was violated. Service load deflections were well below the L/500 limit, where L is the length of the beam.

Nanni et al. (1992c) tested five concrete beams reinforced with hybrid reinforcing bars, steel deformed bars, and FRP reinforcing bars. A beam length of 3.9 ft (1.2 m) and cross-sectional dimensions of 3.9 in. (100 mm) wide by 5.9 in. (150 mm) deep were used. Samples were simply supported and loaded at two locations with a shear span of 13.8 in. (350 mm) and a constant moment length of 3.9 in. (100 mm). Each beam was reinforced with four identical reinforcing bars, two in the compression zone and two in the tension zone. In all beams, shear reinforcement consisted of closed stirrups made of smooth steel wire [f_y = 70 ksi (483 MPa); E = 28.3 x 10^6 psi (195.2 GPa)], 0.16 in. (4 mm) in diameter and spaced at 1.57 in. (40 mm). Clear cover at all surfaces was 0.67 in. (17 mm). Concrete compressive strength was 6320 psi (43.6 GPa). The only parameter varied in the five specimens was the type of longitudinal reinforcement provided. The five beams were reinforced as follows:

Beam 1) Deformed steel bars f_y = 54 ksi (373 MPa); E = 30.3 x 10^6 psi (208.9 GPa); A = 0.079 in.^{2} (51 mm^2); diameter = 0.31 in. (8 mm)

Beam 2) Braided aramid FRP reinforcing bars f_u = 216 ksi (1489 MPa); E = 9.41 x 10^6 psi (64.9 GPa); A = 0.068 in^2 (44 mm^2); diameter = 0.31 in. (8 mm)

Beam 3) Same as Beam two, but the FRP reinforcing bars were coated with silica sand to improve mechanical bond

Beam 4) Hybrid reinforcing bars consisting of high-strength steel core f_y = 1373 MPa (199 ksi); E = 196 GPa (28 x 10^6 psi); A = 28 mm^2 (5.5 in.^{2}) and a braided aramid FRP skin f_u = 489 MPa (70.9 ksi); E = 64.9 GPa (9.4 x 10^6 psi); A = 44 mm^2 (6.8 in.^{2}), diameter = 14 mm (0.55 in.)

Beam 5) Same as Beam four, but the FRP skin was coated with silica sand to improve mechanical bond

Load-deflection behavior for the different reinforcing bar types were characterized as follows:

1) For steel reinforcing bars, a typical three-stage behavior of an under-reinforced concrete beam consisting of uncracked-section, cracked-section linear elastic to yield, and post-yield of reinforcement
2) For FRP reinforcing bars, a two-stage behavior reflecting, uncracked section and cracked-section linear-elastic to failure
3) For hybrid reinforcing bars, a three-stage behavior typical of under-reinforced steel beam characterized by uncracked section and linear-elastic response followed by steel core yielding before ultimate failure

Test results showed that sand-coated reinforcing bars performed better than the corresponding uncoated reinforcing bars. Relative to ultimate flexural capacity, coating the FRP reinforcing bars and hybrid reinforcing bars with sand increased flexural capacity by approximately 25 percent. Smaller crack-widths and higher post-crack flexural rigidity were also reported for the sand-coated reinforcing bars as compared with the corresponding uncoated reinforcing bars. For all beams, it was stated that ultimate strength could be predicted on the basis of the material properties of the concrete and reinforcement as is done with conventional reinforced concrete.

Faza and GangaRao (1993) investigated the behavior of full-size concrete bridge decks 12 ft (3700 mm) long by 7 ft (2100 mm) wide and 8 in. (203 mm) deep reinforced with sand-coated FRP reinforcing bars. The slabs were supported on steel stringers running transverse to the 12 ft (3700 mm) slab length. Three test sets, each consisting of two tests, were run; the first set was noncomposite construction (studs welded to the stringers passed through holes in the concrete deck to eliminate shear transfer) with stringer spacing of 3 ft (914 mm) for one slab and with 5 ft (1524 mm) stringer spacing for the other. The second set developed composite action (the space surrounding the studs in the deck holes was grouted). The third set was composite construction (the decks were cast on the stringers) the stringer spacing was 6 ft (1800 mm). The decks were designed for one-way bending, two 6 ft (1800 mm) long stirrups were used to create a single temperature and shrinkage reinforcing bar. A three-point bending setup was used; the center load was either a pad/(s) load or a load distributed over the 7 ft (2134 mm) width. In all cases, the load-deflection curve was linear. Measured strain on the FRP longitudinal reinforcement were greater than those in the transverse reinforcement.

5.1.2 *Shear strength*—Shear testing was conducted by Zia et al. (1992) on six simply supported samples. For this test a single concentrated load at center span was applied with shear span-to-depth ratios (a/d) of 2.13, 2.55, and 3.62. In all cases, no shear failure developed. Failure was, instead, due to tensile rupture of the longitudinal FRP bars.

Larralde (1992) tested a series of eight FRP-grating/concrete composite slabs in which the shear span-to-depth ratio and concrete deck thickness was varied in an attempt to force different types of failures. Concrete deck thickness ranged from 1.75 in. (44 mm) to 5.5 in. (140 mm). All specimens were simply supported and loaded at two locations with a/d ranging from 3.94 to 9.49. Four of the eight slabs were also reinforced with 0.25 in. (6.35 mm) vertical studs consisting of either FRP reinforcing bars or steel bolts. Concrete strength ranged from 4.30 ksi (29.6 MPa) to 4.65 ksi (32.1 MPa). Test results showed that for samples with a/d ratios of 7.7 or greater, failure occurred by crushing of the concrete. For these samples, the calculated flexural capacity was very close to the test results. For a/d ratios of five or less, failure occurred as a result of diagonal tension cracking. The vertical studs did not prevent shear failure.

Porter et al. (1993) examined the performance of FRP dowel bars (E-glass fiber encapsulated in a vinyl ester matrix) in full-scale laboratory pavement slabs and FRP dowel bars and steel dowels in actual highway pavement. The objective was to compare static, fatigue, and dynamic behavior of FRP dowels to those for steel dowels. Additionally, a laboratory test method was developed for the evaluation of highway pavement dowels which approximates actual field conditions. Testing of four full-scale laboratory pavement specimens was completed, two with 1.5 in. (38.1 mm) diameter steel dowels with 12 in. (304.8 mm) spacing. By simulating the in-service performance of an actual highway pavement, the applicability of FRP dowels as pavement load transfer devices was evaluated relative to that of steel dowels. Static and fatigue testing of full-scale specimens showed that the 1.75 in. (44.5 mm) FRP dowels spaced at 8 in. (203.2 mm) performed at least as well as 1.5 in. (38.1 mm) steel dowels spaced at 12 in. (304.8 mm) in transferring static loads across the joint. FRP dowels spaced at 12 in. (304.8 mm) performed similar to that of the specimens with steel dowels. Both the FRP and steel dowels gave increasing relative displacement at the pavement joints as the number of load cycles increased. Fatigue tests were subjected to up to 10 million cycles. Equations for predicting shear strengths of the dowels were developed (Porter et al., 1993).

Field testing was conducted for two transverse contraction joints replacing the standard 1.5 in. (38.1 mm) steel dowels at 12 in. (304.8) spacing with 1.75 in. (44.5 mm) FRP dowels spaced at 8 in. (203.2 mm). Experimental testing indicated that performance of FRP dowels was equivalent to that of steel dowels. Additionally, no difference in joint performance was noted between FRP dowels and steel dowels during visual inspection.

Porter et al. (1992) also conducted a study on shear behavior and strength of FRP dowel bars subjected to accelerated aging. Overall, accelerated aging equivalent to 50 years in solutions of water, lime, and salt apparently had little or no effect on shear strength.

5.1.3 *Bond and development of reinforcement*—The evaluation of bond characteristics of FRP reinforcements is of prime importance in the design of FRP reinforced concrete members. Due to variations in FRP reinforcing products, bond characteristics are quite variable. Bond characteristics are influenced by factors such as:

1) Size and type of reinforcement (wires or strands)
2) Surface conditions (smooth, deformed, sand-coated)
3) Poisson's ratio
4) Concrete strength
5) Concrete confinement (e.g., helix or stirrups)
6) Type of loading (e.g., static, cyclic, impact)
7) Time-dependent effects
8) Amount of concrete cover
9) Surface preparations (braided, deformed, smooth)
10) Type and volume of fiber and matrix

Bond characteristics of GFRP bars were investigated by GangaRao and Faza (1991) by testing 20 concrete specimens. Different configurations of FRP reinforcement size, type (ribbed, sand-coated) and embedment lengths were tested. The specimens were tested as cantilever beams, to emulate the beam portion adjacent to a diagonal crack. Twelve pull-out cylinder specimens were tested. The following design equation was suggested for development length of GFRP:

$$l_d = K_1 \frac{f_u A_b}{\sqrt{f_c'}} \tag{5.1}$$

where

l_d = development length
A_b = reinforcing bar cross sectional area
f_u = reinforcing bar tensile strength
f = concrete compression strength

$$K_1 = \frac{1}{16}$$

Pleiman (1991) conducted more than 70 pull-out tests to examine the bond strength of GFRP reinforcing bars (E-glass fiber), Kevlar™ 49 reinforcing bars (AFRP) and steel bars. Three different diameters of GFRP reinforcing bars, namely 0.25 in. (6.4 mm), 0.37 in. (9.5 mm) and 0.5 in. (12.7 mm) and one diameter of FRP 0.37 in. (9.5 mm) were tested. Results indicated that AFRP and GFRP reinforcing bars exhibited similar behavior at a performance level below steel reinforcing bars. Two equations were proposed for calculating a safe development length (inches) for E-glass and Kevlar™ 49 FRP bars. They are K_1 = 1/20 and K_1 = $^1/_{18}$ respectively as defined in equation 5.1.

Chaallal et al. (1992) evaluated the development length of GFRP reinforcing bars (E-glass fibers and polyester resin, with a sand-coated surface). Pull-out tests were undertaken using normal-strength concrete, high-strength concrete, and grout. Three different rod diameters were used and the anchor length was varied from five times to ten times the rod diameter. A development length of $20d_b$ was recommended.

Daniali (1992) investigated the bond strength of GFRP bars (E-glass fibers and vinyl ester resin) by testing 30 beams having varying bar diameters and embedment lengths. All beams were 9.8 ft (3000 mm) long and 8 in. (203 mm) by 18 in. (457 mm) in cross-section and of the type described in the ACI Committee 408 report. The study concluded that if shear reinforcement was provided for the entire length of the specimen, development lengths of 8 in. (203 mm) and 17.3 in. (440 mm) would be required to develop ultimate tensile strength for #4 (16 mm) and #6 (23 mm) GFRP bars, respectively. However, all specimens reinforced with #8 bars failed in bond. The study identified the occurrence of premature bond failure under sustained load.

A study on bond of GFRP reinforcing bars was conducted (TAO 1994) on 102 straight and 90-deg hook specimens. New limits for allowable slip were introduced as 0.0025 in. (0.064 mm) at the free end, or 0.015 in. (0.38 mm) at the loaded end. According to this study, the basic development length l_{db} of straight GFRP reinforcing bars should be computed knowing the ultimate strength of the reinforcement and K_1 = 21.3 given in Eq. 5.1. To account for the influence of concrete cover, a factor 1.0 can be used with concrete cover of not less than two times the bar diameter. A factor 1.5 can be used with cover of one bar diameter or less. The development length l_d, computed as the product of the basic development length l_{db} and the confinement factors (1.0 or 1.5), should not be less than

$$l_{db} = 0.00035 d_b f_u \tag{5.2}$$

where d_b is the bar diameter. The bond strength developed for top reinforcing bars was found to be less than that of bottom bars. Therefore, a factor of 1.25 can be used for top reinforcing bars. Moreover, the development length l_d, computed as the product of the basic development length l_{db} and the applicable top bar factor should not be less than 15 in. (381 mm).

For hooked GFRP reinforcing bars with tensile strength equal to 75,000 psi (517 MPa), the basic development length, L_{hb} should be computed by

$$L_{hb} = 1820\frac{d_b}{\sqrt{f_c'}} \tag{5.3}$$

For reinforcing bars with tensile strength other than 75,000 psi (517 MPa), a modification factor f_u/75,000 should be used. When side cover and cover on bar extension beyond hook are not less than 2.5 in. (64 mm) and 2 in. (51 mm), respectively, a modification factor 0.7 should be used. Moreover, to prevent direct pull-out failure in cases where the hooked reinforcing bar may be located very near the critical section, the development length L_{dh} computed as the product of the basic development length L_{hb} and the applicable modification factors should be no less than eight times the bar diameter or 6 in. (152 mm).

Rahman and Taylor (1992) estimated deflections of flat slabs reinforced with FRP by the finite element (FE) method. Similar FE analysis closely predicted the deflections of both steel and FRP reinforced one-way slabs. The study found that a slab reinforced with a typical GFRP, having a tensile modulus of 5801 ksi (40 GPa), will deflect three to six times more than a steel-reinforced slab. Using a typical CFRP with a higher modulus of 11,602 ksi (80 GPa), the deflection could be reduced by 50 percent. If drop panels are added, the deflections become comparable to those of steel-reinforced slabs.

Several weaknesses in standard pullout tests (simply supported beams or pullout specimens) have been identified because they do not sufficiently account for all types of mechanical behavior. Many attempts have been made to find a better standard test method. Researchers (Porter et al. 1993) have developed a new technique that combines two test methods that individually account for these mechanisms. Beams were cast with the cantilever section similar to the Ferguson and Thompson test (Ferguson, 1966) but they also included concrete outcroppings extending from the side of the beam similar to those used by Mathey and Watsein (1961) (see Fig. 5.1). By loading beams on T sections, compressive effects of the load do not confine the reinforcing and therefore does not affect bond characteristics of the reinforcement. The cantilever section allows for investigation of FRP bars subjected to negative moments and can be adjusted by moving the reaction point, thus giving great flexibility in testing scenarios. FRP reinforced concrete cantilever beams have been successfully used in more than 100 full-scale tests. The embedment length L_d, for 0.325 in. and 0.5 in diameters was derived to be the following:

$$L_d = \frac{0.59 f_u A_b}{c_b^2 \sqrt{f_c'}} \tag{5.4}$$

where

f_u = ultimate tensile strength of the reinforcement (psi)
A_b = area of the rod (in.^{2})
c_b = circumference of the rod
f_c' = compression strength of the concrete (psi)

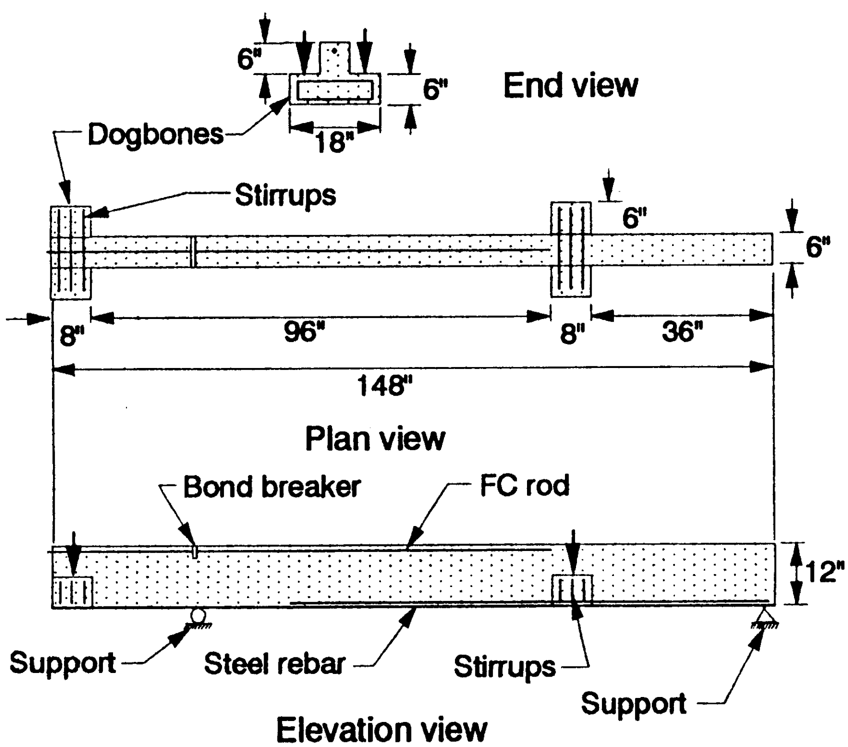

Fig. 5.1—The ISU bond-beam test

Eq. 5.4 is based upon zero end slip criteria. If $^1/_{10}$ in slip is allowed at the end of the embedment, Eq. 5.4 becomes:

$$L_d = \frac{0.42 f_u A_b}{c_b^2 \sqrt{f_c'}} \tag{5.5}$$

5.2—Serviceability

Serviceability of FRP reinforced flexural members is describe in terms of deflection and crack width limitations.

5.2.1 *Deflection considerations*—Nawy and Neuwerth (1971) determined that deflection of FRP-beams at ultimate load was approximately three times greater than that of the corresponding steel-reinforced beams.

Larralde et al. (1988) found that theoretical deflection predictions underestimated test results for loads above 50 percent of ultimate; deflection values were fairly well predicted at load levels up to approximately 30 percent of ultimate. The study suggested a procedure in which values of curvature calculated at different sections of the beam should be used to obtain a better estimate of deflection values.

Larralde and Zerva (1991) investigated the feasibility of using concrete for enhancing the structural properties of a box-type, molded GFRP grating. Although the FRP grating was designed to be used as a structural component independent of concrete, the low modulus of the FRP caused large deflections at load levels only a fraction of the ultimate load carrying capacity. Within this context, concrete is considered a stiffening agent employed to produce a composite section with more favorable structural properties. All samples were 22.5 in. (570 mm) long, simply supported, loaded at two locations, and with a shear-span of 9.125 in. (232 mm). Concrete compressive strength was 4.2 ksi (29 MPa). Failure of the FRP grating specimens without concrete started at center span with the formation of horizontal cracks in the longitudinal grating elements. These cracks propagated to one-third the grating depth at which point compression zone cracking occurred causing failure. The FRP concrete composite specimens initially started to crack in a manner similar to the noncomposite grating. Near the ultimate load, new cracks formed in the concrete compression zone followed by spalling at which point failure was defined. In composite sections with 1 in. (25 mm) of concrete deck, failure occurred as a result of combined concrete spalling in the compression zone and shear between concrete inside the grating and concrete above the grating. It was found that adding concrete to the FRP grating increased the load capacity by approximately 18 percent for concrete cast at the level of the FRP and by 300 percent for concrete cast 1 in. (25 mm) into the grating.

Faza and GangaRao (1992b) found predicted deflections of FRP-reinforced beams to be underestimated using the effective moment of inertia I_e as prescribed by Eq. 9-7 in ACI 318-89. The authors introduced a new method of calculating the effective moment of inertia of concrete beams reinforced with FRP reinforcement. The new expression is based on the assumption that a concrete section between the point loads is assumed to be fully cracked, while the end sections are assumed to be partially cracked. Therefore, an expression for I_{er} is used in the middle third section, and the ACI 318-89 I_e is used in the end sections. Using the moment-area approach to calculate maximum deflection at the center of the beam resulted in an expression for a modified moment of inertia as shown:

$$I_m = \frac{23 I_{cr} I_e}{8 I_{cr} + 15 I_e} \tag{5.6}$$

5.2.2 *Crack width and pattern*—Nawy and Neuwerth (1971) found that beams reinforced with steel had fewer cracks than the corresponding FRP reinforced beams. The large number of well-distributed cracks in the FRP-reinforced beams indicated good mechanical bond was developing between the FRP bar and surrounding concrete.

Faza and GangaRao (1992a) determined that concrete beams reinforced with spiral deformed FRP reinforcing bars using normal-strength concrete, 4000 psi (27.6 GPa), exhibited crack formation which was sudden and propagated toward the compression zone soon after the concrete stress reached its tensile strength. Crack spacing was very close to the stirrup spacing, and cracks formed at or near the stirrups, which were spaced at intervals of 6 in. This sudden propagation of cracks and wider crack widths decreased when higher strength concrete 7.5 to 10 ksi (5.17 to 69 MPa) and sand-coated FRP reinforcing bars were employed. Another important observation in specimens tested with sand-coated reinforcing bar and higher strength concrete is the formation of narrow cracks with smaller crack spacing. The crack patterns of beams reinforced with sand-coated reinforcing bars resembled the crack patterns expected in beams reinforced with steel reinforcing bars, with shorter spacing at ultimate levels.

Based on the assumption that maximum crack width can be approximated by an average strain in FRP reinforcing bar multiplied by expected crack spacing, this resulted in an expression for maximum crack spacing governed by the following parameters:

1) bond strength of FRP reinforcing bar
2) splitting tensile strength of concrete
3) area of concrete cross section in tension
4) number of reinforcing bars in tension
5) size of reinforcing bar
6) effective yield strength or working stress of FRP reinforcing bar

The resulting expression for maximum crack width is

$$W_{max} = \frac{\left(\frac{f_f}{E_f}\right) 2 f_t' A}{\mu_m \pi D} \tag{5.7}$$

where

$f_t' = 75\sqrt{f_c'}$

f_f = Maximum stress (ksi) in FRP reinforcement at service load level with $0.5 f_f$ to be used if no computations are available

A = Effective tension area of concrete surrounding the principal reinforcement divided by the number of reinforcing bars. It is defined as having the same centroid as the reinforcement (in.^{2})

μ_m= Maximum bond stres

D = Diameter of reinforcement

Note: reference not given in SI units

Benmokrane et al. (1994) compared the flexural behavior of concrete beams reinforced with GFRP bars to identical conventionally reinforced ones. Two series of 10.9 ft (3300 mm) long concrete beams were loaded with two concentrated loads applied at the third span points. The section of the beams of series 1 was 7.9 in. (200 mm) wide by 11.8 in. (300 mm) high and 7.9 in. (200 mm) by 21.7 in. (550 mm) for series 2. Each series consisted of two beams reinforced with two FRP bars 0.75 in. (19.1 mm) diameter and two others reinforced with two equal diameter steel bars. Tensile strength and modulus of elasticity of the FRP reinforcement were 101 ksi (700 MPa) and 6000 ksi (42 GPa), respectively. Compressive strength and modulus of concrete were 6.2 ksi and 47.8 ksi (33 GPa). At 25 percent M_u, the crack pattern and spacing in FRP reinforced beams were similar to those in conventionally reinforced beams. At service (50 percent M_u) and ultimate (90 percent M_u) loads, there were more and wider cracks than in the steel-reinforced beams. At service and ultimate loads FRP-reinforced beams experienced maximum deflection three times higher than for steel-reinforced beams. Predicted deflections using the Branson expression for effective moment of inertia I_e as prescribed in ACI 318-89, were underestimated. This is attributed to the width, depth, and spacing of the cracks. Based on experimental data a modified expression for the effective moment of inertia of a simply supported beam reinforced with FRP bars is:

$$I_{ef} = \alpha I_{cr} + \left(\frac{I_g}{\beta} - \alpha I_{cr}\right)\left(\frac{M_{cr}}{M_u}\right)^3 \qquad (5.8)$$

in which α and β are reduction factors equal to 0.84 and 7.0, respectively. These factors account for the reduced area of the compression section when the applied moment reaches M_{cr}.

5.3—FRP tie connectors for sandwich walls

Nonmetallic GFRP reinforcement tie connectors have been tested (Wade et al. 1988). These connectors have been primarily used for concrete sandwich wall systems. Many publications on these tie connectors are still proprietary restricted; however, more than 150 full-scale wall sections have been tested with various kinds of tie connector configurations. Tested wall sections have been prestressed, cast-in-place or reinforced precast systems ranging in span lengths from 8 to 20 ft. (2438 mm to 6096 mm). In addition, aging, pull-out, shear, flexure, and other elemental tests have been conducted on these connectors.

CHAPTER 6—PRESTRESSED CONCRETE ELEMENTS

This chapter summarizes research findings regarding the performance of FRP as a prestressing cable in concrete beams.

6.1—Strength of FRP prestressed concrete beams

FRP tendons are characterized by linear-elastic stress-strain behavior nearly to failure. Thus, failure of a concrete beam prestressed with FRP will occur either as the result of tensile rupture of FRP tendons or crushing of concrete. Tensile failure due to rupture of FRP tendons will occur progressively starting with the FRP tendon farthest from the neutral axis. This type of failure for a concrete beam prestressed with FRP is brittle compared to a similar beam reinforced with prestressing steel. The second type of failure, crushing of concrete, occurs when the concrete strain reaches ultimate before the ultimate tensile strain in the FRP tendons i reached. This mode of failure is comparable to the behavior of concrete beams over-reinforced with prestressing steel. Concrete members are generally under-reinforced with steel, so that the steel will yield before the concrete crushes, therein providing a ductile mode of failure.

6.1.1 *Flexural strength*—Tanigaki et al. (1989) examined the flexural behavior of partially prestressed concrete beam reinforced with braided aramid fiber reinforcing bars (FiBRA™). They used the FiBRA™ reinforcing bars both a post-tensioning tendons and as main reinforcement. Six T-beams were tested with a flange width of 30.2 in. (765 mm) and a total depth of 11.8 in. (300 mm). All beams had a 9.8 ft (3000 mm) clear span with an overhang of 11.8 in. (300 mm). In three beams, the prestressing force applied was 15 percent, 30 percent, and 45 percent of the tendon tensile strength. Also, the type of prestressing and main reinforcement consisted of FiBRA™ with and without silica-sand adhered to the surface and steel tendons. For all specimens, two straight prestressing tendons were placed 3.9 in. (100 mm) from the bottom face. The load was applied in five cycles obtaining a deflection at midspan of L/500, L/300, L/200 and L/100 at the consequent cycles, where L is the span of the beam, and then loaded to failure. The test results were characterized by the following set of findings.

1) For beams reinforced with FiBRA™ both as prestressing tendons and nonprestressing tendons, the post-crack load increased linearly up to failure. For the beam reinforced with steel as nonprestressed reinforcement, the load increased linearly only after the steel had yielded. This action reflects the linear stress-strain behavior characteristic of FiBRA™ up to failure
2) Crack spacing was about 3.9 in. (100 mm) for both beams reinforced with steel and FiBRA™ as prestressing elements. This indicated that the bond of the silica-sand-coated fiber rod with the concrete is similar to that of deformed steel bar
3) There was little difference in the flexural behavior of the beams with and without silica-sand adhered to the

surface of the FiBRA™ tendon

Tanigaki et al. (1989) carried out long-term bending tests to study the load resistance behavior of partially prestressed concrete beams with AFRP reinforcement. Four rectangular prestressed concrete beams with 11.8 in. (300 mm) depth, were tested. Braided AFRP reinforcing bars with and without silica-sand adhered to the surface were used to pretension three beams; the fourth beam was post-tensioned. AFRP reinforcing bars were also used as main reinforcement in all the beams. The initial prestressing force was altered in two beams. Concentrated loads, equal to P_{CT} (initial cracking load) and 1.5 P_{CT} were maintained for 1000 hours and the cracking and deflection of the beams recorded. After 1000 hours, the ratio of measured curvature and deflection to elastic estimates increased 5 to 8 times for beams loaded to P_{CT}, and about 10 times for the beam loaded to 1.5 P_{CT}. Gradual formation of new cracks was observed within the first 100 hours. During the remainder of the test, very few cracks formed and the changes in deflection with time became moderate. In all the specimens, concrete strain increased more rapidly than reinforcement strain, indicating that the neutral axis moved down with time.

Ductility of structures ensures they will not fail in a brittle fashion without warning and will be capable of absorbing large deformations at near maximum carrying capacity. The ductility of a reinforced concrete member is expressed as the ratio of the deformation at ultimate to the deformation at steel yield. The ductility may be expressed in terms of the curvature of a section (f_u/f_y) or in terms of the deflection (D_u/D_y) of a member.

FRP tendons do not yield but rather rupture suddenly, in a brittle failure. Thus, ductility of members prestressed with FRP tendons cannot be defined the same way as members prestressed with steel. Comparing the behavior of concrete beams prestressed with steel and FRP, at equal prestressing force, showed the same ultimate moment and ultimate deflection, if the failure is governed by concrete crushing. Moreover, unloading the beam with FRP just short of failure showed almost a complete recovery of the deflection, while a permanent set of deformation occurred for the beam with prestressing steel. If failure occurs by rupture of the FRP tendons, a similar beam prestressed with steel would fail at the same ultimate capacity. However, the ultimate deformation would be much less for the beam prestressed with FRP. Unfortunately, the design of prestressed concrete beams with FRP tendons under normal live loads may result in sections with low percentages of reinforcement. As a result the failure of these beams is governed by rupture of the tendons resulting in less deformation as compared to similar beams with prestressing steel.

Mutsuyoshi et al. (1991) reported the testing of six externally prestressed concrete T-beams using CFRP, AFRP and steel. The beams had a span of 8.2 ft (2.5 m), a depth of 15.7 in. (400 mm) and 11.8 in (300 mm) flange width. The cables were depressed at two points and the cable angle relative to the beam centerline was 7.1 deg for two beams and 11.3 deg for the remaining four beams. The prestressing in the cables was varied between 36 and 48 percent of its tensile strength. The beams with CFRP failed by crushing of concrete simultaneously with breaking of the cables. The breaking load of CFRP tendons, attached externally to the beams, was about 80 percent of the average breaking load obtained from uniaxial tensile tests. This was attributed to the weakness caused by the bending point in the cables.

Tests by Dolan (1991) examined the behavior of smooth Kevlar™ reinforced FRP tendons. The tests used 8 ft (2.4 m) long beams that were 20 in. (250 mm) wide and 4 in. (100 mm) deep. Both bonded and unbonded tendons were tested. The tests indicated excellent deflection recovery, even after several cyclic loads to near failure limits. The cracking and structural deformation were comparable to those found in steel prestressed beams, even though the tendons remained elastic. Failure was reported to be either crushing of the concrete or bond failure of the tendon.

Sen et al. (1991) monotonically tested six 8 ft (2.4 m) clear span pretensioned concrete beams simply supported an loaded at two points. Three of the beams were reinforced with GFRP tendons and three were reinforced with steel tendons. Three different cross-section sizes were used: 6 by 9 in. (152 by 203 mm), 6 by 10 in. (152 by 254 mm), and 6 by 12 in. (152 by 305 mm). Precrack response for beams reinforced with steel and GFRP tendons, having the same effective prestress, was identical. However, the post-crack response of samples reinforced with GFRP was more flexible than that of the beams reinforced with steel. At failure, cracks in the beams with GFRP were more widely spaced over the constant moment zone than in comparable beams pretensioned with steel. Tension failure accompanied by slip of the tendons occurred in the beams reinforced with GFRP.

Rider and Dolan (1993) reported that shear strength of beams could be increased using an externally bonded FRP fabric. Eight ft (2.4 m) long T-beams were pretensioned with FiBRA™ tendons. The beams were designed to have a shear failure less than the flexural capacity of the beam. After the control beam failed in shear, the second beam in the series was externally reinforced with a Kevlar™ fabric bonded to the beam with an epoxy. The reinforced beam was capable of developing the full flexural capacity of the section. Failure of the externally reinforced beams was by rupture of the tendon.

6.1.2 *Bond and development of reinforcement*—For prestressed construction the characteristics are influenced by the following:

1) Reinforcement internal stress
2) Method of transfer (sudden or gradual release)
3) Shape (circular or rectangular)

The tendon embedment length in the end zone of a pretensioned member in which the prestressing force is transferred to the member is called the transfer length (I_t). Within the transfer length, stresses in the tendons increase from zero, at the end of the member, to an effective stress (f_{se}) at the end of the transfer length (I_t). In order to develop the full design strength of the member, an additional flexural bond length (I_f) is required. To develop the design strength of the section, summation of the flexural bond length (l_f) and the transfer length (l_t) lead to the development length (l_d) of the prestressed tendon.

Iyer and Anigol (1991a) performed pull-out tests to study the bond characteristics of fiberglass cables and compared the results to data obtained from similar tests on steel and graphite cables. The research findings showed that the bond strength of advanced composite cables (fiberglass and graphite) was comparable to the bond strength of steel cables. Iyer et al. (1991b) also tested pretensioned concrete beams using fiberglass, steel and graphite reinforcing bars. Transfer length of the reinforcing bars was measured and found to be 37 d for fiberglass, 61d for steel and 59d for graphite cables. The effective prestress level of the tendons was 47, 48, and 44 percent of the tensile strength, for fiberglass, steel and graphite, respectively. Pull-out tests under a repeated load were performed on a carbon fiber composite cable (CFCC). Ten load cycles were applied with the maximum cyclic-load was equal to 60 percent of the simple pull out load of a CFRP cable. Bond strength was also measured. Results showed the bond strength of a 0.492 in. (12.5 mm) diameter CFRP cable to be higher than that of a 0.488 in. (12.4 mm) diameter prestressing steel strand. It has also been reported that the design bond length for 0.315 in. (8 mm) and 0.472 in. (12 mm) diameter CFRP Leadline™ reinforcing is 70d.

Nanni et al. (1992a) investigated the transfer length of an epoxy-impregnated braided aramid fiber (FiBRA™). The beam samples were 13.1 ft (4.0 m) in length and 4.7 in. (120 mm) by 8.3 in. (210 mm) in cross-section, having tendons of different size and number, surface conditions (sand-coated) and initial prestressing force were used. Transfer length of the AFRP tendon was affected mainly by its size and sand-coating on the tendon surface. The following is the set of study conclusions.

1) For a minimum concrete strength of 4.2 ksi (29 MPa) and initial prestress load not higher than 50 percent of the ultimate strength of the tendon, the unfactored transfer length for bonded AFRP tendons was found to be related to the nominal diameter as follows
   $l_t = 50\ d$, for $d = 0.315$ in. (8 mm)
   $l_t = 40\ d$, for $d = 0.472$ in. (12 mm)
   $l_t = 20\ d$, for $d = 0.472$ in. (12 mm) (if sand-coated)
   $l_t = 35\ d$, for $d = 16$ mm
2) The mechanism of force transfer in AFRP tendons is different from that of steel strands. It was found that the friction component of the transfer bond stress of AFRP tendons is higher than that of the steel. This could be a consequence of the lower rigidity of the AFRP tendons, which is about one-third that of the steel, and higher Poisson's ratio of the AFRP tendons, which was measured to be 1.65 that of steel.
3) Given the same tendon diameters, the transfer length in the steel strands is considerably higher than that of the AFRP tendons. It was also found that the transfer length of sand-coated tendons is significantly smaller as compared with smooth tendons.
4) Larger diameter tendons require longer transfer lengths. The use of two smaller diameter tendons requires a transfer length shorter than that for one larger tendon.

Nanni et al. (1992b) determined the development length by performing a monotonic flexural test on simply supported beams with a single concentrated load. The flexural bond length was determined based on the transfer length of AFRP tendons. A concrete strength of 4.4 ksi (30 MPa), concrete cover ranging between 1 to 2.1 in. (24 to 54 mm), a prestres force in the range from 25 to 50 percent of the ultimate tendon strength, and sand-coated tendons were used for the test beams.

A total of 21 beams were tested using three configurations based on the above mentioned parameters. Damage due to beam failure was limited to one end so that a second test could be performed on the opposite (undamaged) beam end. The different modes of failure were bond slip, bond slip accompanied by split cracking, concrete crushing, and combined concrete crushing and shear failure. The findings of this study were as follows:

1) Unfactored development length of the AFRP tendon was related to the nominal tendon diameter by
   $l_d = 120\ d$, for $d = 0.315$ in. (8 mm)
   $l_d = 100\ d$, for $d = 0.472$ in. (12 mm)
   $l_d = 80\ d$, for $d = 0.630$ in. (16 mm)
2) When the initial stress-to-ultimate nominal strength ratio was around 0.5, the ratio of flexural bond length l_f to transfer length l_t ranged between approximately 0.9 to 1.2
3) When comparing material/manufacturing effects of l_f for cables equal in size and prestressing, the shortest value is for a sand-coated FRP tendon and the longest for a steel strand
4) Increasing the tendon size results in shorter l_f, while higher initial prestress results in larger l_f

Studies on bond of various FRP prestressing tendons are also currently underway at the laboratories of the Federal Highway Administration (Thompson, et al., 1994).

6.1.3 *Prestress losses and fatigue strength*—When a cracked prestressed concrete member is subjected to repeated load applications, fatigue failure of the tendon may occur. Fatigue resistance is investigated by calculating the stres range f_p produced in the prestressing tendon under load cycling and comparing this stress range with that obtained fro the S-N curve for a particular prestressing system. The FI (1992) recommendations define the characteristic fatigue strength of prestressing steel as the stress range which can be resisted two million times, with the maximum stress going to 0.85 f_{py}, and a probability of failure equal to 10 percent.

The fatigue life of tendons in pretensioned beams is shorter than that of tendons tested in air. For loaded post-tensioned beams, curvature of the tendon profile causes the tendon to rub against the teeth of a given crack instigating premature failure. Special attention should also be paid to the fatigue resistance of the anchorage. Such devices can usually develop the full tendon strength under monotonic load conditions, but less than this value when a cyclic load is applied. Currently, the work done to define S-N curves for FRP tendons is very limited. Rostasy and Budelmann (1991) evaluated the S-N curves for GFRP bars. They reported the fatigue strength of FRP tendons is influenced by anchorage properties. However, the fatigue strength of GFRP is markedly below that of wedge-anchored prestressing wire.

Mikami et al. (1990) tested three prestressed concrete beams using braided AFRP tendons under cyclic load. The beams were of 7.87 in. (200 mm) by 9.84 in. (250 mm) in cross-section, each containing one pretensioned tendon, stressed at approximately 45 percent of its tensile strength. The clear span of the beam was 5.2 ft (1600 mm) with 15.7 in. (400 mm) projection from each side of the supports. Two beams having a ratio at one million cycle to initial loading was about 1.3. The third beam was loaded to 0.88 of its ultimate monotonic capacity and failed at 229,000 cycles.

Noritake and Kumagai (1991) reported the testing of two prestressed concrete beams of 32.8 ft (10 m) span using AFRP tendons. A parabolic-shaped cable with 19 reinforcing bars each 0.24 in. (6 mm) in diameter was used to post-tension the beams. The first beam was tested under a monotonic load. The second beam was cyclically loaded to bending moments of 0.45 M_u (bending moment at first crack), 0.55 M_u and 0.5 M_u, where M_u represents the ultimate moment of a similar beam tested under monotonic conditions. The tested beam survived two million cycles without failure. The sample was then loaded to failure and showed a 10 percent decrease in ultimate load carrying capacity. The anchorage of the AFRP cable was not damaged by the fatigue test.

Sen et al. (1991) reported the testing of two pretensioned concrete beams using GFRP reinforcing bars under a cyclic load. The beams were of dimensions 6 in. (152 mm) by 10 in. (254 mm) and 6 in. (152 mm) by 12 in. (305 mm) with an 8 ft (2400 mm) clear span. The GFRP reinforcing bars were initially stressed to 47 percent of their tensile strength. A sinusoidal load was applied at a frequency of 3 Hz and varied between 40 and 60 percent of the ultimate monotonic capacity of the beam, which was determined by monotonically testing a similar beam design. One of the two beams failed after the application of about 1.5 million cycles; the other beam survived two million cycles before it failed. Failure occurred suddenly due to loss of bond and slip of the GFRP reinforcing bars. The following conclusions were made from the study:

1) Overall fatigue characteristics of fiberglass pretensioned concrete beams matched those of similarly loaded steel pretensioned beams
2) Fatigue loading of fiberglass pretensioned beams in the post-crack range resulted in much higher deflection and crack widths than similar beams reinforced with prestressing steel (this was attributed to the low elastic modulus of the GFRP reinforcing bars)
3) Reduction in the ultimate capacity of the beams due to fatigue loading was very small compared to the corresponding monotonic capacity

The stress-strain response of FRP tendons depends upon the rate and time history of loading. If stress is held constant and strain increases, this is known as creep. When strain is held constant and stress decreases, this is known as relaxation. Creep and relaxation of FRP tendons differ greatly according to the type of fiber and matrix.

Fiber composites exhibit the phenomenon of creep-rupture. Therefore, their admissible stress must be chosen well below the creep-rupture strength to preclude reduction of the original strength. The long-term static strength for 100 years of load duration is predicted at about 70 percent of the tensile strength for GFRP and CFRP tendons (Rostasy, 1988). Aramid elements exhibit a slightly lower creep-rupture strength at 100 years

McKay and Erki (1992) examined the fatigue strength of prestressed concrete beams using AFRP reinforcing bars. Three concrete beams of dimensions 5.9 in. (150 mm) by 11.8 in. (300 mm) and 3.4 ft (1.05 m) clear span were tested. The AFRP reinforcing bars were initially stressed to 80 percent of their guaranteed tensile strength. The first beam was loaded in two stages. Stage 1 loaded the beam past cracking to near ultimate before the load was released. Stage 2 loaded the beam to failure. The other two beams were subjected to two static load cycles beyond the cracking limit. Then, the beams were subjected to sinusoidal loading at a frequency of 4 Hz. The maximum and minimum loads were set to simulate partially prestressed conditions by having the lower load just below cracking load, and the upper load producing a stress change in the AFRP rod of 29 ksi (200 MPa), with a maximum stress of about 80 percent of the guaranteed strength. The beams failed after 1.96 and 2.1 million cycles, respectively, by rupture of the reinforcing bars. The increase in the deflection for both beams was in the order of 10 to 20 percent of the original deflection. The following remarks were concluded from the study:

1) Fatigue strength of the AFRP reinforcing bars in service is at least as good as that for steel strands, under the stress conditions used in this investigation
2) Relaxation of AFRP reinforcing bars is higher than that for normal steel strands; a reasonable approximation of the relaxation losses for AFRP at initial stress in the 1200 Mpa (174 ksi) range can be calculated according to

$$\frac{f_p}{f_{pi}} = 1.009 - \frac{\log(t)}{65.1} \tag{6.1}$$

where

f_{pi} = fixing stress

t = time in min

Zhao (1993) conducted fatigue tests on 15 ft (4.6 m) long bonded and unbonded beams prestressed with CFCC cables. The beams were cycled between 40 and 70 percent of their ultimate static load. After one million cycles, the beams were monotonically loaded to failure. The pretensioned beams indicated no loss of static strength after the million cycles. Additionally the loss of stiffness, due to cracking of the concrete, appeared to be stabilizing. The unbonded beams indicated a fatigue deterioration of the concrete, however, no deterioration of the tendon capacity was noted.

6.2—Strength of FRP post-tensioned concrete beams

Mutsuyoshi et al. (1990c) tested ten post-tensioned concrete rectangular beams using carbon fiber reinforced plastic cables in seven beams, while prestressing steel was used in the others for comparison. The design prestressing force was varied from 40 percent up to 60 percent of the tensile strength of CFRP cables. Different surface preparation for CFRP cables were considered to alter the bond characteristics between the cables and the concrete, from bonded to unbonded cables. The beam dimensions were 5.9 in. (150 mm) by 7.9 in. (200 mm) by 4.9 ft (1500 mm) clear span with only one cable in each beam. All beams were simply supported, loaded at two locations and were tested monotonically to failure. Results of this study showed two failure modes, rupture of CFRP cables and crushing of concrete. It also showed that reducing the bond between the CFRP cables and concrete resulted in less ultimate capacity for the beams.

Yonekura et al. (1991) examined the flexural strengths and corresponding failure modes of concrete beams post-tensioned with carbon FRP (CFRP) and aramid FRP (AFRP) as prestressing elements. The area of prestressing tendons, initial prestressing force and type and area of axial reinforcement were varied. Eleven beams reinforced with CFRP and one beam reinforced with AFRP were tested. The behavior of the beams tested was compared to similar ones reinforced with prestressing steel. A total of sixteen beams of I section with total depth of 8.7 in. (220 mm) and flange width of 5.9 in. (150 mm) and a clear span of 4.6 ft (1400 mm), were simply supported, loaded at two locations and tested monotonically. Five levels of prestressing force were used ranging from 0 to 75 percent of the rupture load of the strands. Strains were recorded at different locations on the beams to indicate the strain distribution on the critical section for different load levels. The results of this study revealed the following:

1) Two classical failure modes of prestressed beams with FRP tendons were obtained: rupture of the tendons and crushing of concrete (the failure due to rupture of prestressing tendons could be avoided by arrangement of ample axial reinforcement)
2) Comparing a beam with prestressing steel to a similar one with FRP tendon, but with less initial prestressing force, the latter showed larger deflection after the crack initiation and less ultimate load (by increasing the prestressing force and the area of prestressing tendons, the difference in the ultimate load and ultimate deflection of the two beams became less and the failure mode became identical)
3) Increasing the magnitude of the prestressing force resulted in less deflection at the same load level and higher ultimate loads (the gradient of the post-crack load-deflection curves was unchanged)
4) In the case of beams with large amounts of prestressing steel, the strength of the beams increased only slightly even when the prestressing force was increased
5) Strains recorded in the beams with FRP reinforcing bars were larger than those with steel tendons, the neutral axis depths were less, since modulus of elasticity of FRP reinforcing bars were about $^1/_4$ to $^2/_3$ that of the prestressing steel bars

Taerwe and Miessler (1992) reported the testing of three post-tensioned concrete beams using GFRP (Polystal) tendons. T-beams, 23.6 in. (600 mm) depth and 11.8 in. (300 mm) in flange width and having a 6.6 ft (2000 mm) clear span were tested. The GFRP tendons were bonded to the concrete in two beams, while the tendons were left unbonded in the third beam. All beams failed by rupture of the tendons. Miessler (1991) also reported the testing of another T-bea having a clear span of 65.6 ft (20.0 m). The sample had a total depth of 39.4 in. (1000 mm) and flange width of 31.5 in. (800 mm). A GFRP tendon, consisting of 19 glass fiber bar 0.3 in. (7.5 mm) in diameter, was used to post-tension the sample. The tendon had a parabolic shape and was stressed to approximately 50 percent of its ultimate strength. The beam was simply supported and loaded at two locations to failure which occurred by rupture of the tendon. Strains in the GFRP tendon were monitored using optical fiber sensors.

CHAPTER 7—EXTERNAL REINFORCEMENT

The bonding of steel plates, using epoxy resins, to the tension zone of concrete beams is a method of improving structural performance. The technique is effective and has been used extensively in the rehabilitation of bridges and buildings. However, corrosion of the steel plates can cause deterioration of the bond at the glued steel-concrete interface, and consequently, render the structure vulnerable to loss of strength and possible collapse. The inherent corrosive property of ferrous materials has focused attention on FRP as a potential structural strengthening agent to be used in rehabilitation and post-tensioning applications.

Unidirectional FRP sheets made of carbon (CFRP), glass (GFRP) or aramid (AFRP) fibers bonded together with a polymer matrix (e.g., epoxy, polyester, vinyl ester) are being used as a substitute for steel. Initial developments in this area took place in Switzerland (Meier, 1987). FRP sheets offer immunity to corrosion, a low volume to weight ratio, and eliminate the need for the formation of joints due to the practically unlimited delivery length of the composite sheets. External post-tensioning FRP tendons have also been used to increase flexural member performance. This section discusses some of the preliminary findings and significant issues in this new construction technology.

7.1—Strength of FRP post-reinforced beams

Meier (1987) reported the use of thin CFRP sheets as flexural strengthening reinforcement of concrete beams; he showed that CFRP can replace steel with overall cost savings in the order of 25 percent. Kaiser (1989) load tested CFR composites on full scale reinforced concrete beams and showed the validity of the strain compatibility method in the analysis of cross-sections. It was suggested that inclined cracking may lead to premature failure by peeling-off of the strengthening sheet. The study included the development of an analytical model for composite plate anchoring, which was shown to be in agreement with test results.

Kaiser (1989) studied the temperature effect over 100 freeze-thaw cycles from +20 C to -25 C on concrete beams strengthened with CFRP and found no negative influence on the flexural capacity.

Plevris and Triantafillou (1993) developed an analytical model for predicting the creep and shrinkage behavior of concrete beams strengthened with various types of FRP plates. Using the model in the analysis of cross-sections, it was concluded that:

1) CFRP and GFRP affect the long-term response of strengthened elements
2) Increasing the area of these materials decreases the creep strains without affecting the time-dependent curvature, the tensile steel reinforcement stress, and the stress in the composite material
3) Strengthening with AFRP, a material that itself creeps considerably, increases the curvature due to creep without decreasing the creep strains
4) Increasing the AFRP area fraction gives a larger increase of the tensile steel reinforcement stress and a smaller decrease of the stress in the laminate over time
5) Increasing the area of FRP in general tends to restrain the reduction of stress in the concrete compressive zone

In terms of sensitivity to ultraviolet radiation, fatigue performance, tensile strength over time and low modulus of elasticity, CFRP laminates may offer the highest potential as a replacement of steel in strengthening applications [Triantafillou and Plevris (1992); Plevris and Triantafillou (1994)].

Ritchie (1991) tested a series of concrete beams strengthened with GFRP, CFRP, and AFRP, and developed an analytical method based on strain compatibility to predict the strength and stiffness of the plated beams.

Saadatmanesh and Ehsani (1991b) studied the static behavior of reinforced concrete beams with GFRP plates bonded to their tension zone. They concluded the following:

1) Concrete surface preparation and selection of the adhesive is of primary importance
2) Strengthening technique is particularly effective for beams with relatively low steel reinforcement ratios

Triantafillou and Plevris (1990, 1992) used the strain compatibility method and an analytical model for the FRP peeling-off mechanism based on the shearing dowel actions of both the steel reinforcement and the FRP plate to study the short-term flexural behavior of reinforced concrete beams strengthened with FRP laminates. The analytical results of failure mechanisms and corresponding loads were validated through a series of experiments employing thin CFRP sheets.

Plevris (1993) analyzed the flexure behavior of concrete beams strengthened with CFRP sheets. The concrete strength, CFRP failure strain, and CFRP area fraction were found to be the most influential on the variability of member strength. A reliability-based design procedure was also developed: two strength reduction factors were derived to achieve a reliability index of about three over a broad spectrum of design conditions. The analysis indicated a general strength reduction factor $f = 0.83$ and a partial reduction factor $f_{fc} = 0.94$ for the fiber composite strength. Through a more refined analysis, somewhat different sets of strength reduction factors were also obtained, depending on the characteristics of the design. In the last part of this study, the effect of each design variable on the reliability of the system was examined. It was concluded that the concrete, steel, and CFRP strengths, the steel and CFRP area fractions, and the ratio of live to dead load all have important effects on the reliability against flexural failure.

The time-dependent behavior of concrete beams strengthened with FRP plates was studied by Kaiser (1989), Duering (1993), and Plevris and Triantafillou (1993). In a series of tests, failure under fatigue loading was always initiated by rupture of the tensile reinforcing bars. This resulted in transfer of stresses from the reinforcing bars to the CFRP, which eventually failed as well. Hence, the flexural capacity of the members was controlled by the strength of steel under repeated failure. Creep experiments were performed to determine the effect of CFRP on the behavior of strengthened beams. It was concluded that the composite sheet can be modeled as a creep-free element perfectly bonded to the concrete.

The Kattenbusch bridge (Meier 1987), an eleven-span post-tensioned concrete bridge consisting of two hollow box girders was strengthened with GFRP plates. The bridge was built with working joints at the points of contraflexure, where wide cracks appeared several years after construction. Additional reinforcement to control the crack width and to reduce the tendon stresses was provided by strengthening eight joints with steel plates and two joints with GFRP plates (30 mm thick, 150 mm wide, and 3200 mm long) per box girder. The plates were bonded to the top face of the bottom flange.

In the Ibach bridge (Meier et al., 1992), a continuous, multi-span box beam, core borings performed in 1991 to mount new traffic signals damaged one of the tendons in the outer web. The bridge was strengthened with four CFRP sheets 5.9 in. (150 mm) wide and 16.4 ft (5.0 m) long. The sheets were epoxy-bonded to the tension face of the span. Approximately 14 lbs (6.2 kg) of CFRP were used in lieu of 385 lbs of steel.

7.1.2 *Prestressed plating*—Meier and Kaiser (1991) strengthened concrete beams using FRP sheets by prestressing the sheets before applying them to the concrete surfaces. Analytical models describing the maximum achievable prestress level, so that the FRP-prestressed system does not fail near the two ends (through shearing in the concrete) upon releasing the prestressing force, have been developed by Triantafillou and Deskovic (1991), and verified experimentally using small-scale specimens by Triantafillou et al. (1992). The results suggest that the method's efficiency (defined as the level of prestress at the bottom concrete fiber) is improved by increasing the thickness of the adhesive layer and/or increasing the are fraction of the FRP sheet. Clamping devices are needed at the ends of the composite sheets to provide confinement to the concrete and thus increase the prestressing force from 5-10 percent to about 50 percent of the sheet's tensile capacity. Such devices were developed by Deuring (1993) and consisted of pretensioned braided or unidirectional AFRP or CFRP straps, wrapped around the cross-section and anchored in the compression zone. Meier et al. (1992) and Deuring (1993) showed that the application of the method to large-scale specimens tested in flexure under static, fatigue and sustained loads demonstrated that pretensioning of the bonded element represents a significant contribution towards improving the serviceability of a concrete structure.

A variation of the technique described above was presented by Saadatmanesh and Ehsani (1991b). They introduced prestressing in both uncracked and precracked rectangular concrete beams by cambering the beams before bonding to their tension face a GFRP plate. The beams were then tested in bending and failed suddenly in shear through the concrete layer between the plate and the reinforcing bars.

7.2—Wrapping

In the mid-eighties, Ohbayashi Co. and Mitsubishi Kasei Co. developed the concept of strengthening and retrofitting existing RC structures using carbon fiber (CFRP) strands and tapes. Three types of structures were targeted: building columns (Katsumata et al., 1987; Katsumata et al., 1988); bridge columns (Kobatake et al., 1990); and chimneys (Katsumata et al., 1990). According to their method, CFRP strands impregnated with resins are spiral wound onto the surface of an existing RC member. In the case of bridge columns and chimneys, CFRP tapes may be glued first to the concrete in the longitudinal direction so that flexural strength is also enhanced. The primary function of the spiral wound strand is to improve shear capacity and ductility of the reinforced concrete member.

Experimental work to evaluate the potential of this method and the development of the first winding machine has been undertaken at the Technical Research Institute of Ohbayashi Co. Improvements in strength of 50 percent and maximu deformation ability up to four times greater than that of the original member were recorded using non-prestressed winding. Both circular and prismatic cross-section elements were investigated; however, test samples did not include conventional steel hoop or spiral reinforcement. Specimens were not subjected to axial load, only shear and bending moment were applied. Tests have shown that the low strain capacity for carbon fiber and its brittleness (even when epoxy impregnated) are a limiting factor. For prismatic elements, corners needed to be beveled prior to fiber winding (Kobatake 1989).

Nanni et al. (1993) reported an experimental and analytical study on the effect of wrapping conventional concrete compression cylinders, double-length compression cylinders, and $^1/_4$-scale column-type reinforced concrete specimens with different longitudinal/transverse steel reinforcement characteristics. The latter specimens are subjected to cyclic flexure with and without axial compression. The lateral FRP reinforcement consisted of a continuous flattened tube made of braided aramid fiber in one case, and in the other, of a continuous glass strand placed by a filament winding machine. The effects of different areas and spiral pitches for the tape, and thickness of the FRP shell for filament winding were investigated. Significant enhancement of strength and ductility were reported. Similar conclusion were obtained by Harmon and Slattery (1992) on smaller size concrete cylinders wrapped with carbon FRP strands. Li et al. (1992) have shown analytically the advantages of FR external wrapping on concrete strength and ductility.

7.3—External unbonded prestressing

Burgoyne (1992) reported the testing of two beams prestressed with Parafil ropes. The first beam had a single strength unbonded tendon contained within a duct on the centerline of a simply supported I beam of 15 ft (4.57 m) clear span and 15.7 in. (400 mm) depth. The second bea had two external deflected tendons, one on each side of a bulb T-shaped cross-section with 23.6 in. (600 mm) depth and 19.7 in. (500 mm) flange width. The clear span of the beam was 24.6 ft (7500 mm). Both beams were simply supported, loaded at two locations and taken through several elastic loading cycles, the second beam was kept under sustained load for 42 days to monitor the effects of creep and relaxation.

The elastic test of the first beam was designed to apply a load that would induce the allowable flexural tensile stress in the bottom fiber of the beam. When the load was removed, 94 percent of the midspan deflection was recovered. In the second cycle, after passing the cracking load, the stiffness was reduced considerably. When unloaded from the cracked-state (but still elastic), the stiffness remained low until the cracks had closed recovering the full elastic stiffness. At this point, there was virtually no permanent set when unloaded. When loaded to failure, curvature occurred with large cracks until failure by crushing of the top flange occurred.

The second beam was subjected to two loading cycles to service load. The sustained load was such that a small tensile strain was obtained at the bottom of the beam, but without cracking of the concrete. This load was maintained for 42 days. The total loss of prestress, due to shrinkage and creep of concrete and due to stress relaxation in the tendon, within 42 days and after the application of service load, was 11 and 12 percent in the first and second ropes, respectively. Most of these losses occurred one day after prestressing. The bea was then reloaded to failure which occurred by crushing of the top flange. As the tendons were external, they were removed and tested after collapse of the beam. The breaking force of the tendons was greater than the mean value, and even greater than the maximum value as observed in the tensile tests conducted on similar ropes. This phenomenon wa attributed to the greater extent of creep in the more heavily loaded filaments.

In recent years, attention has been given to strengthening other structural components with FRPs as well. One application is to provide lateral confinement for concrete columns by means of wrapping the entire length or portions of it in a jacket of composite material (Saadatmanesh 1994). Tests of large-scale bridge piers have demonstrated the effectivenes of this approach in increasing the strength and ductility of such members (Priestley 1992).

A further application is in masonry structures. Unreinforced masonry (URM) comprises the largest type of construction in existence worldwide. Due to the small tension and shear capacity of this material, URM buildings have been severely damaged in past earthquakes. Epoxy bonding a thin sheet of composite materials to the exterior surfaces of these walls will force the individual brick elements to act as an integrated system. The high tensile strength of composites can be utilized to increase the shear and flexural capacity of URM members significantly. Tests of URM beams have demonstrated that by proper design, it is possible to achieve the full capacity of the masonry at failure, and to obtain very large deflection before the ultimate capacity of the strengthened system is reached (Ehsani 1993).

CHAPTER 8—FIELD APPLICATIONS

Composite materials have been used in a variety of civil engineering applications with both reinforced and prestressed concrete. They are manufactured as reinforcing elements, as prestressing and post-tensioning tendons and rods, and as strengthening materials for rehabilitation of existing structures. Several new structures utilizing FRP reinforcement are currently underway by the West Virginia Department of Transportation and by the Florida Department of Transportation. One such application is a 52 m (170 ft) three-span continuous bridge deck reinforced with FRP reinforcing bars. This chapter describes FRP applications in concrete reinforcement. The projects are grouped under the method of application, either as reinforced concrete, prestressed concrete, or external reinforcement.

8.1—Reinforced concrete structures

Although FRP reinforcing bars and grids have not been widely used for concrete reinforcement, there have been a number of projects completed in the United States and Japan. GFRP reinforcing bars have been commercially used in more than 40 structures in the United States and Canada, including sea walls, chemical plants, concrete tanks, hospital MRI facilities, electrical sub-stations, architectural structures, and highway barriers.

8.1.1 *Applications in North America*—University building; San Antonio, Texas (Fig. 8.1)—GFRP reinforcing bars were used in the perimeter wall/beams and in a primary girder beam of a reinforced concrete floor system. The project, which required a non-ferrous structural environment, was constructed in 1986 and included girder beams, joists, and one-way slabs. The girder beams were designed to support maximum concentrated point loads of approximately 40 kN (9000 lbs).

Hospital buildings; San Antonio, Texas (Fig. 8.2 and 8.3)—In 1985, GFRP reinforcing bars were used in the construction of piers, columns, beams and joists for an MRI unit. A 1988 project used GFRP reinforcement for concrete pedestals supporting a large magnet.

Precast channel slabs; Atlanta, Georgia (Fig. 8.4)—Quality Precast Limited, Inc. developed a nonferrous channel slab reinforced with GFRP reinforcing bars. These channel slabs conform to a U.S. federal procurement specification calling for a service load of 3 kN/ ^{2} (65 psf), and a factor of safety of 4. The slabs are engineered utilizing the special properties

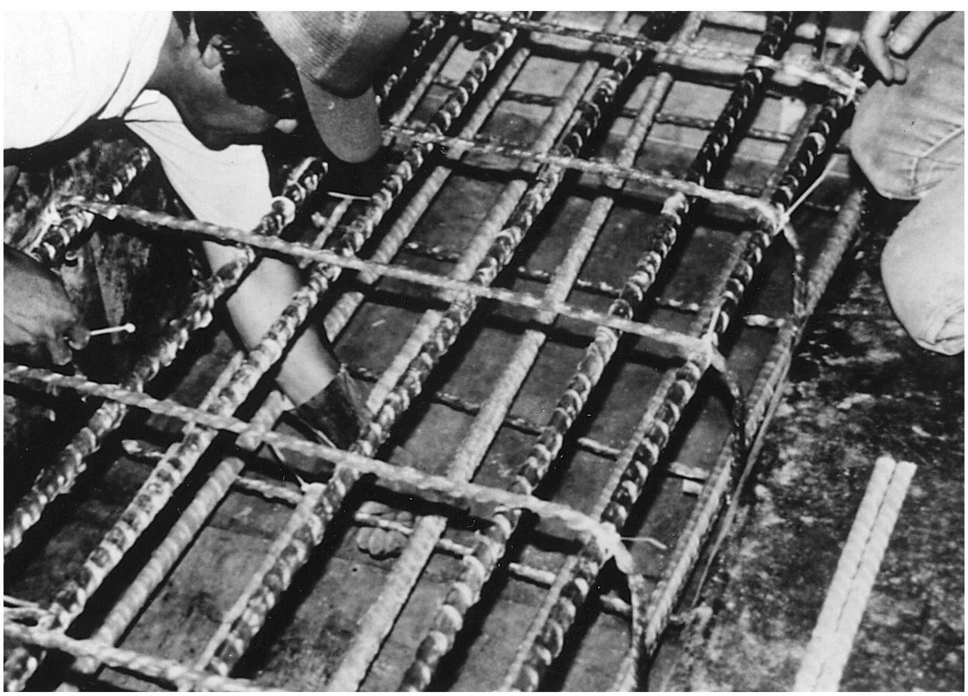

Fig. 8.1—University Building, San Antonio, Texas

Fig. 8.2—South Texas MRI, San Antonio, Texas

Fig. 8.3—Hospital building, San Antonio, Texas

of GFRP reinforcing bars. They are currently being used in industrial applications affected by such severe environmental conditions as high humidity, high temperatures, and corrosive atmosphere. Some of these applications are roof decks, special walls, floor slabs, tanks and chests, stair towers, and trench systems.

8.1.2 *Applications in Japan*—Partition panel —Partition panels for a chlorine gas storeroom in a purification plant were the first application of three-dimensional fiber reinforced composite (3D-FRC) planks in an actual structure. The 3D fabric was made of polyacrylonitrile (PAN) carbon fiber (48,000 filament) and the matrix was fiber reinforced concrete with 1.0 percent content of vinylon short fibers. The weight of a standard panel is 250 kg (550 lbm). Panels were installed over an area of 80 ^{2} (860 ft^2).

Parapet panels—The curtain walls of the Suidobashi building utilized a 3D-FRC reinforced concrete system with aramid fabrics (24,000 filament) for the X and Y axes and carbon fibers (12,000 filament) for the Z axis, in consideration of possible radio-wave interference. Displacement and

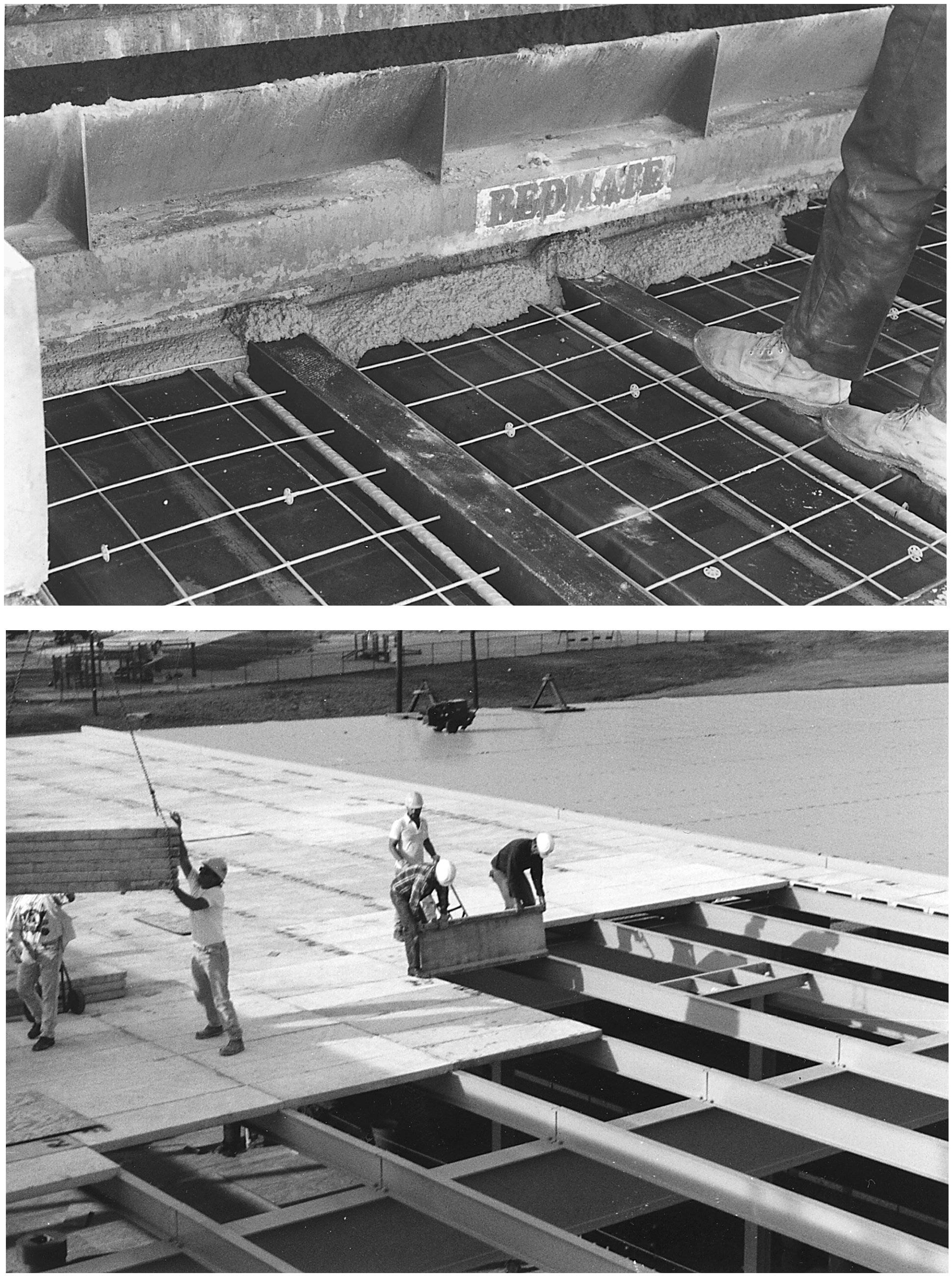

Fig. 8.4—Construction and placement of precast channel slabs, Atlanta, Georgia

strain of the specimen display elastic relationships with variation of wind load, indicating that 3D-FRC reinforced panels have stable deformation behavior and sufficient flexural strength and rigidity to withstand the design wind loads.

Curtain walls—3D-FRC was applied on a large scale, 1500 m^2 (16,000 ft^2), to the 23-story Sea Fort Square building in the sea front area of Shinagawa, Tokyo. A single unit of this tile-finished curtain wall consists of a 3D-FRC reinforced panel and a steel stud frame. In the original design aluminum panels were specified, but due to concern over salt-induced aluminum corrosion and because a significantly heavier wall material such as conventional precast concrete would not suit the load-carrying capacity of the steel skeleton, 3D-FRC was adopted.

Reinforcing grid for shotcrete (Fig. 8.5)—GFRP (NEFMAC™) grids are being used as substitutes for welded wire mesh for shotcrete applications in tunnel linings. In this case, consideration is given to GFRP features such as light weight, ease of handling, corrosion resistance, and better flexibility in following the shotcrete surface (Sekijima et al, 1990). One example of such an application is a concrete retaining wall at an underground liquid gas storage tank (Nagata and Sekijima, 1988).

8.2—Pretensioned and post-tensioned concrete structures

In this section, FRP strands and tendons for pretensioning and post-tensioning applications are highlighted.

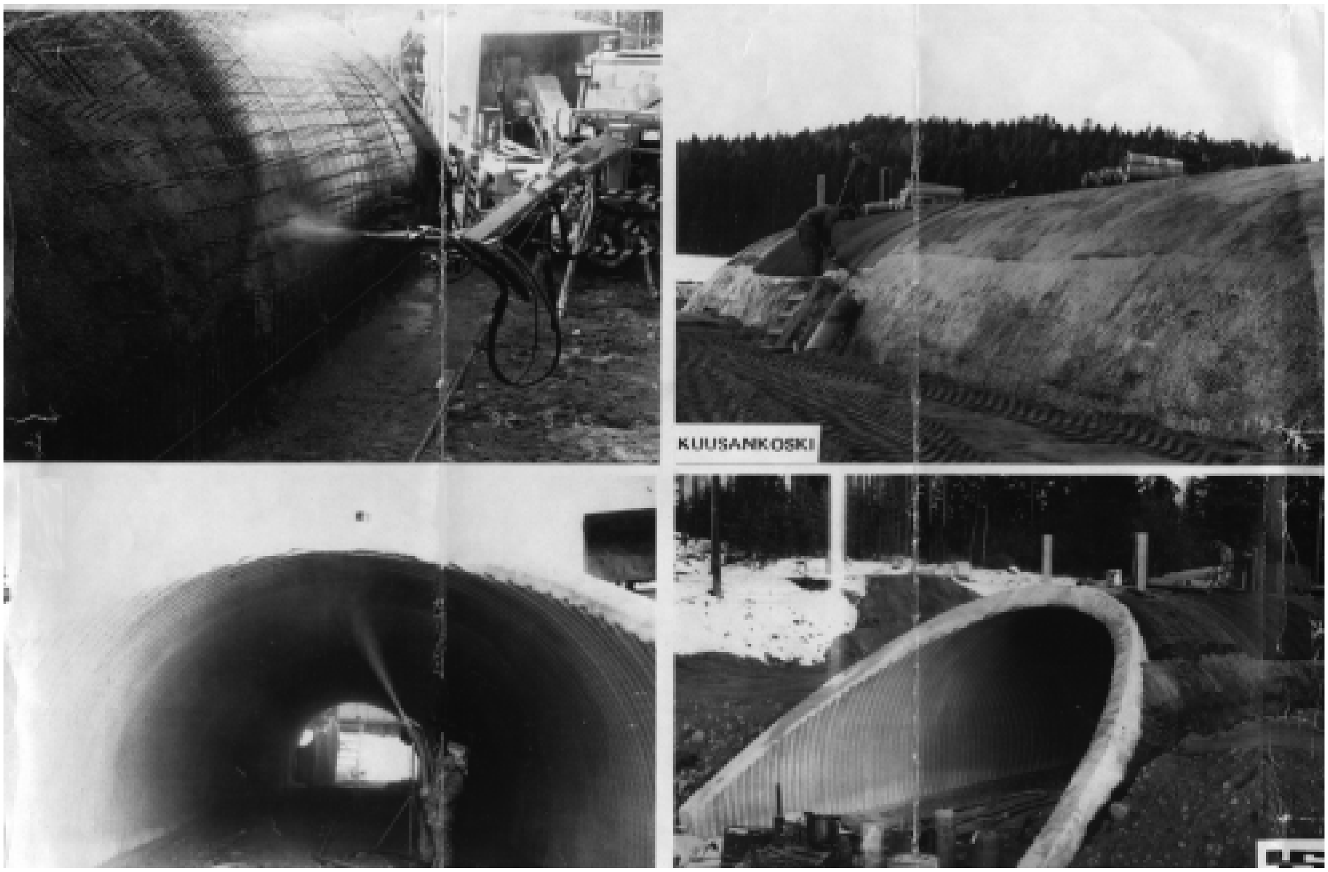

Fig. 8.5—Reinforcing grid, Japan

Fig. 8.6—Bridge at Calgary, Canada

8.2.1 *Applications in North America*—Calgary bridge, Canada (Fig. 8.6)—A concrete highway bridge was constructed using carbon fiber composite cables (CFCC) and Leadline™ tendons. The structure is a two-span continuous skew (33.3 deg) bridge with spans of 22.83 and 19.23 m (75 and 63 ft). Thirteen bulb-tee section precast prestressed concrete girders were used for each span. The girders are 1.1 m (43 in.) deep and have a 160 mm (6 in.) web thickness. Of the 26 girders, four (two in each span) were prestressed using CFCC cables 15.2 mm (0.6 in.) in diameter. Two additional girders (one in each span) were prestressed by 8 mm (0.3 in.) diameter Leadline™ rods. These six girders were located at the center of the bridge. The remaining girders were prestressed with steel strands. This bridge is monitored by an optical fiber system consisting of intra-core Bragg grating optic fiber sensors and electric strain gauges attached to CFCC, Leadline™ rods, and steel strands (Rizkalla, et al, 1994).

Rapid City Bridge, South Dakota (Fig. 8.7)—A precast post-tensioned bridge of 9 m (30 ft) span and 5.2 m (17 ft)

Fig. 8.7—Composite prestressed bridge at Rapid City, South Dakota

Fig. 8.8—Waterfront structure, Port Hueneme, California

width was erected at a cement plant in 1992 (Iyer, 1992). It has a 180 mm (7 in.) thick deck slab supported by three longitudinal girders. Cables of three different materials were used to prestress the slab. Thirty GFRP cables were used to prestress one-third of the length of the slab, 30 CFRP cables were used to prestress the next third of the length, and steel cable prestressing was adopted for the remaining length. Each GFRP and CFRP cable consists of seven 4-mm (0.156-in.) diameter rods. The initial prestress and final prestress after all losses were set at $0.6P_u$ and $0.5P_u$, respectively. Plastic ducts housing the cables were grouted with high-strength epoxy and mortar for bonding purposes, and the temporary anchorages were removed. Monitoring of bridge deflections and stresses in cables and concrete was carried out to assess losses in the cables and the actual deflections under moving loads.

Waterfront structure; Port Hueneme, California (Fig. 8.8)—This waterfront structure, constructed in 1994, consists of two full-scale bays; one is a prestressed deck with graphite cables, and the other is a fiberglass composite deck. The bays are supported by 12 356 x 356 mm (14 x 14 in.) piles, 13.7 m (45 ft) long, and reinforced with six prestressed CFRP cables and CFRP spirals. The pile caps were post-tensioned with GFRP (E-glass) cables. The deck was designed for 1000 kN (225 kips) applied on a 762 mm (30 in.) square area and tested at service load conditions. A total of 180 CFRP cables were used to prestress the 6 m (20 ft) long deck, which has a 5.5 m (18 ft) width and a 457 mm (18 in.) thickness.

8.2.2 *Applications in Japan*—Sun Land Golf Club building—The flat slabs of this building were post-tensioned using 15 mm (0.6 in.) diameter braided AFRP (FiBRA™) rods. The slab was designed based on controlling cracks and deflection. The post-tensioning is monitored by a load cell and oil pressure meter.

Omuro housing complex—Constructed in 1992, this

Fig. 8.9—Tsukude Golf Country Club bridge, Japan

building contains partially prestressed beams utilizing 13 mm (1 in.) diameter FiBRA™ tendons. The tension in the tendons is monitored by load cells. FRP rods are also used as a nonprestressed reinforcement.

Hakui cycling road bridge—Erected in 1991, this pretensioned hollow slab system utilizes CFCC strands as tendons (Minosaku, 1991). In addition to the prestressing tendons, straight CFRP wires were used as substitutes for diagonal tension reinforcing bars in one-half of the outside girders.

Tabras Golf Club bridge—Fourteen mm (0.55 in.) diameter braided AFRP (FiBRA™) rods were used in three of the 21 girders for this slab bridge constructed in 1990 (Tamura and Tezuka, 1990). The bridge is 2.40 m (7.9 ft) wide, with three 11.98 m (39.3 ft) spans, and conforms to the Japan Institute of Standards (JIS) specifications. The allowable tensile force in the AFRP rods was set at $0.5P_u$.

Takahiko floating bridge—Completed in 1992, this bridge was partially prestressed using 13 and 15 mm (1.0 and 1.2 in.) diameter FiBRA™ rods.

South Yard Country Club suspension bridge—Constructed in 1990, this bridge was post-tensioned with 4.86 by 19.5 mm (0.2 by 0.75 in.) flat AFRP (Arapree™) strips. The anchorage for the post-tensioning was provided using mortar-filled sleeves.

Iwafune Golf Club cable stayed-bridge—Constructed in 1992, this bridge was partially prestressed using GFRP and CFRP rods. Anchorage was provided by resin-filled sleeves. The initial prestressing was $0.3P_u$.

Tsukude Golf Country Club bridge (Fig. 8.9)—Constructed in 1993, this cantilevered-type pedestrian bridge has a single span of 99.0 m (325 ft) and a width of 3.6 m (12 ft). The main girders were post-tensioned with 12.5 mm (0.5 in.) diameter CFCC rods. The bridge was designed based on the Japan Road Association (JRA) specification for highway bridges.

Nakatsugawa pedestrian overbridge—CFCC strands were used in this pretensioned simple slab bridge built in 1989 (Hanzawa et al, 1989). The bridge was prefabricated in a single piece, and has a width of 2.5 m (8.2 ft) and a length of 8.0 m (26.2 ft). The anchorages used in fabrication were made of threaded steel pipes, with CFCC tendon ends inserted into the pipes and epoxy-injected. The allowable tensile force were $0.60P_u$ during prestressing, $0.55P_u$ immediately after prestressing, and $0.50P_u$ under service loads. This bridge wa fabricated as a nonmetallic structure, and CFRP reinforcing bars with surface ribs and lugs to improve the bond with concrete were used as stirrups and temperature reinforcement.

Birdie bridge (Fig. 8.10)—AFRP tapes were used in 1990 as tendons in a pedestrian bridge on a golf course (Kunichika et al., 1991). This was a post-tensioned suspended slab bridge 2.1 m (6.9 ft) wide and 54.5 m (179 ft) long. Eight AFRP tapes, each with a 4.86 by 19.5 mm (0.2 by 0.75 in) cross section, were bundled to make a single cable. A total of 16 cables were used, with allowable tensile forces for the cables set at $0.5P_u$ under initial force, and $0.33P_u$ under service load. The cables were anchored by a sleeve filled with an expansive mortar. AFRP tapes were also used as nonprestressed reinforcement in the slab. For ground anchors, cables consisting of nine 8-mm (0.3 in.) diameter CFRP rods were used.

Shinmiya bridge (Fig. 8.11)—This was the first application where carbon fiber composite cable (CFCC) strand were used as tensioning materials for a prestressed concrete bridge, constructed in October 1988 on a national highway. The span and width of the bridge are 5.76 and 7.0 m (19 and

Fig. 8.10—Birdie Bridge, Mito City, Japan

Fig. 8.11—Shinmiya Bridge, Japan

23 ft), respectively. Seven-wire 12.5 mm (0.5 in.) diameter CFCC strands were arranged into six tendons at the bottom flange and two tendons at the top flange. Allowable tensile forces were $0.60P_u$ during prestressing, $0.53P_u$ immediately after prestressing, and $0.45P_u$ under service loads. Epoxy-coated steel reinforcing bars were used for stirrups.

Bachigawa Minamibashi bridge—This 18.6 m (61 ft) single-span bridge was constructed in 1989. The design conformed to the JRA specification for highway bridges. The post-tensioning for the precast hollow girder used Leadline™ CFRP cable.

Sumitomo bridge—A road bridge, erected at the freight entrance to a concrete products plant, was reinforced with pultruded AFRP rods (Mizutani et al., 1991). This was a prestressed concrete road bridge consisting of a 12.5 m (41 ft) span pretensioned composite slab, and a 25.0 m (82 ft) span post-tensioned box girder. AFRP rods were used for all tendons in both spans. The pretensioned composite slab used AFRP as stirrups and reinforcing bars. AFRP cables consisted of three twisted strands for the pretensioned composite slab, and 19 and seven twisted strands, respectively, for the internal and external cables of the post-tensioned box girder. The allowable tensile forces were $0.8P_u$ during prestressing, $0.7P_u$ immediately after prestressing, and $0.6P_u$. under service loads.

Kitakyushu bridge—CFRP rods were used in 1989 in a prestressed highway bridge with two simply supported spans (Sakai et al, 1990). The width of this bridge is 12.3 m (40);

Fig. 8.12—Floating bridge, Japan

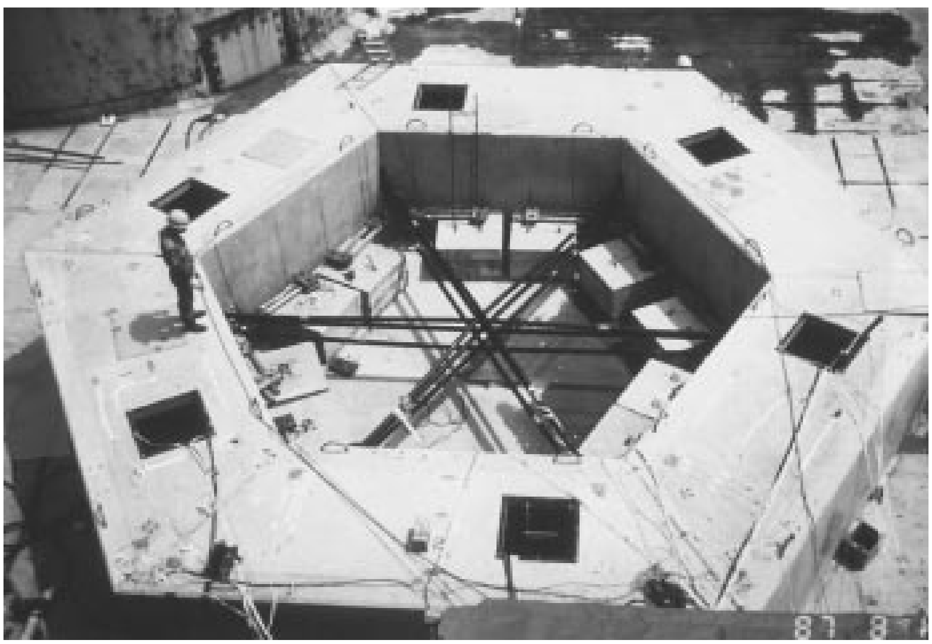

Fig. 8.13—Hexagon marine structure, Kanagawa, Japan

and the total length is 35.8 m (117.5 ft), with one span an 18.25 m (60 ft) pretensioned girder and the other span a 17.55 m (57.5 ft) post-tensioned girder. The tendons used were multi-cable bundles of eight 8-mm (0.3-in.) diameter CFRP rods. Eight multi-cables in all were used. The allowable tensile force under service loads was set at $0.55P_u$. Anchoring of the cables was achieved with a steel wedge-type anchor, with field monitoring in progress. The tensioning operation was conducted in three stages to reduce the difference in tensioning force between cables.

Maglev guideway—The precast side wall beams of this guideway are simply supported girders partially prestressed with 12.5 mm (0.5 in.) diameter CFCC strands. The nonmagnetic property of FRP materials was well utilized here. The FRP strand is twisted seven-wire impregnated in epoxy resin. Anchoring was obtained by direct bonding to concrete.

Kuzuha Quay landing pier—This structure was built in 1993 based on Japan's harbor structures design standards. Both pretensioning and post-tensioning tendons were CFCC strands of 12.5 mm (0.5 in.) diameter. End anchorage was obtained by direct bonding to concrete and by metal die-cast wedges.

Airport pavement—The nonmagnetic test pavement at Haneda International Airport has concrete pavement post-tensioned with CFRP and AFRP bonded tendons. The anchorage for post-tensioning was obtained using resin casting and wedges, and was not required to hold the permanent load.

Ichinoe water channel—This 30 m (98 ft) long and 4.5 m (15 ft) wide irrigation channel has precast concrete walls which are connected by post-tensioned AFRP tendons.

Marine structures (Fig. 8.12 and 8.13)—Because of their excellent corrosion resistance, the use of FRP products as reinforcing bars, prestressing tendons, and tie materials in port and harbor structures is being considered. An application of GFRP as the reinforcement for fenders attached to an existing pier has been reported (Tatsumi et al, 1989). Lattice-form GFRP was used as an alternative to epoxy-coated reinforcing bars for a hexagonal floating structure with six hollow pontoons joined together by tendons. The tendons consist of nine multi-cables, each made up of eight 8-mm (0.3-in.) diameter GFRP rods. The hexagonal floating marine structure shown in Fig. 8.13 was constructed in 1993. The concrete floating blocks were connected by post-tensioning CFRP tendons. The anchorage was provided using multi-type anchor heads and wedges.

Rock bolts and ground anchors—GFRP and AFRP rods and cables are being used as temporary rock bolts in mountain tunneling projects (Yamamoto et al, 1989; Sekijima et al, 1989-a). Although FRP materials are more expensive than conventional steel bolts, bolt shearing is significantly easier. AFRP rods have also been used as ground anchors for a temporary retaining wall.

8.2.3 *Applications in Europe*—Marienfelde bridge, Germany—A pedestrian bridge in a Berlin park was constructed in 1988 using external prestressing. The superstructure is a two-span double-T beam slab partially prestressed by seven FRP tendons. Spans are 27.6 and 22.9 m (90 and 75 ft); the double-T beam is 5 m (16.5 ft) wide and 1.1 m (3.6 ft) thick.

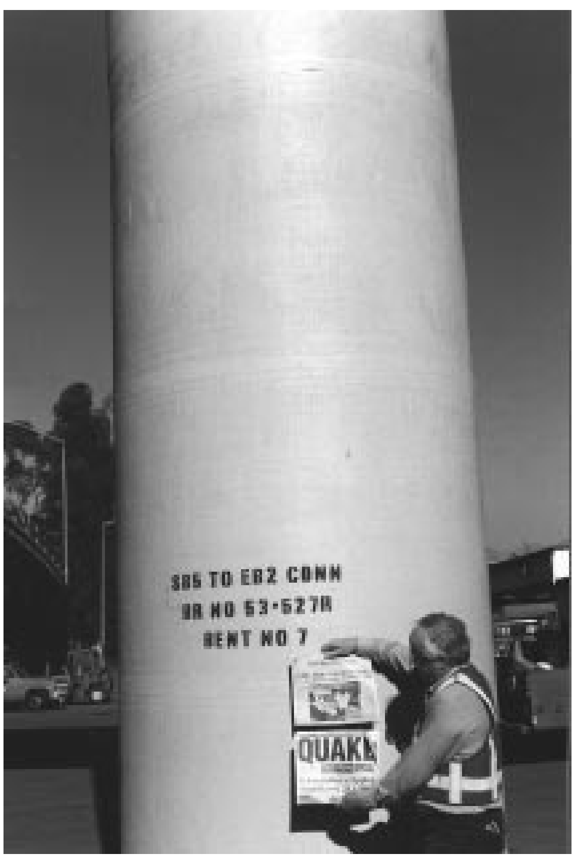

Fig. 8.14—Bridge column wrapped with GFRP,California

Ludwigshafen bridge, German—CFCC strands developed in Japan were used as part of the tensioning materials for a prestressed concrete bridge near a chemical plant. The bridge, which crosses a number of railway tracks, carries heavy truck traffic and was designed to a German road bridge specification requiring a 600/300 kN (135/67.5 kip) load rating. The two-lane bridge has a total length of 85 m (280 ft), and consists of four equal-length spans: two straight spans and two curved spans with a radius of 62.8 m (206 ft). The superstructure is modeled as a slab-and-beam bridge with a total width of 11.2 m (36.8 ft), and a height of 1.12 m (3.7 ft).

Lunen’sche-Gasse bridge, Germany—Polystal™ GFRP tendons were used in 1980 in a 6.55 m (21.5 ft) single-span slab bridge. One hundred GFRP rods 7.5 mm (0.3 in.) in diameter were grouped in unbonded prestressing tendons with a length of 7.00 m (23 ft). Four different anchorage systems were tested. Monitoring of the tensile forces, carried out over a period of 5 years on the grouted anchorage, demonstrated satisfactory performance.

Ulenberg-Strasse bridge, Germany—This post-tensioned bridge was erected in 1986 with a total of 59 multi-cables, each consisting of nineteen 7.5-mm (0.3 in.) diameter Polystal™ GFRP rods. The allowable tensile force of the cables under service loads was set at $0.47P_u$. Cables were anchored with a mortar made of silica sand and polyester resin. Synthetic resin mortar was injected as a grout to overcome the weakness of glass fiber to alkali attack. Similar to the “Lunen’sche-Gasse” bridge, E-glass fibers in a polyester resin matrix with external coating of polyamide were used. The slab bridge was designed for the German 600/300 kN (135/67.5 kip) load class, and has two spans which are 21.3 m (70 ft) and 25.6 m (84 ft) long.

Schiessbergstrasse bridge, Germany—This post-tensioned road bridge, designed for the German 600/300 kN (135/67.5 kip) load class, was built in 1990. This is a two-lane triple-span bridge with spans of 16.3, 20.4 and 16.3 m, (53.5, 67 and 53.5 ft). The slab width is 9.70 m (32 ft) and its thickness is 1.12 m (3.7 ft). A total of 27 FRP tendons were used. The bridge has a sophisticated permanent monitoring system with the possibility of on-line diagnostics.

Notsch bridge, Austria—This post-tensioned bridge, which has Polystal™ GFRP prestressing tendons, was built in 1991. It is a two-lane three-span bridge, with spans of 13.00, 18.00, and 13.00 m (42.7, 59, and 42.7 ft). The slab width is 12.00 m (39.4 ft), and its thickness is 0.65 m (2.1 ft). It has a permanent monitoring system similar to that on the Schiessbergstrasse bridge.

8.3—Strengthening of concrete structures

Strengthening of concrete structures by bonding FRP plates to concrete surfaces using polymer adhesives is becoming an effective method of improving performance under service and ultimate limit states. Traditionally, steel plates have been used, resulting in several disadvantages including difficulty in handling heavy plates at the site, possibility of corrosion at the adhesive-steel interface, and difficulty in forming clean butt joints between short plates.

Fig. 8.15—Columns strengthened with GFRP, Reno, Nevada

8.3.1 *Applications in North America*—Column wrapping projects, California (Fig. 8.14)—As a part of its general seismic upgrading program, the California Department of Transportation (Caltrans) placed confining jackets around bridge columns using fiberglass mat. Epoxy was used to provide the lap bond splices for the fiberglass mats. Expansive grout was injected beneath the mat to assure contact with the original concrete. Eleven 1.8 m (6 ft) diameter and four 1.2 m (4 ft) diameter columns were wrapped. These columns, which are located 32 km (20 miles) from Northridge, suffered no damage in the January 17, 1994 earthquake. This wrapping technique has been used on other projects in California. For example, the cities of Los Angeles and Santa Monica used composite wrapping materials on approximately 200 columns in 1993 and 1994.

Column wrapping projects; Reno, Nevada (Fig. 8.15)—In 1993, the Nevada Department of Transportation wrapped 96 0.3-m (3 ft) diameter columns with a proprietary FRP wrapping system. The columns were part of an interstate highway bridge constructed over a casino, with 48 of those columns actually located within the casino. No odor was detected from the TYFO™S epoxy.

Strengthening of walls; Glendale, California (Fig. 8.16 and 8.17)—Fiber composite fabrics can be epoxy-bonded to the surfaces of masonry or concrete walls to increase the strength of those elements. This technique was used in 1994 to repair damage caused by the Northridge earthquake to an exterior wall of a one-story building (Ehsani, 1995). The wall, constructed of 200 mm (8 in.) wide unreinforced masonry blocks, was severely cracked throughout an 18.3 m (60 ft) long by 6 m (20 ft) high section. Thin sheets of composite fabric were applied to both the interior and exterior faces of the wall. An additional application was on 175 mm (7 in.) wide tilt-up wall panels which were also damaged in the Northridge earthquake. The panels, which were 10 m (32 ft) high and either 6 m (20 ft) or 7.5 m (25 ft) wide, had horizontal cracks near midheight. Approximately 1850 m^2 (20,000 ft^2) of wall surface was strengthened by bonding thin sheets of composite fabric to both faces of the wall panels. The corners of the door openings were also strengthened with additional layers of fabric.

Fig. 8.16—Seismic retrofitting of unreinforced masonry wall in California

Fig. 8.17—Repair and strengthening of tilt-up concrete walls damaged during the 1994 Northridge earthquake

Foulk Road bridge; Wilmington, Delaware (Fig. 8.18)—Carbon fiber Forca™ tow sheets were used on this 16.5-m (54-ft) long, simple-span, prestressed, precast box beam structure that exhibited cracking indicative of the lack of transverse reinforcement. The bridge's superstructure was composed of 24 prestressed box beams placed adjacent to each other. For demonstration purposes, six of the beams were retrofitted. The design of the rehabilitation replicated the strength that 12.5 mm (0.5 in.) diameter steel bars would have provided had they been installed in the original casting of the beams. One ply of unidirectional CFRP sheet, with the fibers running transverse to the beam, was used on four beams. Two other beams used a higher modulus, higher weight fabric, with one of those beams fitted with two plies rather than a single ply.

8.3.2 *Applications in Japan*—Wrapping projects (Fig. 8.19, 8.20 and 8.21)—Forca™ tow sheet has been used extensively in Japan in over 200 projects including tunnels, chimneys, side walls, and slabs.

8.3.3 *Applications in Europe*—Ibach bridge, Switzerland—Accidental damage to a prestressing tendon during maintenance work necessitated repair of this bridge in 1991. Three 5 m (16.5 ft) long CFRP laminates, two with 150 by 1.75 mm (6 by 0.07 in.) cross sections and one with a 150 by 2.0 mm (6 by 0.08 in.) cross section were applied to the bottom surface of the bridge.

CHAPTER 9—RESEARCH NEEDS

FRP reinforcing bars and tendons are relatively new products and require extensive testing before they can be recommended for widespread application in concrete construction. This should not, however, preclude carefully controlled and

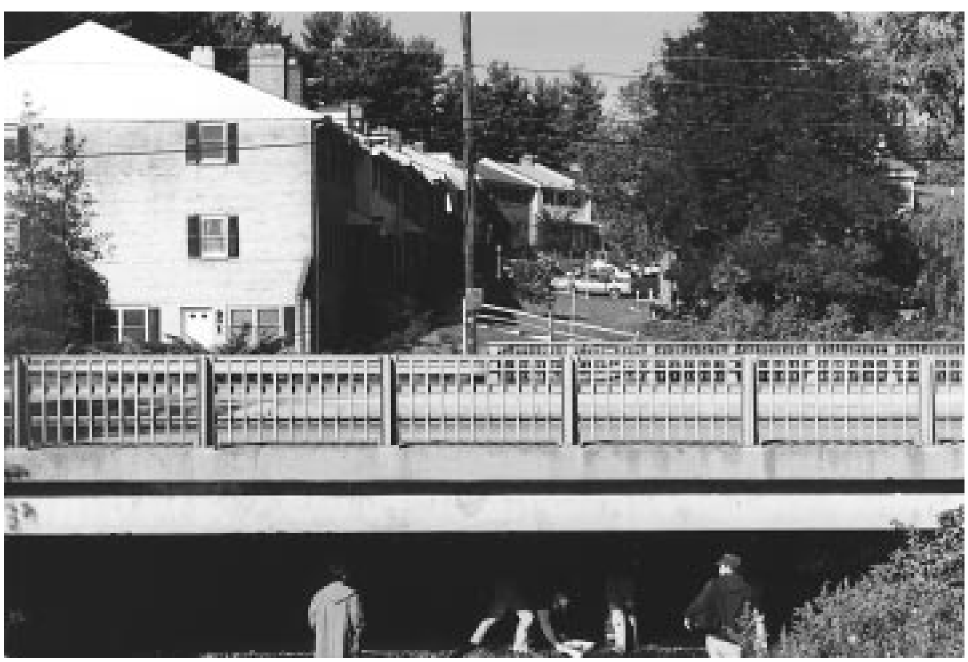

Fig. 8.18—Foulk Road bridge, Delaware

Fig. 8.19—Overview of bridge rehabilitation with Forca Tow Sheet, Japan

monitored demonstration projects. It is paramount that complete product reliability be assured in these projects.

It is anticipated that there will be significant obstacles and criticism in the introduction of these materials in concrete construction. Such questions and criticisms need to be considered in the overall research program, and efforts made to answer as many pressing questions as possible.

For the research results to be universally acceptable, it is imperative to standardize the test methods for evaluating such basic properties as ultimate strength, modulus of elasticity, elongation at failure, coefficient of thermal expansion, creep, relaxation, fatigue, bond, etc. Considering the dynamic nature of these materials and the fact that new commercial materials will frequently enter this field, standardization plays an important role in streamlining and effectively categorizing FRP reinforcements for inclusion in design specifications and standard codes.

The research needs of FRP reinforcing bars and tendons can be divided into three main categories: (1) materials behavior; (2) concrete element behavior; and (3) analysis and design guidelines. A separate section deals with the use of FRP sheets for external reinforcement of concrete structures.

9.1—Materials behavior

The overall research on materials' properties can be divided into two groups: mechanical and physical properties; and chemical properties.

9.1.1 *Physical and mechanical properties*—The following

Fig. 8.20—Application of Forca Tow Sheet inside tunnel, Japan

parameters related to the mechanical and physical characteristics of FRP require additional research:

Fiber/resin combination—Many different combinations of fiber and resin can be used in the manufacture of FRP. This is an advantage because it allows modification and enhancement of properties for a particular application. Significant research could be undertaken in this area to determine optimum fiber/resin combinations for development of FRP bars and tendons with desirable characteristics such as

Fig. 8.21—Overview of building wrapped with Forca Tow Sheet, Japan

strength, modulus, and durability. Cost should be considered in these developments.

Stress-strain relationship—FRP bars and tendons generally have higher strengths than steel. However, the modulus of elasticity of most FRP, particularly GFRP, reinforced materials, is less than that of steel. This shortcoming can pose a problem with regard to serviceability considerations and could limit the application of FRP. Therefore, studies are needed on basic properties of constituent materials, such as the resin and fibers, for improved stiffness of FRP bars and tendons.

Fatigue—Very little information is available on fatigue strength of FRP reinforcing bars and tendons. Because a major area of application will be in bridges, it is imperative that information on fatigue strength of the reinforcing elements be available to bridge designers. Again, studies are needed to determine the fatigue strength of different types of FRP tendons and bars under various levels of stress ranges. Fatigue studies need to be conducted under tension as well as stress reversal. If applicable, the results can be presented in standard forms such as S-N curves with fatigue limits clearly indicated for each type of FRP.

Relaxation—In pretensioned and post-tensioned applications, stress relaxation of FRP tendons becomes an important design parameter that needs investigation. Studies are needed to investigate stress relaxation in FRP tendons under different stress levels and lapsed times. The results can be categorized under different FRP groups having a particular fiber/resin combination as well as a specific fiber/resin ratio.

Creep—Designers have always been concerned with long-term deflections in concrete structures. This problem is perhaps more of a concern in FRP reinforced concrete structures due to the lack of information on long-term behavior of FRP bars and tendons. Studies specifically related to long-term performance of FRP bars and tendons under sustained loads need to be conducted. Because of the different constituent materials (resin and fiber) significantly different creep behavior can be observed for different FRP materials. This is inherently a problem associated with FRP studies, and therefore, methodologies need to be developed for categorizing and reducing the data in a manner that allows simple incorporation in design specifications and codes.

Creep rupture—Creep rupture is a phenomenon associated particularly with glass reinforced composites. This phenomenon relates to sudden rupture and the failure material under sustained loads. The nature of this behavior must be understood to provide adequate safety against its occurrence.

Thermal expansion—Thermal stresses play a major role on cracking and splitting of concrete. Basic knowledge on thermal expansion of various FRP reinforcing bars and tendons is required for effective crack control. In addition, studies are also needed on relative coefficients of thermal expansion of fiber and resin and their effect on mechanical properties of reinforcing bars and tendons.

Freeze-thaw cycles—Polymers are known to become brittle at low temperatures. This characteristic could adversely affect the properties of polymer composites. Therefore, research is needed to determine the effect of low temperature on FRP, as well as to determine any loss of strength under freeze-thaw cycles.

Bond and development length—Bond characteristics and development length are among the most important design parameters in reinforced concrete construction. Comprehensive studies are needed to determine the development lengths of FRP bars and tendons. These studies should include primary variables such as bar size, concrete compressive strength, bar shape (hooked versus straight), bar surface configuration, top bar versus bottom bar, etc. Transfer and development lengths of FRP tendons for application to prestressed concrete structures must also be investigated. It is noted that additional variables play important roles in bond and development length characteristics of FRP as compared to steel. For example, due to variations in ultimate strength, modulus of elasticity, surface texture, etc., different bond characteristics will be observed, significantly increasing the size of the data base on bond behavior. Methodologies need to be developed to efficiently synthesize all data generated from bond studies so that they can be incorporated in a simple manner in design codes.

9.1.2 *Chemical properties*—This is perhaps the area where the most pragmatic information is currently available. It would appear appropriate to secure and utilize data already complied by raw material manufacturers, both for reinforcement and resin.

FRP reinforcing bars and tendons are not prone to electrochemical corrosion and this attribute is the most important characteristic for construction applications. However, there are chemicals that could adversely affect the matrix, fibers, or both. Investigations need to be carried out to determine the effect of pertinent chemicals on FRP. The following are factors which affect FRP strength, and, therefore, require further investigation.

Fire—Perhaps the most frequently asked question and the least studied one is “How does FRP perform during a fire?” This is a good general question that needs to be answered with regard to specific application and specific type of FRP. Due to the wide range of constituent materials, significantly different fire ratings could be observed for different types of FRP. Polymers generally loose a significant part of their strength when subjected to high temperatures. Therefore, research is needed to determine the endurance limit of FRP during fire. Because most applications for FRP bars and tendons will have some form of concrete cover, tests are needed to establish a minimum cover to achieve a desired fire rating for concrete members reinforced with commonly used FRP bars and tendons.

Alkaline attack—In general, glass fibers used to reinforce polymer composites are susceptible to damage from alkaline attack. Because concrete is an alkaline environment, this issue may pose serious problems with regard to the durability of GFRP bars and tendons. Durable, impervious alkaline-resistant matrices could provide an answer to this problem. Other investigations could consider the use of alkaline-resistant glass or other types of alkaline-resistant fibers as alternative materials. Long-term studies are also required to examine the effect of alkalinity on resins and fibers commonly used to make FRP bars and tendons.

Acids—FRP bars and tendons are good candidates for application in structures located in aggressive environments. In order to establish the suitability of these types of reinforcement for applications in harsh environments, long-term studies are required to examine the durability of FRP bars and tendons under acidic environments.

Salts and deicing chemicals—It is anticipated that bridges are a major opportunity for FRP bars and tendons. In many cold regions, salt and other deicing chemicals are frequently used on bridges. Therefore, studies are needed to determine the effect of such chemicals on FRP bars and tendons. Furthermore, nonmetallic reinforcements are ideal candidates for application in structures located in coastal regions where corrosion of steel reinforcement has been a major problem. In this regard, long-term investigations of the effect of seawater on FRP reinforcements are required.

Ultraviolet radiation—Polymers, if not protected, generally degrade with time when subjected to UV light. Because most composites for construction applications contain a polymer matrix, it would be prudent to undertake studies on the effect of UV light on such polymers and on methods of protecting them against the deteriorating effects of UV light (additives or UV-proof coatings).

Environmental impact—In addition to technical aspects of the effects of environmental factors on FRP properties, environmental impact studies are also needed to address such issues as possible pollution from manufacture and application of FRP, disposal of by-products from manufacture and application of FRP, and the possibility of recycling of FRP.

9.2—Behavior of concrete members

The research needs of concrete members utilizing FRP reinforcement can be divided into two groups: (a) reinforced concrete; and (b) prestressed concrete.

9.2.1 *Reinforced concrete*—Several studies have been reported on concrete beams and slabs reinforced with FRP bars. Additional studies are required to create a database for development of design equations. In particular, the following areas require further investigation.

Serviceability—Due to the lower modulus of FRP reinforcements and tendons, deflection and crack control could present serious challenges to the designers. Comprehensive studies are required to examine not only the physical response of FRP reinforced structures with regard to deflection and crack control, but also to examine the existing philosophies related to these design aspects. Information on bond characteristics is important for crack width prediction and serviceability studies. Such information needs to be synthesized and developed into empirical relationships for limiting deflection and crack width.

Ductility—FRP reinforcing bars and tendons behave linearly elastic to failure, in other words, they do not yield. This characteristic of FRP could cause a problem with regard to ductility of reinforced concrete members. The implications of this behavior of FRP need to be addressed and incorporated in design guidelines. Additional studies could investigate the dynamic response of FRP-reinforced concrete members. Earthquake and dynamic response of concrete members are important design considerations in many parts of the world, and as a result, warrant a comprehensive investigation when it comes to structures reinforced with FRP.

Flexural strength—Studies are required to determine the flexural behavior of concrete beams and slabs reinforced with FRP throughout the entire range of loading up to failure. The effect of reinforcement ratio on failure modes needs to be investigated and recommendations given with regard to minimum reinforcement ratio to prevent rupture of the reinforcement. Because FRP reinforcement does not yield, it will result in different load-deformation history as compared with steel. Such load-deformation histories need to be generated and their implications in design guidelines discussed. Investigations should include use as compression as well as tension reinforcements.

Shear—Behavior of concrete beams reinforced with FRP stirrups needs additional investigation. Fewer studies have been reported on shear strength of FRP reinforced concrete elements compared to studies on flexural strength. Experimental and analytical studies are necessary to determine the required number of stirrups to prevent brittle shear failures in beams. Anchorage of stirrups, crack distribution, and stirrup spacing requirements are among additional topics that need further investigation.

Confinement—For applications where FRP will be used as reinforcement for concrete columns, information on the level of confinement provided by FRP ties and spirals will be required for developing proper design guidelines. Analytical and experimental studies are necessary to examine the effectiveness of FRP ties and spirals with regard to concrete confinement and to develop models that can quantify properties of concrete confined with FRP. Equations will be necessary to determine the required size, spacing, and amount of FRP ties and spirals for design of columns.

9.2.2 *Prestressed concrete members*—Many of the studies described above for FRP reinforcing bars need to be repeated also for FRP tendons. In addition to these studies, the following areas require special investigation for application to prestressed concrete structures.

Anchors—Among the most challenging problems for FRP tendon applications is the development of a suitable anchorage system. Due to their anisotropic behavior, FRP tendons generally have a much lower strength in transverse direction which makes gripping a difficult task. Development of anchorage grips that can be produced commercially at low cost will significantly facilitate introduction of FRP tendons in prestressed concrete construction.

Bond behavior and transfer length—For bonded tendon applications, development length and bond characteristics of tendons play important roles in the behavior of prestressed concrete structures. Therefore, comprehensive testing programs need to be conducted to establish the minimum required development length. Furthermore, for pretensioned concrete structures, transfer lengths need to be determined for FRP tendons. Here, again, sufficient number of tests need to be conducted to allow development of empirical relationships for determining the transfer length for each type of FRP tendon.

Stress limitations—Stress levels at jacking and transfer are important factors in design and construction of prestressed concrete members. Studies need to be undertaken to determine safe levels of stress at jacking and transfer of forces. Appropriate factors need to be developed with respect to the tensile strength of FRP tendons to determine these stress levels.

9.2.3 *External reinforcement*—External reinforcement of concrete structures with resin-bonded composite laminates and wraps is becoming increasingly popular with researchers and engineers in recent years. The following areas require special investigation for application of external reinforcement.

Composite material—An array of composite materials is available in the forms of tapes, fabrics, and sheets and can be used as wraps or laminates for strengthening of concrete structures. The strength and stiffness of the composite play important roles on the gain in strength and ductility of the retrofit structure. Studies are required to establish the most economical types of composites with sufficient strength, stiffness, and elongation at failure that result in an optimum overall performance of the retrofit structure.

Adhesive—The success of strengthening concrete structures by external reinforcement critically depends on the performance of the adhesive that bonds concrete to the composite reinforcement. For applications of this kind, various resins need to be evaluated and minimum required properties such as tensile and shear strengths, modulus of elasticity, toughness, and elongation at failure, be identified.

Interface behavior—The interface behavior between the external composite and the concrete is the most important factor affecting structural performance. Studies are necessary to evaluate this behavior for composite laminates as well as for composite wraps. Specifications need to be developed to quantify the gain in strength and ductility of an existing structure as a function of interface and bond performance. The effect of slip at the concrete/composite interface also needs additional investigation.

Durability—Long-term performance of the concrete/composite interface bond is a critical area requiring extensive investigation. Long-term and accelerated aging studies are required to examine bond strength under severe environmental conditions, as well as extreme temperatures and freeze/thaw cycles.

Fire—Because the external reinforcement in most cases might be exposed to fire, the effect of fire on the composite as well as on bond performance between concrete and composite requires investigation. Additives or fire protection measures should be identified to provide the required fire rating for the specific type of structure being retrofitted.

Flexural strength—External reinforcement by means of composite laminates can be used to enhance flexural strength of concrete elements. For this application, a composite laminate is bonded to concrete and acts as an added tension element enhancing the tensile component of the internal moment couple. Analytical and experimental studies are required to quantify the relationship between the amount of composite material added to the concrete element and the gain in flexural strength. The effect of the initial steel reinforcement ratio in this relationship needs to be evaluated.

Shear strength—External reinforcement can also be used to increase shear strength of concrete elements. For example, in concrete girders composite materials can be bonded to the web of the girder. Comprehensive analytical and experimental studies are required to establish relationships among initial shear reinforcement, the amount of composite materials to be bonded, and the required gain in shear capacity.

Fatigue—Strengthening by means of resin-bonded external reinforcement has a great potential for application to bridges. Investigation of the fatigue strength of resin-bonded joints consisting of concrete and composite materials is necessary to assure a safe and durable service.

9.3—Design guidelines

Many researchers have studied various aspects of the behavior of concrete members reinforced and/or prestressed with FRP reinforcing bars and tendons. However, to facilitate the introduction of nonmetallic reinforcement in practice, analysis techniques, as well as design specifications, are required. Therefore, reduction and synthesis of data and development of analysis techniques and design guidelines, that are verified by experimental data, should form a major portion of the overall research program. The inherent differences between the behaviors of FRP materials and steel could have major implications on design requirements. Therefore, extensive evaluation of existing design philosophies and their merits with regard to application to FRP-reinforced concrete structures will be necessary. These philosophies and axioms can then be modified to become suitable for application to FRP-reinforced and prestressed concrete structures. Furthermore, with the ever-growing trend in the universal application of the limit state design, reliability and probabilistic studies could be undertaken for FRP-reinforced structures. Load and resistance factors can then be developed for incorporation into codes and design specifications.

In conclusion, all efforts need to be focused on developing a design code for FRP-reinforced and prestressed concrete structures based on fundamental understanding of the behavior of these types of structures and a sufficient experimental database.

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APPENDIX A—TERMINOLOGY

This terminology section of the report lists only those common composites' terms that are referenced within this State-of-the-Art Report. For a more comprehensive list of composite technical terms and definitions, please reference publications on this subject by the American Concrete Institute (ACI Committee 116, 1990), American Society of Civil Engineers (SPRC 1984), and American Society of Material (ASM International 1989).

A

Aramid fiber—Highly oriented organic fiber derived fro polyamide incorporating aromatic ring structure.

AFRP—Aramid fiber reinforced plastic.

B

b-stage—Intermediate stage in the polymerization reaction of thermosets, following which material will soften with heat and is plastic and fusible. The resin of an uncured prepreg or premix is usually in b-stage.

BMC—Bulk molding compound.

Bar—Resin-bound construction made of continuous fibers in the shape of a bar used to reinforce concrete monoaxially.

Barcol hardness test—Test to determine degree of cure by measuring resin hardness (ASTM D 2583).

Binder—Chemical treatment applied to the random arrangement of glass fibers to give integrity to mats. Specific binders are utilized to promote chemical compatibility with the various laminating resins used.

Braided string or rope—String or rope made by braiding continuous fibers or strands.

Braiding—Intertwining of fibers in an organized fashion.

C

Carbon fiber—Fiber produced by pyrolysis of organic precursor fibers. Used interchangeably with graphite.

Carbon fiber, types—Mesophase pitch carbon; pan carbon (polyacrylonitrile).

Catalyst—Organic peroxide used to activate the polymerization.

CFRP—Carbon fiber reinforced plastic (includes graphite fiber reinforced plastic).

Commingled yarn—Hybrid yarn made with tow types of materials intermingled in a single yarn; for example, thermoplastic filaments intermingled with carbon filaments to form a single yarn.

Continuous filament—Fiber that is made by spinning or drawing into one long continuous entity.

Continuous-filament yarn—Yarn that is formed by twisting two or more continuous filaments into a single continuous strand.

Continuous roving—Parallel filaments coated with sizing, drawn together into single or multiple strands, and wound into a cylindrical package.

Continuous fiber reinforcement—Any construction of resin-bound continuous fibers used to reinforce a concrete matrix. The construction may be in the shape of continuous fiber bars, tendons or other shapes.

Coupling agent—Part of a surface treatment or finish which is designed to provide a bonding link between the fiber surface and the laminating resin.

Crimp—Waviness of a fiber, a measure of the difference between the length of the unstraightened and straightened fibers.

D

Denier—Measure of fiber diameter, taken as the weight in grams of 9000 meters of the fiber.

Doff—Roving package.

Durability—Ability of a system to maintain its properties with time.

E

Epoxy resin—Resin formed by the chemical reaction of epoxide groups with amines, alcohols, phenols, and others.

Extrusion—Process by which a molten resin is forced through a die of a desired shape.

F

Fabric—Arrangement of fibers held together in two dimensions. A fabric may be woven, nonwoven, or stitched.

Fabric, nonwoven—Material formed from fibers or yarns without interlacing. This can be stitched, knit or bonded

Fabric, woven—Material constructed of interlaced yarns, fibers, or filaments

FEM—Finite element modeling

Fiber—General term for a filamentary material. Any material whose length is at least 100 times its diameter, typically 0.10 to 0.13 mm.

Filament—Smallest unit of a fibrous material. A fiber made by spinning or drawing into one long continuous entity

Filament winding—Process for forming FRP parts by winding either dry or resin saturated continuous roving onto a rotating madrel

FRP—Fiber reinforced plastic.

G

GFRP—Glass fiber reinforced plastic.

Glass fiber—Fiber drawn from an inorganic product of fusion that has cooled without crystallizing.

Glass fiber, types—Alkali resistant (AR-glass); general purpose (E-glass); high strength (S-glass).

Graphite fibe —Fiber containing more than 99 percent elemental carbon made from a precursor by oxidation.

Gratin —Large cross-sectional area construction, usually in two axial directions, made up using continuous filaments.

Grid—Large cross-sectional area construction in two or three axial directions made up using continuous filaments

H

Hand lay-up—Fabrication method in which reinforcement layers, pre-impregnated or coated afterwards, are placed in a mold by hand, then cured to the formed shape.

Hardener—Substance added to thermoset resin to cause curing reaction. Usually applies to epoxy resins.

I

Impact resistance—Ability of a resin system to absorb energy when it is applied at high rates of strain.

Impregnation—Saturation of voids and interstices of a reinforcement with a resin.

Initiato —See catalyst.

Isophthalic polyester resin—High quality polyester resin (good thermal, mechanical, chemical resistance).

K

Knitwork—Construction made by knitting.

L

Laminate—Two or more layers of fiber, bound together in a resin matrix.

M

Matrix—Resin phase of fiber resin composite.

Mesh—Small cross-sectional area construction in two or three axial directions made up of continuous filaments.

Multifilament—Yarn consisting of many continuous filaments.

N

Nylon—Thermoplastic resin.

P

PET—Thermoplastic polyester resin (polyethylene terephthalate).

Phenolic resin—Thermoset resin produced by condensation of aromatic alcohol (high thermal resistance).

Pitch carbon fiber—Carbon fiber made from petroleum pitch.

Pan carbon fiber—Carbon fiber made from polyacrylonitrile (pan) fiber.

Polyester resin—Resin produced by the polycondensation of dihydroxy derivatives and dibasic organic acids or anhydrides yielding resins that can be compounded with styrol monomers to give highly cross-linked thermoset resins.

Prepreg—Semi-hardened construction made by soaking strands or roving with resin or resin precursors.

Pultrusion—Process by which a molten or curable resin and continuous fibers are pulled through a die of a desired structural shape of constant cross-section, usually to form a rod or tendon.

R

Reinforcement—Material, ranging from short fibers through complex textile forms, that is combined with a resin to provide it with enhanced mechanical properties.

Resin—Polymeric material that is rigid or semi-rigid at room temperature, usually with a melting-point or glass transition temperature above room temperature.

Roving—Two or more strands.

S

SCRIMP—Acronym for Seemans Composite Reinforcement Infusion Molding Process —a vacuum process to combine resin and reinforcement in an open mold.

Shape—Construction made of continuous fibers in a shape other than used to reinforce concrete mono-axially, or in the specific shape of a grid or mesh. Generally, not a bar, tendon, grid or mesh, although may be used generically to include one or more of these.

Sizing—Surface treatment or coating applied to filaments to improve the filament-to-resin bond and to impart processing and durability attributes.

SMC—Sheet Molding Compound.

Spray-up—Method of contact molding wherein resin and chopped strands of continuous filament roving are deposited on the mold directly from a chopper gun.

Spun yarn—Yarn made by entangling crimped staple.

Staple—Short fibers of uniform length usually made by cutting continuous filaments. Staple may be crimped or uncrimped.

Strand—Bundle of filaments bonded with sizing.

Synthetic fiber, types—Polyacrylonitrile (pan, acrylic); polyamide: nylon (aliphatic) and aramid (aromatic); polyvinyl alcohol (PVA); polyethylene (PE) (olefin).

T

Tenacity—Tensile strength of a fiber, that is the force to break divided by the cross-sectional area.

Tendon—Resin-bound construction made of continuous fibers in the shape of a tendon used to reinforce concrete mono-axially. Tendons are usually used in prestressed concrete.

Textile—Fabric, usually woven.

Thermoplastic—Resin that is not cross linked. Thermoplastic resin generally can be remelted and recycled.

Thermoset—Resin that is formed by cross linking polymer chains. A thermoset cannot be melted and recycled because the polymer chains form a three dimensional network.

Tow—Bundle of fibers, usually a large number of spun yarns.

Twisted string or rope—String or rope made by twisting continuous fibers or strands.

U

Uncrimped—Fibers with no crimp.

Unsaturated polyester—Product of a condensation reaction between dysfunctional acids and alcohols, one of which, generally the acid, contributes olefinic unsaturation.

V

Vinyl ester resin—Resin characterized by reactive unsaturation located primarily in terminal positions which can be compounded with styrol monomers to give highly cross-linked thermoset copolymers.

V-RTM (VA-RTM)—Acronym for vacuum resin transfer molding—a vacuum process to combine resin and reinforcement in an open mold.

Y

Yarn—Group of fibers held together to form a string or rope.

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